ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS
TCEQ PERMIT NO. MSW-1614B

MAJOR PERMIT AMENDMENT APPLICATION

VOLUME 4 OF 7

Prepared for

Pine Hill Farms Landfill TX, LP

May 2024

STATE OF TEXAS
JASON E. EDWARDS
99336
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

Prepared by

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Project No. 0120-076-11-106

This document is intended for permitting purposes only.

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PART III – SITE DEVELOPMENT PLAN
APPENDIX IIID
LINER QUALITY CONTROL PLAN

Prepared for

Pine Hill Farms Landfill TX, LP

May 2024

STATE OF TEXAS
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TABLES

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1 INTRODUCTION

1.1 Purpose

This appendix addresses §330.63(d)(4)(G), §330.337, §330.339, and §330.341.

This Liner Quality Control Plan (LQCP) has been prepared to provide the Operator, Design Engineer, Construction Quality Assurance Professional of Record, and the Contractor the means to govern the construction quality and to satisfy the environmental protection requirements under current Texas Commission on Environmental Quality (TCEQ) Municipal Solid Waste Rules (MSWR). More specifically, the LQCP addresses the soil and geosynthetic components of the liner system. The provisions of this LQCP were developed based on the latest technical guidelines of the TCEQ, including quality control of construction, testing frequencies and procedures, and quality assurance of sampling and testing procedures.

This LQCP is divided into the following parts:

- Section 1 – Introduction
- Section 2 – Construction Quality Assurance for Earthwork and Drainage Aggregates
- Section 3 – Construction Quality Assurance for Geosynthetics
- Section 4 – Construction Quality Assurance for Geosynthetic Clay Liner
- Section 5 – Construction Quality Assurance for Piping
- Section 6 – Liners Constructed Below the Highest Groundwater Level
- Section 7 – Geotechnical Strength Testing Requirements
- Section 8 – Documentation

1.2 Definitions

Whenever the terms listed below are used, the intent and meaning will be interpreted as indicated.

ASTM

The American Society for Testing and Materials

Construction Quality Assurance (CQA)

A planned system of activities that provides the Operator and permitting agency assurance that the facility was constructed as specified in the design. Construction quality assurance includes observations and evaluations of materials, and workmanship necessary to determine and document the quality of the constructed facility. Construction quality assurance (CQA) refers to measures taken by the CQA organization to assess if the installer or contractor is in compliance with the plans and specifications for a project.

Construction Quality Assurance Professional of Record (POR)

The POR is an authorized representative of the Operator and has overall responsibility for construction quality assurance that confirms that the facility was constructed in accordance with plans and specifications approved by the permitting agency. The POR must be registered as a Professional Engineer in Texas and experienced in geotechnical testing and its interpretations. Experience and education must include geotechnical engineering, engineering geology, soil mechanics, geotechnical laboratory testing, construction quality assurance and quality control testing, and hydrogeology. The POR must show competency and experience in certifying like installations, and be approved by the permitting agency, and be presently employed by or practicing as a geotechnical engineer in a recognized geotechnical/environmental engineering organization. POR or his designated representative will be on-site during all liner system construction. Reference within this appendix to the field inspection or monitoring obligations of the POR implies "the POR or designated representative under the supervision of the POR".

The POR may also be known in applicable regulations and guidelines as the CQA Engineer, Resident Project Representative, or the Geotechnical Professional (GP).

Construction Quality Assurance (CQA) Monitors

These are representatives of the POR who work under direct supervision of the POR. The CQA monitor is responsible for quality assurance monitoring and performing on-site tests and observations. The CQA monitor performing QA/QC observation and testing will be a qualified professional meeting one of the following qualifications: NICET-certified in geotechnical engineering technology at level II or higher for soils testing; a minimum of four years of directly related experience; a minimum of six months of directly related experience and has completed the Geosynthetic Institutes (GSI) Construction Quality Assurance Inspectors Certification Program (CQA-ICP); or a graduate engineer or geologist. Field observations, testing, or other activities associated with CQA may be performed by the CQA monitor(s) on behalf of the POR.

Additional CQA monitors may be used if they work under the direct supervision of a qualified CQA monitor who is on-site.

Contract Documents

These are the official set of documents issued by the Operator. The documents include bidding requirements, contract forms, contract conditions, specifications, contract drawings, addenda, and contract modifications.

Contract Specifications

These are the qualitative requirements for products, materials, and workmanship upon which the contract is based.

Contractor

This is the person or persons, firm, partnership, corporation, or any combination, private or public, who, as an independent contractor, has entered into a contract with the Operator, and who is referred to throughout the contract documents by singular number and masculine gender.

Design Engineer

These individuals or firms are responsible for the design and preparation of the project construction drawings and specifications. Also referred to as "designer" or "engineer."

Earthwork

This is a construction activity involving the use of soil materials as defined in the construction specifications and Section 2 of this plan.

Film Tear Bond (FTB)

A failure in the geomembrane sheet material on either side of the seam and not within the seam itself.

Geomembrane Liner (GM)

This is a synthetic lining material, also referred to as geomembrane, membrane liner, or sheet. The term Flexible Membrane Liner (FML) is also used for GM.

Geomembrane Liner Evaluation Report (GLER)

Certification report for the geomembrane liner, prepared and sealed by the POR that is submitted to the TCEQ for approval. Also referred to as flexible membrane liner evaluation report (FMLER).

Geosynthetic Clay Liner (GCL)

This is a synthetic lining material, which in the most basic form consists of bentonite sandwiched between two geotextiles. Also referred to as prefabricated bentonite blankets, mats or panels, or clay blankets, mats, or panels.

Geosynthetic Clay Liner Evaluation Report (GCLER)

Certification report for the geosynthetic clay liner, prepared and sealed by POR, which is submitted to TCEQ for approval.

Geosynthetics Contractor

This individual is also referred to as the "contractor" or "installer," and is the person or firm responsible for geosynthetic construction. This definition applies to any person installing FML or geotextile, even if not his primary function.

Independent Testing Laboratory

A laboratory that is independent of ownership or control by the permittee or any party to the construction of the liner system or the manufacturer of the liner system products used.

Manufacturing Quality Assurance (MQA)

A planned system of activities that provides assurance that the raw materials were constructed (manufactured) as specified.

Manufacturing Quality Control (MQC)

A planned system of inspection that is used to directly monitor and control the manufacture of a material.

Nonconformance

This is a deficiency in characteristic, documentation, or procedure that renders the quality of an item or activity unacceptable or indeterminate. Examples of non-conformances include, but are not limited to, physical defects, test failures, and inadequate documentation.

Operator

The organization that will operate the disposal unit.

Organics

Organic matter is material that may be capable of decay (e.g., plant material), the product of decay, or both.

Permittee's Representative

This is the person that is an official representative of the permittee responsible for planning, organizing, and controlling the design and construction activities.

Panel

This is a unit area of the FML, which will be seamed in the field.

Quality Assurance

This is a planned and systematic pattern of procedures and documentation to ensure that items of work or services meet the requirements of the contract documents. Quality assurance includes quality control. Quality assurance will be performed by the POR and CQA monitor.

Quality Control

These actions provide a means to measure and regulate the characteristics of an item or service to comply with the requirements of the contract documents. Quality control will be performed by the contractor.

Soil Liner Evaluation Report (SLER)

Construction report for the soil liner prepared and sealed by the POR and submitted to the TCEQ.

2 CONSTRUCTION QUALITY ASSURANCE FOR EARTHWORK AND DRAINAGE AGGREGATES

2.1 Introduction

This section of the LQCP addresses the construction of the soil and drainage components of the liner system and outlines the LQCP program to be implemented with regard to materials selection and evaluation, laboratory test requirements, field test requirements, and treatment of problems.

The scope of earthwork and related construction quality assurance includes the following elements:

- Subgrade preparation
- Soil liner stockpile
- Soil liner placement
- General fill
- Drainage aggregates
- Anchor trench backfill
- Excavation dewatering

2.2 Composite Liner

The landfill is designed to include a Subtitle D composite liner for the undeveloped liner area. The liner system for the undeveloped area will consist of a 2-foot-thick compacted clay liner and a 60-mil-thick high-density polyethylene (HDPE) Flexible Membrane Liner (FML). A GCL may be used in lieu of the 2-foot-thick compacted clay liner.

The liner systems are detailed in Appendix IIIA – Landfill Unit Design Information. A structural stability analysis for the liner system, including calculations for anchor trench runout lengths, stress on the liner components, and an interface slope stability analysis, is included in Appendix IIIE – Geotechnical Report.

2.3 Earthwork Construction

The following paragraphs describe general construction procedures to be used for various earthwork components within the landfill. The earthwork construction specifications will be developed based on the material and construction procedures outlined in this section of the LQCP for each specific liner construction. The earthwork construction specifications will include details for compaction of soils and cross sections showing typical slopes, widths, and thicknesses for compacted lifts.

2.3.1 Subgrade

Subgrade refers to a surface which is exposed after stripping topsoil or excavating to establish the grade directly beneath the composite liner. The prepared subgrade must conform to the Excavation Plan included in Appendix IIIA – Landfill Unit Design Information.

Prior to beginning liner construction, the subgrade area will be stripped to a depth sufficient to remove all loose surface soils or soft zones within the exposed excavation. The liner subgrade area will be proof rolled with heavy, rubber-tired construction equipment to detect unstable areas. Unstable areas will be undercut to firm material and refilled with suitable compacted general fill. Soil used for backfill will meet the same material requirements as the soil liner and will be installed in accordance with the soil liner installation procedures. The fill will be free of organic matter, foreign objects, and other deleterious matter, and compacted sufficiently to provide a firm base for composite liner placement. The subgrade will also be scarified a minimum of 2 inches prior to placement of the first lift of soil liner. The subgrade preparation specifications for each liner construction event will be developed in accordance with this section. Construction project specifications and construction plans will be developed for each cell construction event in accordance with this LQCP and consistent with the Excavation Plan (included in Appendix IIIA) and the sector design as contained in the approved Site Development Plan.

Subgrade voids and cracks are expected to be minor. However, the subgrade will be re-worked as necessary to provide a foundation suitable for composite liner placement. Visual examination of the subgrade preparation by the CQA monitor will generally be sufficient to evaluate its suitability as a foundation for the composite liner. The CQA monitor may find that physical testing is necessary to evaluate the prepared subgrade or fill placed in large voids.

The POR will approve the prepared subgrade prior to the placement of composite liner or underdrain. Approval will be based on a review of test information, if applicable, and CQA monitoring of the subgrade preparation. Additionally, during the subgrade acceptance, the POR will verify that the underlying material is consistent with the geotechnical design assumptions included in Appendix IIIE.

Surveying will be performed to verify that the finished subgrade is to the lines and grades specified in design with a vertical tolerance of -0.2 feet to +0.0 feet to ensure that the soil liner will achieve a 2-foot minimum thickness. The surface slope of the top layer of composite liner will conform to the slope requirements of the leachate collection layer.

2.3.2 Soil Liner

The soil liner will consist of a minimum 2-foot-thick compacted clay liner (measured perpendicular to the subgrade surface) that will extend along the floor and side slopes of the landfill. The soil liner will be constructed in continuous, single, compacted lifts (6 inches thick) parallel to the floor and sideslope subgrades. A GCL may be used in lieu of the 2-foot-thick compacted clay liner. Details depicting the liner system are included in Appendix IIIA – Landfill Unit Design Information.

2.3.2.1 Soil Borrow Material

Adequate soil liner material will be available from proposed landfill excavations and/or on-site or off-site borrow sources. The liner soil will be free of debris, rock greater than 1 inch in diameter, vegetative matter, frozen materials, foreign objects, and organics. Laboratory tests will verify that materials are adequate to meet the compacted clay liner requirements listed in §330.339(c)(5) prior to liner construction.

Soils used in soil liners will have the following minimum values verified by testing in a soil laboratory prior to liner construction.

Table 2-1
Required Borrow Soil Properties

Test[1]	Specification
Coefficient of Permeability (Remolded Sample)[2]	$1.0\text{x}10^{-7}$ cm/s or less
Plasticity Index	15 minimum
Liquid Limit	30 minimum
Percent Passing No. 200 Mesh Sieve	30 minimum
Percent Passing 1-inch Sieve	100

[1] Testing will be performed in accordance with the test methods included in Section 2.4.
[2] The coefficient of permeability for remolded sample is run at a minimum of 95% of the maximum dry density at or above the optimum moisture content.

Representative preliminary sampling and testing will be performed on on-site soils to be used as liner material or on off-site borrow source material. The CQA monitor, Earthwork Contractor, and/or Operator will identify the clay material in on-site stockpiles or during excavation, and the clay material will be stockpiled separately, if stockpiling is required. Prior to construction of each new cell, conformance tests that include liquid limit, plasticity index, percent passing the No. 200 and 1-inch sieves, Standard Proctor (ASTM D 698) compaction test, and coefficient of permeability test will be performed for each material proposed for each individual liner construction. The coefficient of permeability test specimens will be prepared by laboratory compaction to a dry density of approximately 95 percent of the Standard Proctor maximum dry density at or above the optimum moisture content. One Proctor moisture-density relationship and remolded coefficient of permeability test will be required for each different material. Additional conformance tests will be conducted if there are visual changes (color, texture, etc.) in borrow material or as determined necessary by the POR. The soil is considered as a separate soil borrow source if the liquid limit or plasticity index is determined to vary by more than 10 points. The liquid limit and plasticity index testing will be performed on the separate borrow source as an initial determination. If the liquid limit or plasticity index varies by more than 10 points then all other testing listed in Table 2-1 will be performed on the separate borrow source.

The physical characteristics of the liner materials will be evaluated through visual observation before and during construction. To adjust moisture of the material properly, any clod sizes will first be crushed into manageable sizes of 4 inches in diameter or less. Rocks within the compacted liner must be less than 1 inch in diameter. Soil clod size will be reduced to the smallest size necessary to achieve the coefficient of permeability reported by the testing laboratory. Additionally, the rock content of the soil liner will not be more than 10 percent by weight. Water used for the soil liner moisture adjustment must be clean and not contaminated by waste or any objectionable material. Stormwater collected on-site may be used if it has not come into contact with waste.

2.3.2.2 Liner Construction

The soil liner material will be placed in maximum 8-inch-thick loose lifts to produce compacted lift thicknesses of approximately 6 inches. The soil liner will have elevations, slopes, thickness, and widths as depicted on the Excavation Plan and Liner System Details in Appendix IIIA – Landfill Unit Design Information. A temporary hydrostatic pressure relief system will be installed as discussed in Appendix IIID-C.

The soil liner material will be compacted to a minimum of 95 percent of the maximum dry density at or above the optimum moisture content as determined by Standard Proctor (ASTM D 698). The soil liner must be compacted with a pad/tamping-foot (preferable) or prong-foot (sheepsfoot) roller. The lift thickness will be controlled so that there is total penetration through the loose lift under compaction into the top of the previously compacted lift; therefore, the lift thickness must not be greater than the pad or prong length. Use of pad/tamping-foot or prong-foot rollers will provide sufficient roughening of liner lifts surface for bonding between lifts. These procedures are necessary to achieve adequate bonding between lifts and reduce seepage pathways. Adequate cleaning devices must be in place and maintained on the compaction roller so that the prongs or pad feet do not become clogged with clay soils to the point that they cannot achieve full penetration during initial compaction. The footed roller is necessary to achieve this bonding and to reduce the individual clods and achieve a blending of the soil matrix through its kneading action.

In addition to the kneading action, weight of the compaction equipment is important. The minimum weight of the compactor should be 40,000 pounds (in no case should ground pressure be less than 1,500 lbs per linear foot for each drum or wheel length), and a minimum of four passes are recommended for the compaction process. A pass is defined as one pass (1 direction) of the compactor, not just an axle, over a given area. The recommended minimum of four passes is for a vehicle with front and rear drums. The Caterpillar 815B and 825C are examples of equipment typically used to achieve satisfactory results. The soil liner will not be compacted with a bulldozer or any track-mobilized equipment unless it is used to pull a pad-footed drum which is at a minimum 1,500 lbs per linear foot of drum length.

During the construction of continuous liners, the new liner segment will not be constructed by "butting" the entire thickness of the new liner directly against the edge of the old liner. The tie-in will be constructed by a sloped transition (typical 5 horizontal to 1 vertical) as shown in Appendix IIIA – Landfill Unit Design Information. The length of the tie-in must be at least 5 feet per foot of liner thickness. The tie-in will be scarified prior to placement of the next lift.

CQA testing of the soil liner will be performed as the liner is being constructed. Testing of the soil liner is addressed in Section 2.4. Soil liner construction and testing will be conducted in a systematic and timely fashion on each lift. Delays will be avoided in liner construction. Construction and testing of the soil liner will generally not exceed 60 working days from beginning of liner installation to completion. The TCEQ will be notified during construction if delays in excess of 60 days are anticipated. Reasons for liner construction taking more than 60 days to complete will be fully explained in the SLER submittal.

The finished surface of the final lift of soil liner must be rolled with a smooth, steel-wheeled roller to obtain a hard, uniform, and smooth surface. The surface of the final lift of soil liner will then be inspected by the CQA monitor. All undesired materials will be removed from the liner surface, and any voids created by removing undesired materials will be backfilled with liner material to the density specifications outlined for liner construction and tested at the discretion of the CQA monitor. Surveying will be performed to verify that the finished top of liner grade is to the lines and grades specified in construction plans for a particular cell. Top of soil liner surveying will be performed within a tolerance of 0.0 feet to +0.2 feet. The surface slope of the top layer will conform to the slope requirements of the leachate collection layer. Survey frequency is included in Table 2-2.

The POR will submit to the TCEQ a SLER for approval of each soil liner area. This LQCP has been developed in accordance with the TCEQ regulations. The requirements for testing and evaluation of the soil liner during construction are included in this LQCP. The construction methods and test procedures documented in the SLER will be consistent with this LQCP and TCEQ regulations.

The soil liner will be prevented from losing moisture during the SLER approval process. Preserving the moisture content of the installed soil liner will be dependent on the earthwork contractors means and methods and is subject to POR approval.

Upon completion of liner construction, SLER markers will be installed to clearly indicate the limits of constructed and approved liner areas in accordance with Section 4.7 – Landfill Markers and Benchmark of the approved Site Operating Plan. SLER markers will be located so that they are not destroyed during operations. The POR will document in the GLER that SLER markers are installed prior to approval of the GLER.

2.3.3 General Fill

General fill material will be uncontaminated earthen fill. General fill includes soils placed for earthen berm or embankment construction, channel swales, roadways, or other earthen features at the landfill. General fill material will be placed in uniform loose lifts which do not exceed 12 inches in loose thickness. General fill will be compacted to at least 95 percent of Standard Proctor maximum dry density (ASTM D 698) at a moisture content range of plus or minus 3 percent of the optimum moisture content.

Proctor and index property (i.e., gradation, Atterberg limits) tests will be performed for each of the general fill borrow sources used for construction. Field density and moisture testing will be limited to embankment construction at a frequency of 1 test per 20,000 square feet of soil placement per 12-inch loose lift. Field testing of non-landfill related fill areas (e.g., roadways, stormwater impoundment features, drainage features) will not be required.

2.3.4 Drainage Aggregate Around Pipes

The coarse aggregate selected for placement around the leachate collection pipes used in the leachate collection system (LCS) for the composite liner and for the temporary hydrostatic pressure relief system discussed in Section 6 will consist of normal (e.g., unit weight of 90 to 110 pcf) or lightweight (e.g., unit weight less than 70 pcf) materials that comply with the following criteria. The LCS aggregate will have a calcium carbonate content less than 15 percent. Either the J&L Testing method or the ASTM D 3042 method, modified to use a solution of hydrochloric acid having a pH of 5, can be used to determine calcium carbonate content. The drainage aggregate will meet the following gradation for ASTM D 448, size number 467.

Sieve Size Square Opening	Percent Passing
2 inches	100
1½ inches	95 - 100
¾ inch	35 - 70
3/8 inch	10 - 30
No. 4 (3/16 inch)	0 - 5

However, if approved by the POR, coarse aggregates not complying with the size number 467 gradation may also be used if demonstrated to have a hydraulic conductivity of at least 1.0 cm/s and meet the filter gradation requirements given below (in no case will the maximum rock size be more than 2 inches) for the specific leachate collection pipe perforation design:

For circular holes in the leachate collection pipe:

$$\frac{\text{85 Percent Size of Filter Material}}{\text{Hole Diameter}} > 1.7$$

For slots in the leachate collection pipe:

$$\frac{\text{85 Percent Size of Filter Material}}{\text{Slot Width}} > 2.0$$

The coarse aggregate will be tested for gradation (ASTM D 448) at the supply source or from the on-site stockpile prior to acceptance. Gradation testing will be conducted at a minimum frequency of 1 test per 3,000 cubic yards of coarse aggregate or per liner construction event if less than 3,000 cubic yards of coarse aggregate is required for the specific construction. The aggregate will be free of organics, angular rocks, foreign objects, or other deleterious materials. The physical characteristics of the aggregate will be evaluated through visual observation and laboratory classification testing before construction and visual observation during construction. The coarse aggregate may be tested during construction at the discretion of the CQA monitor. The test results for the coarse aggregate will be included in the SLER and GLER.

2.3.5 Protective Cover

Protective cover will be placed over the drainage layer in accordance with this section and project plans and specifications. The geosynthetics of the composite liner system will be covered with a minimum of 2 feet of protective cover for the Subtitle D composite liner. The protective cover will consist of soil materials that have not previously come in contact with solid waste or other deleterious materials, and do not contain materials detrimental to the underlying geosynthetics. The protective cover will be free of organic matter, foreign objects, or other deleterious materials. The physical characteristics of the protective cover will be evaluated through visual observation (and laboratory testing if the POR deems it necessary) before construction and visual observation during construction. Additional testing during construction will be at the discretion of the CQA monitor and POR. The protective cover will have passageways (i.e., chimney drains) to allow moisture to drain to the leachate collection system.

The protective cover layer will be placed using any low ground pressure equipment as outlined in Section 3.6. The protective cover will be placed by spreading in front of the spreading equipment with a minimum of 12 inches of soil between the spreading equipment and the installed geosynthetics. Under no circumstances will the construction equipment come in direct contact with the installed geosynthetics.

The thickness of the protective cover layer placed over the composite liner will be verified with surveying procedures at a minimum of 1 survey point per 5,000 square feet of constructed area by a qualified surveyor or professional engineer with a minimum 2 reference points. Thickness may be verified with settlement plates. The survey results and method of surveying for the protective cover will be included in the GLER.

During construction the CQA monitor will:

- Verify that grade control is performed prior to work.
- Verify that underlying geosynthetic installations are not damaged during placement operations or by survey grade controls. Mark damaged geosynthetics and verify that damage is repaired.
- Verify that the cover soil for sideslopes is pushed from the toe up the slope.
- Monitor haul road thickness over geosynthetic installations and verify that equipment hauling and materials placement meet equipment specifications (see Section 3.6).
- The POR will coordinate with the project surveyor to perform a thickness verification survey of the protective cover materials upon completion of placement operations. Verify corrective action measures as determined by the verification survey.

2.3.6 Anchor Trench Backfill

The anchor trench backfill material for geosynthetic anchoring will be uncontaminated earthen material and will be placed and compacted. In-place moisture/density tests may be performed at the discretion of the CQA monitor to evaluate the quality of the backfill. The test results will not be required as part of the GLER or GCLER.

2.3.7 Surface Water Removal

The excavation may encounter water from storm events or groundwater. Soil liner will not be placed in standing water. The excavation area will therefore have a temporary sump area to collect water entering the excavation and be graded to allow drainage at planned areas. Portable pumps will be on site to dewater the sumps. Temporary earthen berms will be constructed to divert surface flow away from the excavation. Surface water that accumulates on the constructed soil liner or geosynthetics surface will be removed promptly after the end of a rainfall event. POR will inspect and approve the constructed area that received rainfall prior to placement of the overlying liner system component. The criteria for approval of the finished surface of the soil liner for geomembrane placement will follow the requirements of Section 3.3.3 and for geocomposite placement on top of geomembrane will follow the requirements of Section 3.5.3. Surface water from the site will be discharged per the site's TPDES permit requirement.

2.3.8 Excavations Below Groundwater

Construction of liners below groundwater is discussed in Section 6 of this appendix.

2.3.9 Liner Tie-In Construction

Newly constructed liners will be tied-in with any adjoining existing liners. Additionally, terminations will be constructed for future tie-ins along edges where the liner will be extended in the future. The tie-ins with existing clay liners will be constructed utilizing a sloped transition a minimum of 10 feet wide for the 2-foot-thick clay liner. Terminations for future tie-ins will be constructed by extending the clay liner approximately 10 feet past the limits for the cell under construction. The liner tie-in details are shown in Appendix IIIA – Landfill Unit Design Information. Waste and intermediate cover will not be deposited closer than 10 feet to the edge of any cell or 20 feet from the leading edge of a constructed clay liner (whichever is greater) where a future tie-in will be constructed. Red-colored markers (i.e., SLER markers) will be placed along the limits of the cells with constructed clay liners and tied to the site grid system in accordance with Title 30 TAC §330.143(b)(1).

2.4 Construction Testing

2.4.1 Standard Operating Procedures

Qualified CQA monitors will perform field and laboratory tests in accordance with applicable standards specified in this LQCP. All quality control testing and evaluation of soil liners will be performed during construction of the liner and must be complete before placement of the leachate collection system, except for the testing required for the final constructed lift, verification of liner thickness, or cover material thickness. Standard operating and test procedures will be utilized per the POR's direction. Sampling from the constructed soil liner lifts will be performed in accordance with ASTM D 1587. The sampling holes (e.g., samples for coefficient of permeability test) will be backfilled with bentonite or bentonite/liner soil material mixture. Prior written approval from the TCEQ via a permit modification will be obtained if any changes will be made to material requirements or procedures set forth on this LQCP.

The following test standards apply as called out in this LQCP and in the technical specifications provided in this LQCP.

Standard Test Method	Test Description
ASTM D 698	Moisture-density relations of soils and soil-aggregate mixtures, using 5½-lb hammer and 12-inch drop
ASTM D 422	Particle size analysis of soils
ASTM D 6938	Standard test method for in-place density and water content of soil and soil aggregate by nuclear methods (shallow depth)
ASTM D 1587	Thin-walled tube sampling of soils for geotechnical purposes
ASTM D 2167	Density and unit weight of a soil in place by the rubber balloon method
ASTM D 6938	In-place density and water content of soil and soil-aggregate by nuclear methods (shallow depth)
ASTM D 2216	Laboratory determination of water (moisture) content of soil, rock, and soil-aggregate mixtures
ASTM D 2434	Method of test for permeability of porous granular material
ASTM D 5084	Method of test for permeability of fine-grained soils
ASTM D 4318	Atterberg limits
ASTM D 1140	Amount of material in soils finer than the No. 200 sieve
ASTM D 2487	Classification of soils for engineering purposes
ASTM D 2488	Description and identification of soils (visual-manual procedure)
EM 1110-2-1906, Appendix VII	U.S. Army Corps of Engineers permeability test
ASTM D 448	Standard classification for sizes of aggregate for road and bridge construction

Standard Test Method	Test Description
ASTM D 3042	Test method for insoluble residue in carbonate aggregates

2.4.2 Test Frequencies

This LQCP establishes the minimum test frequencies for the soil liner construction quality assurance. The test frequencies for soil liner are listed in Table 2-2. Additional testing must be conducted whenever work or materials are suspect, marginal, or of poor quality. Additional testing may also be performed to provide additional data for engineering evaluation. The minimum number of tests is interpreted to mean minimum number of passing tests, and any tests that do not meet the requirements will not contribute to the total number of tests performed to satisfy the minimum test frequency.

Table 2-2
Required Tests and Observations on Soil Liner

Parameter	Frequency	Test Method	Passing Criteria
Field Density and Moisture	1 each per 8,000 SF per 6-inch parallel lift	ASTM D 6938 and ASTM D 2216[1]	95% Maximum Standard Proctor Dry Density. Standard Proctor optimum moisture content or greater determined during preconstruction testing.
Sieve Analysis (passing no. 200 and 1-inch)	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 1140	30 percent minimum (#200) 100 percent minimum (1-inch)
Atterberg Limits	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 4318	PI = 15 percent minimum LL = 30 percent minimum
Coefficient Permeability (Hydraulic Conductivity)[2]	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 5084 (Falling head, flex wall) Corps of Engineers EM 1110-2-1906, Appendix VII (Falling head permeameter)	$1.0\text{x}10^{-7}$ cm/s or less
Thickness Verification	1 each 5,000 square feet with a minimum of 2 reference points by a qualified surveyor	Survey subgrade and top of soil liner and protective cover layer	2 feet minimum compacted soil liner thickness and 2 feet minimum protective cover thickness

[1] This method is not applicable if the field nuclear gauge reads both density and moisture.
[2] Field permeability testing performed in accordance with Title 30 TAC §330.339(c)(7), may be performed to augment this testing program if a permit modification is submitted and approved by the TCEQ.

2.4.3 Soil Liner Testing

CQA testing of the soil liner will be performed as the liner is being constructed. Sections of compacted soil liner which do not pass both the density and moisture requirements will be reworked with additional passes of the compactor until the section in question passes. All field density and moisture test results will be incorporated into the SLER.

Soil liner field density and moisture testing will be completed on each 6-inch compacted lift at a frequency of one test per 8,000 square feet of soil liner installed. Passing tests will be achieved with a minimum of 95 percent compaction of the Standard Proctor maximum dry density at a moisture content at or above optimum moisture content. Areas that do not receive satisfactory field density and moisture testing will be moisture conditioned and recompacted to achieve satisfactory results.

Hydraulic conductivity samples will be obtained by pushing a sampler through each lift of the constructed clay liner prior to construction of the next lift. The sample from each test location will be sealed and transported to the laboratory. Two samples may be collected at each sample location and labeled the “A” and “B” sample. The sampling holes (e.g., samples for hydraulic conductivity) will be backfilled with bentonite or a bentonite/clay liner soil material mixture consisting of at least 20 percent bentonite and compacted by hand tamping.

If the integrity of the “A” sample appears to have been compromised during the transportation of the sample prior to testing, the “B” sample may be tested. In addition, if an “A” sample hydraulic conductivity test does not comply with the minimum allowable value, the “B” sample collected at the same location may be tested to determine compliance with the hydraulic conductivity requirements if during testing of the “A” sample the ASTM D 5084 or EM 1110-2-1906 procedure was not followed or the permeameter malfunctioned. The POR will provide a detailed justification of the use of the “B” sample, if applicable, in the SLER.

If the “B” sample passes, the area will be considered in compliance. If the “B” sample fails (or Sample “A” fails in such a way that there is not an option to use the “B” sample), the test interval will be considered unsatisfactory for the area bounded by passing test locations (but not extending past a satisfactory test location). Additional tests may be taken to further define the unsatisfactory area. The area defined unsatisfactory will be reworked and retested in accordance with this section.

Furthermore, if it is determined that the “B” sample may not be used to replace the “A” sample result, then the test interval will be considered unsatisfactory for the area bounded by passing test locations (but not extending past a satisfactory test location).

Once the exact area is determined, the constructed liner lifts will be removed to the bottom of the lift that did not pass the hydraulic conductivity test and reconstructed until all the samples obtained from the failed area meet the hydraulic conductivity requirements. At a minimum, one hydraulic conductivity test will be performed for each lift, given that the reconstructed liner area is not larger than 100,000 square feet (i.e., 4 hydraulic conductivity tests per 100,000 square feet of reconstructed liner area). The reconstructed liner area will be tied into the currently constructed liner with a 5H:1V transition slope according to the tie-in detail included in Appendix IIIA – Landfill Unit Design Information. Reconstructed liner area is also subject to field density and moisture content testing per Table 2-2 (at least one field density and one moisture content test is required for each lift regardless of the size of the area that is reconstructed).

Each lift of the reconstructed liner area will be tested for hydraulic conductivity. Reconstruction activities, including additional testing and surveying, will be incorporated into the SLER.

2.4.4 Material Strength Requirements

The geotechnical analysis is included in Appendix IIIE – Geotechnical Report and includes slope stability, foundation heave, and settlement analyses. Soil parameters used in the geotechnical analysis were obtained from subsurface investigations and geotechnical reports, as well as from geotechnical testing performed on soil samples recovered at the site. The POR will verify that the proposed liner material meets the minimum soil properties used in the geotechnical analysis included in Appendix IIIE prior to liner construction, as applicable. These soil properties include unit weight, moisture content, cohesion, friction angle, and consolidation strength parameters used in the slope stability and settlement analyses. The POR will verify that the underlying material below the composite liner is consistent with design assumptions. If the POR determines that the underlying material or borrow material is not consistent with design assumptions, the appropriate geotechnical analysis (e.g., slope stability) will be updated consistent with the procedures in Appendix IIIE. The updated analysis will be incorporated into the SLER/GLER.

2.5 Reporting

The POR will submit to the TCEQ a SLER for approval of each Subtitle D soil liner area. Section 8 describes the documentation requirements.

3 CONSTRUCTION QUALITY ASSURANCE FOR GEOSYNTHETICS

3.1 Introduction

Section 3 describes CQA procedures for the installation of geosynthetic components, except GCL for which procedures are provided in Section 4.

The scope of geosynthetic related construction quality assurance includes the following elements:

- Bottom Liner Geomembrane
  - Floor Grades: 60-mil HDPE – smooth or textured on both sides
  - Sideslopes: 60-mil HDPE – textured on both sides
- Geotextile
- Drainage Layer
  - Single-sided drainage geocomposite (on bottom liner floor grades)
  - Double-sided drainage geocomposite (underdrain and bottom liner side slopes)

The overall goal of the geosynthetics quality assurance program is to assure that proper construction techniques and procedures are used, the geosynthetic contractor implements his quality control plan in accordance with this LQCP, and that the project is built in accordance with the project construction drawings and technical specifications that will be developed in accordance with this LQCP for each liner construction. The quality assurance program is intended to identify and define problems that may occur during construction and to observe that these problems are avoided and/or corrected before construction is complete. A GLER, prepared after project completion, will document that the constructed facility meets design intent and specifications outlined in this LQCP.

3.2 Geosynthetics Quality Assurance

3.2.1 General

The composite liner system provides the primary means for preventing leachate infiltration into groundwater. A geomembrane is a component of the bottom liner. Proper geomembrane installation is a crucial work element, which greatly affects the performance of the liner systems. Construction quality control for the geomembrane installation will be performed by the geomembrane installation contractor. Construction quality assurance for the geomembrane installation will be performed by the POR to assure the geomembrane is constructed as specified in the design. Construction must be conducted in accordance with the procedures outlined in this LQCP. To monitor compliance, a quality assurance program will include the following:

- A review of the manufacturer's quality control testing
- Material conformance testing by an independent third-party laboratory
- Field and construction testing
- Construction monitoring

The manufacturer's quality control testing will include resin and geomembrane testing. The required tests for material properties are included in Section 3.3.

Conformance testing refers to material testing performed by an independent third-party laboratory that takes place prior to material installation. Field and construction testing includes testing that occurs during geosynthetics installation.

Quality assurance testing will be conducted in accordance with this LQCP. Field testing will be observed by the CQA monitor. Documentation must meet the requirements of this LQCP.

3.3 Bottom Liner Geomembrane

The bottom liner geomembrane will consist of a 60-mil HDPE geomembrane. The geomembrane will be smooth or textured on both sides on the floor and textured on both sides on the sideslopes. Required manufacturer's quality control tests for the bottom liner geomembrane are included in Table 3-1 and required material properties for the bottom liner geomembrane are included in Table 3-2.

3.3.1 Delivery

Upon delivery of FML, the CQA monitor will observe that:

- The geomembrane is delivered in rolls and is not folded. Folded geomembrane is not acceptable because the highly crystalline structure of the geomembrane will be damaged if it is folded. Any evidence of folding (other than from the manufacturing process) or other shipping damage is cause for rejection of the material.
- Equipment used to unload and store the rolls does not damage the geomembrane.
- The geomembrane is stored in an acceptable location in accordance with the manufacturer's specifications and stacked not more than 5 rolls high. The geomembrane is protected from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, excessive heat, or other damage.
- All manufacturing documentation required by the specifications outlined in this LQCP has been received and reviewed for compliance. This documentation will be included in the GLER.
- A geosynthetics receipt log form has been completed for all materials received.

Damaged geomembrane will be rejected and removed from the site or stored at a location separate from accepted geomembrane. Geomembrane that does not have proper manufacturer's documentation must be stored at a separate location until all documentation has been received, reviewed, and accepted.

3.3.2 Conformance Testing

Tests. One geomembrane sample will be obtained for every resin lot of material supplied and for each 100,000 square feet of geomembrane installed. The material will be sampled at the manufacturing plant by the third-party testing laboratory or the site by the CQA monitor. The samples will be forwarded to the independent third-party laboratory for the following conformance tests:

- Specific gravity/Density (ASTM D 1505 or alternate ASTM D 792, Method A if approved by the POR)
- Carbon black content (ASTM D 4218)
- Carbon black dispersion (ASTM D 5596)
- Thickness (ASTM D 5199 for smooth FML and for textured FML use ASTM D 5994
- Tensile properties (ASTM D 638/Type IV, ASTM D 6693 may be used upon approval by POR)

Table 3-1
Required Testing for 60-mil-thick Smooth and Textured (Both Sides) HDPE Geomembranes[1]

Test	Type of Test	Standard Test Method	Frequency of Testing (Minimum)
Resin	Specific Gravity/Density	ASTM D 792, Method A or ASTM D 1505	Per 200,000 SF and every resin lot
	Melt Flow Index	ASTM D 1238	Per 100,000 SF and every resin lot
Manufacturer's Quality Control	Thickness	ASTM D 5199 (smooth) or ASTM D 5994[2] (textured)	Per Roll of Geomembrane
	Specific Gravity/Density	ASTM D 1505/D 792	Per 200,000 pounds
	Carbon Black Content	ASTM D 4218	Per 20,000 pounds
	Carbon Black Dispersion	ASTM D 5596	Per 45,000 pounds
	Tensile Properties	ASTM D 638 / Type IV (ASTM D 6693 may be used as an alternative upon POR's approval)	Per 20,000 pounds
	Tear	ASTM D 1004	Per 45,000 pounds
	Puncture	ASTM D 4833	Per 45,000 pounds
	Stress Crack Resistance	ASTM D 5397	Per GRI-GM 10
	Oxidative Induction Time	ASTM D 3895 or ASTM D 5885	Per 200,000 pounds
	Oven Aging @ 85°C Standard OIT (min. avg.) or High pressure OIT - % retained after 90 days for both	ASTM D 5721 ASTM D 3895 ASTM D 5885	Per each formulation
	UV Resistance[3] High Pressure OIT (min. avg.) - % retained after 1,600 hours	ASTM D 7238 ASTM D 5885	Per each formulation
	Asperity Height	ASTM D 7466	Every 2nd roll[4]

[1] All tests will conform to the minimum requirements set forth by GRI testing standard GM13. Required values for the parameters are listed in Table 3-2.
[2] ASTM D 1593 may also be used for thickness of textured geomembrane.
[3] 20 hours of UV cycle at 75°C followed by 4 hours condensation at 60°C.
[4] Measurement side will be alternated for double-sided textured sheet. This testing is specified for textured geomembrane only.

Table 3-2
Minimum Required Properties of 60-mil-thick Smooth and Textured (Both Sides) HDPE Geomembranes

Property	Test Method	Minimum Required Property[8]	
		Smooth	Textured
Thickness, mils Minimum average Lowest individual reading Lowest individual of 8 of 10 readings	ASTM D 5199 (smooth) ASTM D 5994 (textured)	60 54 NA	57 51 54
Density, g/cc	ASTM D 1505/D 792	0.94	0.94
Asperity Height, mils	GRI GM12	N/A	16
Tensile Properties[1] 1. Yield Strength, lb/in 2. Break Strength, lb/in 3. Yield Elongation, % 4. Break Elongation, %	ASTM D 638 (Type IV Specimen @ 2 in/min) (ASTM D 6693 may be used as an alternative upon approval by POR)	126 228 12 700	126 90 12 100
Tear Resistance, lb	ASTM D 1004	42	42
Puncture Resistance, lb	ASTM D 4833	108	90
Stress Crack Resistance[2], hrs	ASTM D 5397	500	500
Carbon Black Content[3], %	ASTM D 1603	2.0 – 3.0	2.0 – 3.0
Carbon Black Dispersion[4], Category	ASTM D 5596	see note 4	see note 4
Oxidative Induction Time (OIT)[5] (Minimum Average) Standard OIT, minutes High Pressure OIT, minutes	ASTM D 3895 ASTM D 5885	100 400	100 400
Oven Aging at 85ºC Standard OIT – % retained after 90 days High Pressure OIT – % retained after 90 days	ASTM D 5721 ASTM D 3895 ASTM D 5885	55 80	55 80
UV Resistance[6] High Pressure OIT[7] – % retained after 1600 hrs	ASTM D 7238 ASTM D 5885	50	50
Seam Properties (5 out of 5 specimens, per GRI-GM19) 1. Shear Strength, lb/in 2. Peel Strength, lb/in	ASTM D 6392	120 91 & FTB (78, Extrusion Weld)	120 91 & FTB (78, Extrusion Weld)

[1] Machine direction (MD) and cross machine direction (XMD) average values will be on the basis of 5 test specimens each direction. Yield elongation is calculated using a gauge length of 1.3 inches; break elongation is calculated using a gauge length of 2.0 inches.
[2] The yield stress used to calculate the applied load for the Single Point Notched Constant Tensile Load (SP-NCTL) test will be the mean value via MQC testing.
[3] Other methods such as ASTM D 4218 or microwave methods are acceptable if an appropriate correlation can be established.
[4] Carbon black dispersion for 10 different views: 9 in Categories 1 and 2 and 1 (max) in Category 3.
[5] The manufacturer has the option to select either one of the OIT methods listed to evaluate the antioxidant content in the geomembrane.
[6] The condition of the test will be 20 hr UV cycle at 75ºC followed by 4 hr. condensation at 60ºC.
[7] UV resistance is based on percent retained value regardless of the original HP-OIT value.
[8] Minimum required properties are based on GRI-GM13, except for the seam properties which are based on GRI-GM19. At the time of each liner construction event, an updated GRI-GM13 and GRI-GM19 will be used if available.

The density of the geomembrane must be greater than 0.94 g/cc; the carbon black content must be between 2 percent and 3 percent; and recycled or reclaimed material must not be used in the manufacturing process.

The design engineer may require additional test procedures and will inform the third-party laboratory in writing. The POR must review all test results and report any nonconformance to the design engineer prior to product installation. In addition to the conformance thickness tests shown above, field thickness measurements must be taken at maximum 5-foot intervals along the leading edge of each geomembrane panel. No single measurement will be less than 10 percent (15 percent for textured) below the required nominal thickness for the panel to be accepted, and the average must be at least 60 mils (57 mils for textured). Refer to Table 3-2 for a complete listing of the material requirements for both smooth and textured geomembranes that will be used for the composite Subtitle D bottom liner.

Sampling Procedure. Samples will be taken across the entire roll width. Unless otherwise specified, samples will be approximately 15 inches long by the roll width. The third-party testing laboratory or CQA monitor must mark the machine direction and the manufacturer's roll identification number on the sample. The third-party testing laboratory or CQA monitor must also assign a conformance test number to the sample and mark the sample with that number.

3.3.3 Geomembrane Installation

Surface Preparation. Prior to any geomembrane installation, the installed soil liner surface will be inspected by the CQA and geosynthetics contractor. The POR or CQA monitor must observe the following:

- All lines and grades for the soil liner or GCL have been verified by the surveyor and accepted by the contractor for geosynthetic installation. The POR or his representative, the owner, and geomembrane installer will certify and accept in writing the finished final lift of the soil liner or GCL surface.
- The soil liner has been prepared in accordance with the earthwork construction plans and specifications as outlined in Section 2.
- The GCL has been prepared in accordance with the construction plans and specifications as outlined in Section 4.
- The soil liner is free of surface irregularities and protrusions. The soil liner will be rolled and compacted to ensure a clean surface.
- The soil liner or GCL surface does not contain stones or other objects that could damage the geomembrane or underlying soil liner or GCL. The surface of the soil liner or GCL will be smooth and free of foreign and organic material, sharp objects, exposed soil or aggregate particles greater than 3/4 inch (or less if recommended by the geosynthetic manufacturer), or other deleterious material.

- The anchor trench dimensions have been checked, and the trenches are free of sharp objects and stones.
- There are no excessively soft areas in the soil liner that could result in geomembrane damage.
- The geomembrane will not be placed over soil liner or GCL during inclement weather such as rain or high winds.
- The soil liner is not saturated, and no standing water is present above the soil liner or GCL.
- The soil liner has not desiccated (e.g., areas with desiccation cracks).
- All construction stakes and hubs have been removed and the resultant holes have been backfilled. There are no rocks, debris, or any other objects on the soil liner surface.
- The geosynthetics contractor has certified in writing that the soil liner or GCL surface on which the geomembrane will be installed is acceptable.

Panel Placement. Prior to the installation of the geomembrane, the contractor must submit drawings showing the panel layout, indicating panel identification number, both fabricated (if applicable) and field seams, as well as details not conforming to the drawings.

The CQA monitor must maintain an up-to-date panel layout drawing showing panel numbers that are keyed to roll numbers on the placement log. The panel layout drawing will also include seam numbers and repair and destructive test locations.

During panel placement, the POR or CQA monitor must:

- Observe that geomembrane is placed in direct and uniform contact with the underlying soil liner or GCL.
- Record roll numbers, panel numbers, and dimensions on the panel or seam logs. Measure and record thickness of leading edge of each panel at 5-foot maximum intervals. No single thickness measurement can be less than 10 percent (15 percent for textured) below the required nominal thickness.
- Observe the sheet surface as it is deployed and record all panel defects and repair of the defects (panel rejected, patch installed, extrudate placed over the defect, etc.) on the repair sheet. All repairs must be made in accordance with the specifications as outlined in Section 3.3.5 and located on a repair drawing.
- Observe that support equipment is not allowed on the geomembrane during handling (see Section 3.6 also).
- Observe that the surface beneath the geomembrane has not deteriorated since previous acceptance.

- Observe that there are no stones, construction debris, or other items beneath the geomembrane that could cause damage to the geomembrane.
- Observe that the geomembrane is not dragged across a surface that could damage the material. If the geomembrane is dragged across an unprotected surface, the geomembrane must be inspected for scratches and repaired or rejected, as necessary.
- Record weather conditions including temperature, wind, and humidity. The geomembrane must not be deployed in the presence of excess moisture (fog, dew, mist, or wind, etc.). In addition, geomembrane will not be placed when the air temperature is less than 41°F or greater than 104°F, or when standing water or frost is on the ground, unless this requirement is waived by the design engineer or TCEQ. Excessive wind is that which can lift and move the geomembrane panels.
- Observe that people working on the geomembrane do not smoke, wear shoes that could damage the liner, or engage in activities that could damage the liner.
- Observe that the method used to deploy the sheet minimizes wrinkles but does not cause bridging and that the sheets are anchored to prevent movement by the wind (the contractor is responsible for any damage to or from windblown geomembrane). Excessive wrinkles will be walked-out or removed at the discretion of the CQA monitor.
- Observe that no more panels are deployed than can be seamed on the same day.
- Observe that there are no horizontal seams on side slopes, and the textured material extends a minimum of approximately 5 feet out past the toe of the slope where textured geomembrane is used. This requirement may be waived if textured material is utilized on the floor.

The CQA monitor must inform both the contractor and the POR of the above conditions.

Field Seaming. The contractor must provide the POR with a seam and panel layout drawing and update this drawing daily as the job proceeds. No panels will be seamed until the panel layout drawing has been accepted by the POR. A seam numbering system must provide a unique number for each seam and be agreed to by the POR and contractor prior to the start of seaming operations. One procedure is to identify the seam by adjacent panels. For example, the seam located between Panels 306 and 401 would be Seam No. 306/401.

Prior to geomembrane welding, each welder and welding apparatus (both wedge and extrusion welders), must be tested, at a minimum, at daily start-up and at midday break, or any break that the seaming machine is stopped more than 30 minutes to determine if the equipment is functioning properly. The GLER will include the names for each seamer and the time and the temperatures for each seaming apparatus used each day. The trial weld sample must be 3 feet long and 12 inches wide, with the seam centered lengthwise. The minimum number of specimens per trial weld test must be two coupons for shear and two coupons for peel. Both the inner and outer welds of dual track fusion welds must be tested for each peel test coupon (or additional coupons will be required). Trial weld samples must comply with "Passing Criteria for Welds" included in Section 3.3.4 – Construction Testing. The CQA monitor must observe all welding operations, quantitative testing of each trial weld for peel and shear and recording of the results on the trial weld form. The trial weld will be completed under conditions similar to those under which the panels will be welded. Regarding the locus-of-break patterns of the different seaming methods in shear and peel, the following are unacceptable break codes per their description in ASTM D 6392 and GRI-GM19:

Hot Wedge: AD and AD-Brk>25%

Extrusion Fillet: AD1, AD2, AD-WLD (unless strength is achieved)

Additionally, there will be no apparent weld separation. The strength tests must meet the manufacturer's specifications for the sample sheets, or the percentage of the manufacturer's parent sheet strength as determined by the manufacturer. For dual-track fusion welds, both sides (the inner and outer weld) must meet the minimum requirements for a satisfactory peel test. Reference to 25% peel or separation during testing means 25% of the width of a single weld (i.e., full width of an extrusion weld, or a single track of a dual track fusion weld). If, at any time, the CQA monitor believes that an owner or welding apparatus is not functioning properly, a weld test must be performed. If there are wide changes in temperature (±30° Fahrenheit), humidity, or wind speed, the test weld will be repeated. The test weld must be allowed to cool to ambient temperature before testing. If a weld test fails the shear or peel test, the length of the non-passing weld will be identified at a 10-foot interval and the failed area will be patched. Patching will performed by placing additional geomembrane over the failed area or removing the failed area geomembrane weld and patching it with additional geomembrane per POR's direction. Welding for patches must comply with the welding passing criteria requirements outlined in this section.

Construction quality assurance documentation of trial seam procedures will include, at a minimum, the following:

- Documentation that trial seams are performed by each welder and welding apparatus prior to commencement of welding and prior to commencement of the second half of the workday.
- The welder, the welding apparatus number, time, date, ambient air temperature, and welding machine temperatures.

During geomembrane welding operations, the CQA monitor must observe the following:

- The contractor has the number of welding apparatuses and spare parts necessary to perform the work.
- Equipment used for welding will not damage the geomembrane.
- The extrusion welder is purged prior to beginning a weld until all the heat-degraded extrudate is removed (extrusion welding only).
- Seam grinding has been completed less than one hour before seam welding, and the upper sheet is beveled (extrusion welding only).
- The ambient temperature, measured 6 inches above the geomembrane surface, is between 41° and 104° Fahrenheit unless more stringent limits are required by the manufacturer.
- The end of old welds, more than five minutes old, are ground to expose new material before restarting a weld (extrusion welding only).
- The contact surfaces of the sheets are clean, free of dust, grease, dirt, debris, and moisture prior to welding.
- The weld is free of dust, rocks, and other debris.
- The seams are overlapped a minimum of 3 inches for extrusion and hot-wedge welding, or in accordance with manufacturer's recommendations, whichever is more stringent. Panels will be overlapped (shingled) in the downgrade direction.
- No solvents or adhesives are present in the seam area.
- The procedure used to temporarily hold the panels together does not damage the panels and does not preclude CQA testing.
- The panels are being welded in accordance with the plans and specifications that will be developed in accordance with this section for each liner construction. Seams will be oriented parallel to the line of maximum slope with no horizontal seams on side slopes. In corners and odd-shaped geometric locations, the number of field seams will be minimized.
- There is no free moisture in the weld area.
- Measure surface sheet temperature every two hours.
- Observe that at the end of each day or installation segment, all unseamed edges are anchored with sandbags or other approved device. Penetration anchors will not be used to secure the geomembrane.

3.3.4 Construction Testing

Nondestructive Seam Testing. The purpose of nondestructive testing is to detect discontinuities or holes in the seam. It also indicates whether a seam is continuous and non-leaking. Nondestructive tests for geomembrane include vacuum testing for extrusion welds and air pressure testing for dual track fusion welds. Nondestructive testing must be performed over the entire length of the seam.

Nondestructive testing is performed entirely by the contractor. The CQA monitor's responsibility is to document the date, time and location of seaming and testing, and to observe and document that testing was performed in compliance with this section and document any seam defects and their repairs.

Nondestructive testing procedures are described below.

- For welds tested by vacuum method, the weld is placed under suction utilizing a vacuum box made of rigid housing with a transparent viewing window, a soft neoprene rubber gasket attached to the open bottom perimeter, a vacuum gauge on the inside, and a valve assembly attached to the vacuum hose connection. The box is placed over a seam section, which has been thoroughly saturated with a soapy water solution (1 oz. soap to 1 gallon water). The rubber gasket on the bottom perimeter of the box must fit snugly against the soaped seam section of the liner, to ensure a leak-tight seal. The vacuum pump is energized, and the vacuum box pressure is reduced to approximately 3 to 5 psi gauge. Any pinholes, porosity or non-bonded areas are detected by the appearance of soap bubbles in the vicinity of the defect. Dwell time must not be less than ten seconds.
- Air pressure testing is used to test double seams with an enclosed air space (i.e., dual-track fusion welds). Both ends of the air channel will be sealed. The pressure feed device, usually a needle equipped with a pressure gauge, is inserted into the channel. Air is then pumped into the channel to a minimum pressure of 30 psi or ½ psi per mil of geomembrane thickness, whichever is greater. The air chamber must sustain the pressure for five minutes without losing more than 4 psi. Following a passed pressure test, the opposite end of the tested seam must be punctured to release the air. The pressure gauge must return to zero; if not, a blockage is most likely present in the seam channel. Locate the blockage and test the seam on both sides of the blockage. The penetration holes must be sealed after testing.

During nondestructive testing, the CQA monitor must perform the following work:

- Review technical specifications regarding test procedures.
- Observe that equipment operators are fully trained and qualified to perform their work.
- Observe that test equipment meets project specifications that will be developed in accordance with this LQCP for each liner construction.
- Observe that the entire length of each seam is tested in accordance with the specifications outlined in this section.

- Observe all continuity testing and record results on the appropriate log.
- Observe that all testing is completed in accordance with the specifications outlined in this section.
- Identify the failed areas by marking the area with a waterproof marker compatible with the geomembrane and inform the contractor of any required repairs, then record the repair area on the repair log.
- Observe that all repairs are completed and tested in accordance with the project specifications outlined in this section and Section 3.3.5.
- Record all completed and tested repairs on the repair log and the repair drawing.

Destructive Seam Testing. Destructive seam tests for geomembrane seams will be performed at intervals of at least one test per 500 linear feet of seam length. At a minimum, a destructive test will be completed for each welding machine used for seaming. A destructive test will also be performed for individual repairs (or additional seaming for the failed seams) at intervals of at least one test per 500 linear feet. Only individual repairs (or additional seaming for failed seams) requiring more than 10 feet of seaming shall count toward the testing interval. The CQA monitor must perform additional tests if he suspects a seam does not meet specification requirements outlined in this section. Reasons for performing additional tests may include, but are not limited to the following:

- Wrinkling in seam area
- Non-uniform weld
- Excess crystallinity
- Suspect seaming equipment or techniques
- Weld contamination
- Insufficient overlap
- Adverse weather conditions
- Possibility of moisture, dust, dirt, debris, and other foreign material in the seam
- Failing tests

There are two types of destructive testing required for the geomembrane installation: peel adhesion (peel) and bonded seam strength (shear) in accordance with ASTM D 6392. The purpose of peel and shear tests is to evaluate seam strength and to evaluate long-term performance. Shear strength measures the continuity of tensile strength through the seam and into the parent material. Peel strength determines weld quality. Test welds must be allowed to cool naturally to ambient temperature prior to testing.

The CQA monitor selects locations where seam samples will be cut for laboratory testing. Select these locations as follows:

- A minimum of one random test within each 500 feet of seam length. This is an average frequency for the entire installation; individual samples may be taken at greater or lesser intervals.
- Sample locations will not be disclosed to the contractor prior to completion of the seam.
- A maximum frequency must be agreed to by the contractor, POR, and the Operator at the preconstruction meeting. However, if the number of failed samples exceeds 5 percent of the tested samples, this frequency may be increased at the discretion of the POR. Samples taken as the result of failed tests do not count toward the total number of required tests.

Sampling Procedures. The contractor will remove samples at locations identified by the CQA monitor. The CQA monitor must:

- Observe sample cutting.
- Mark each sample with an identifying number, which contains the seam number and destructive test number.
- Record sample location on the panel layout drawing and destructive seam log.
- Record the sample location, weather conditions, and reason sample was taken (e.g., random sample, visual appearance, result of a previous failure, etc.).

For each destructive test obtain one sample approximately 45 inches long by 12 inches wide, with the weld centered along the length. Cut two 1-inch-wide coupons from each end of the sample. The contractor must test two of these coupons in shear and two in peel (one shear and one peel from each end) using a tensiometer capable of quantitatively measuring the seam strengths. For double wedge welding, both sides of the air channel will be tested in peel. The CQA monitor must observe the tests and record the results on the destructive seam test log. A geomembrane seam sample passes the field testing when the break is Film Tear Bond (FTB) and the seam strength meets the required strength values for peel and shear given previously for trial seams under field seaming and below for third-party laboratory testing. As previously discussed, both welds have to pass for dual-track welds. Also, it is recommended that additional samples be obtained as discussed in the following paragraph if there is apparent separation of the weld during peel testing.

If one or both of the 1-inch specimens fail in either peel or shear, the contractor can, at his discretion: (1) reconstruct the entire seam between passed test locations, or (2) take two additional test samples 10 feet or more in either direction from the point of the failed test and repeat this procedure. For tracking purposes, the additional samples will be identified by assigning an identifying letter to the initial destructive test sample number (e.g., DS-6A and B). Only satisfactory tests count toward the required minimum number, and additional tests (i.e., A and B) count as one test, if passing. If the second set of tests passes, the contractor can reconstruct or cap-strip the seam between the two passed test locations. If subsequent tests fail, the sampling and testing procedure is repeated until the length of the poor quality seam is established. Repeated failures indicate that either the seaming equipment or operator is not performing properly, and appropriate corrective action must be taken immediately.

If the field test coupons are satisfactory, divide the remaining sample into three parts: one 12-inch by 12-inch section for the contractor, one 12-inch by 16-inch section for the third-party laboratory for testing, and one 12-inch by 12-inch section for the operator to archive. The laboratory sample will be shipped to the third-party laboratory for overnight delivery and next day testing.

If the laboratory test fails in either peel or shear, the contractor must either reconstruct the entire seam between passing test locations or recover additional samples at least 10 feet on either side of the failed sample for retesting. Sample size and disposition must be as described in the preceding paragraph. This process is repeated until passed tests bracket the failed seam section. All seams must be bounded by locations from which passing laboratory tests have been taken. Laboratory testing governs seam acceptance. In no case can field testing of repaired seams be used for final acceptance.

Third-party Laboratory Testing. Destructive samples must be shipped to the third-party laboratory for seam testing. Testing for each sample will include 5 bonded seam shear strength tests and 5 peel adhesion tests (10 for dual-track welds). For dual-track welds each peel test specimen (coupon) will be tested on both sides of the air channel (i.e., the inner and outer welds). All five specimens tested in peel and shear shall meet the minimum strength requirements. The minimum peel strength and the minimum shear strength values must meet the passing criteria listed below. Additionally, all 5 of the peel test coupons must have no greater than 25 percent seam separation. For dual-track welds if either weld exhibits greater than 25 percent separation or does not meet the required strength, that coupon is considered out of compliance and causes the weld to fail. The third-party laboratory must provide test results within 24 hours, in writing or via telephone, to the CQA monitor. Certified test results are to be provided within 5 days. The CQA monitor must immediately notify the POR in the event of a calibration discrepancy or failed test results.

Passing Criteria for Welds. Passing criteria are established by GRI GM19 for geomembranes. A passing extrusion or fusion welded seam will be achieved when the following values are tested. The following values listed for shear and peel strengths are for all 5 test specimens. Elongation measurements will be omitted for field testing.

- Shear strength (lb/in) 120 (90 for Textured)
- Shear elongation at break (%) 50
- Peel strength (lb/in) 91 (78 Extrusion Weld) & FTB
- Peel separation (%) 25

A passing extrusion or fusion welded seam will be achieved in peel when:

- Yield strength for all 5 specimens (10 tests for dual-track welds) is not less than the above minimum peel strength value and the average of all 5 specimens is not less than the minimum value.
- No greater than 25 percent of the seam width peels (separates) at any point for all 5 specimens (both inner and outer welds for dual-track welds).

A passing extrusion or fusion weld will be achieved in shear when:

- Yield strength for all 5 specimens is not less than the above minimum shear strength value and the average for all 5 specimens is not less than the minimum value.
- Break strain for all 5 specimens is at least 50 percent.

3.3.5 Repairs

Any portion of the geomembrane with a detected flaw, or which fails a nondestructive or destructive test, or where destructive tests were cut, or where nondestructive tests left cuts or holes, must be repaired in accordance with the specific liner construction specifications and consistent with all the applicable parts (e.g., material requirement, installation, testing, etc.) of this section. The CQA monitor must locate and record all repairs on the repair sheet and panel layout drawing. Repair techniques include the following:

- Patching – used to repair large holes, tears, large panel defects, undispersed raw materials, contamination by foreign matter, and destructive sample locations.
- Extrusion – used to repair small defects in the panels and seams. In general, this procedure will be used for defects less than ⅜-inch in the largest dimension.
- Capping – used to repair failed welds or to cover seams where welds or bonded sections cannot be nondestructively tested.
- Removal – used to replace areas with large defects where the preceding methods are not appropriate. Also used to remove excess material (wrinkles, fishmouths, intersections, etc.) from the installed geomembrane. Areas of removal will be patched or capped.

Repair procedures include the following:

- Abrade geomembrane surfaces to be repaired (extrusion welds only) no more than one hour prior to the repair.
- Clean and dry all surfaces at the time of repair.
- Extend patches or caps at least 6 inches beyond the edge of the defect, and round all corners of material to be patched and the patches to a radius of at least 3 inches. Bevel the top edges of patches prior to extrusion welding.
- Testing of repaired seams consistent with Section 3.3.4 – Construction Testing.

3.3.6 Wrinkles

During placement of cover materials over the geomembrane, temperature changes or creep can cause wrinkles to develop in the geomembrane. Any wrinkles which can fold over must be repaired either by cutting out excess material or, if possible, by allowing the liner to contract by temperature reduction. In no case can material be placed over the geomembrane, which could result in the geomembrane folding. The CQA monitor must monitor geomembrane for wrinkles and notify the contractor if wrinkles are being covered by soil. The CQA monitor is then responsible for documenting corrective action to remove the wrinkles.

3.3.7 Folded Material

All folded geomembrane must be removed. Remnant folds evident after deployment of the roll, which are due to manufacturing process, are acceptable.

3.3.8 Geomembrane Anchor Trench

The geomembrane anchor trench will be left open until seaming is completed. Expansion and contraction of the geomembrane will be accounted for in the liner placement. Prior to backfilling, the depth of penetration of the geomembrane into the anchor trench must be verified by the CQA monitor at a minimum of 100-foot spacings along the anchor trench. The anchor trench will be filled in the morning when temperatures are coolest to reduce bridging of the geomembrane. The material used will meet the criteria outlined in Section 2.3.7.

3.3.9 Geomembrane Acceptance

The contractor retains all ownership and responsibility for the geomembrane until acceptance by the Operator. In the event the contractor is responsible for placing cover over the geomembrane, the contractor retains all ownership and responsibility for the geomembrane until all required documentation is complete, and the cover material is placed. After panels are placed, seamed, tested successfully, and any repairs are made, the completed installation will be walked by the Operator's and contractor's representatives. Any damage or defect found during this inspection will be repaired properly by the installer. The installation will not be accepted until it meets the requirements of both representatives. In addition, the geomembrane will be accepted by the POR only when the following has been completed:

- The installation is finished.
- All seams have been inspected and verified to be acceptable.
- All required laboratory and field tests have been completed and reviewed.
- All required contractor-supplied documentation has been received and reviewed.
- All as-built record drawings have been completed and verified by the POR. The as-built drawings show the true panel dimensions, the location of all seams, trenches, pipes, appurtenances, and repairs.
- Acceptance of the GLER by TCEQ.

3.3.10 Bridging

Bridging must be removed.

3.4 Geotextiles

Geotextiles will be used to prevent clogging of drainage materials. The main usage of geotextiles will be enveloping drainage stone used for chimney drains in the leachate collection system (LCS) and underdrain. Geotextiles for the LCS will meet the design requirements set forth in Appendix IIIC – Leachate and Contaminated Water Management Plan. Manufacturer's testing for geotextile is listed in Table 3-6.

3.4.1 Delivery

During delivery the CQA monitor must observe the following:

- Equipment used to unload the rolls will not damage the geotextile.
- Rolls are wrapped in impermeable and opaque protection covers.
- Care is used when unloading the rolls.
- All documentation required by this LQCP and the specifications has been received and reviewed for compliance with this LQCP.
- Each roll is marked or tagged with the manufacturer's name, project identification, lot number, roll number, and roll dimensions.

- Materials are stored in a location that will protect the rolls from precipitation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions.

Any damaged rolls must be rejected and removed from the site or stored at a location separate from accepted rolls, designated by the Operator. All rolls which do not have proper manufacturer's documentation must also be stored at a separate location until all documentation has been received and approved.

3.4.2 Testing

The geotextile manufacturer will conduct manufacturer quality control (MQC) testing and certify that the materials delivered to the site comply with project specifications outlined in this LQCP. The material certification will be reviewed by the POR and approved for the project prior to acceptance of any of the material. The MQC testing will include the following tests with at least one test for each 100,000 square feet of geotextile delivered.

- Grab tensile strength/elongation (ASTM D 4632)
- Mass per unit area (ASTM D 5261)
- Thickness (ASTM D 5199)
- Puncture resistance (ASTM D 4833)
- Trapezoidal tear strength (ASTM D 4533)
- Hydraulic tests (ASTM D 4491)
- Apparent opening size (ASTM D 4751)

Where optional procedures are noted in the test method, the specification requirements of this LQCP prevail. The POR will review all test results and report any nonconformance.

3.4.3 Geotextile Installation

Surface Preparation. Prior to geotextile installation, the CQA monitor must observe the following:

- All lines and grades have been verified by the surveyor.
- The supporting surface does not contain stones that could damage the geotextile or the underlying geomembrane.
- There are no excessively soft areas that could result in damage to the geotextile, or other components of the liner system.
- Construction stakes and hubs have been removed.

Geotextile Placement. During geotextile placement, the CQA monitor must:

- Observe the geotextile as it is deployed and record all defects and disposition of the defects (panel rejected, patch installed, etc.). Repairs are to be made in accordance with the specifications outlined in Section 3.5.4.
- Observe that equipment used does not damage the geotextile by handling, equipment transit, leakage of hydrocarbons, or other means.
- Observe that people working on the geotextile do not smoke, wear shoes that could damage the geotextile, or engage in activities that could damage the geotextile.
- Observe that the geotextile is securely anchored in an anchor trench.
- Observe that the geotextiles are anchored to prevent movement by the wind.
- Observe that the panels are overlapped a minimum of six inches.
- Examine the geotextile after installation to ensure that no potentially harmful foreign objects are present.
- Observe that seams (where required) are continuously sewn or thermal bonded in accordance with the manufacturer's recommendations and the project specifications outlined in this LQCP.

The CQA monitor must inform both the contractor and POR if the above conditions are not met.

3.4.4 Repairs

Repair procedures include:

- Patching – used to repair large holes, tears, and large defects.
- Removal – used to replace areas with large defects where the preceding method is not appropriate.

Holes, tears, and defects must be repaired in the following manner. Soil or other material which may have penetrated the defect must be removed completely prior to repair. If located on a slope, the defect must be patched using the same type of geotextile and continuously seamed into place. Should any tear, hole, or defect exceed 30 percent of the width of the roll, the roll will be cut off and the defect removed or the roll removed and replaced. If the defect is not located on a slope, the patch must be made using the same type of material seamed into place with a minimum of 24 inches overlap in all directions. Seams will be either thermal bonded or sewn in accordance with the manufacturer's recommendations.

3.5 Drainage Geocomposite – Geonet and Geotextile

A drainage geocomposite will be used for the liner LCS and temporary groundwater dewatering system (see Section 6). The drainage geocomposite will meet the requirements set forth in Appendix IIIC – Leachate and Contaminated Water Management Plan of the Site Development Plan along with this LQCP. Manufacturer's testing for geotextile and drainage geocomposite for the composite liner are listed in Table 3-3. Third-party laboratory transmissivity conformance testing for the geocomposite liner is listed in Table 3-4. The drainage geocomposite for the composite liner will meet the required properties listed in Table 3-3 and Table 3-4. The drainage geocomposite for the groundwater dewatering system will meet the required properties listed in Table 3-5.

Reference to "geocomposite thickness" within this LQCP and in supporting calculations (Appendix IIIC) refers to the thickness of the geonet, not the overall thickness of the geocomposite. The transmissivity values used for the calculations supporting this LQCP may or may not be representative of actual transmissivity values for every geocomposite manufacturer and may require a prospective material supplier to provide a geocomposite that varies in thickness from the geocomposite presented in this LQCP to meet the minimum transmissivity criteria set forth in this LQCP.

3.5.1 Delivery

Upon delivery the CQA monitor must observe the following:

- The drainage geocomposite is wrapped in rolls with protective covering.
- The rolls are not damaged during unloading.
- Protect the drainage geocomposite from mud, soil, dirt, dust, debris, cutting, or impact forces.
- Each roll must be marked or tagged with proper identification.

Any damaged rolls will be rejected and removed from the site or stored at a location, separate from accepted rolls, designated by the Operator. All rolls which do not have proper manufacturer's documentation will also be stored at a separate location until all documentation has been received and approved.

3.5.2 Testing

The drainage geocomposite manufacturer (or supplier) will conduct quality control testing and certify that all materials delivered to the site comply with the specifications listed in Table 3-3, Table 3-4, and Table 3-5. The minimum testing frequency will be one test sample per 100,000 square feet of geocomposite (or geonet/geotextile). See footnotes 2 and 3 of Table 3-4 and footnote 2 of Table 3-5 for testing frequency for transmissivity. The material certifications will be reviewed by the POR to verify that the geocomposite meets the values given in Table 3-3, Table 3-4, and Table 3-5.

Geonet will be tested by the manufacturer for thickness, tensile strength, and carbon black content. Geotextile will be tested for mass per unit area, grab tensile strength, and AOS. The finished geocomposite will be tested for peel adhesion and transmissivity (note that the geocomposite transmissivity tests need to be conducted by a third-party laboratory only under the specific conditions listed in Table 3-4 and Table 3-5). The manufacturer's testing for drainage material is also summarized in Table 3-3 and Table 3-5.

Where optional procedures are noted in the test method, the specification requirements of this LQCP prevail. The CQA monitor will review all test results and will report any nonconformance to the POR and to the contractor.

3.5.3 Installation

Surface Preparation. Prior to drainage geocomposite installation, the CQA monitor will observe the following:

- All lines and grades have been verified by the surveyor (where required).
- The subgrade has been prepared in accordance with the earthwork specifications outlined in Section 2.
- When placed over a geomembrane, the geomembrane installation, including all required documentation, has been completed.
- The supporting surface does not contain stones that could damage the geocomposite or the geomembrane.

Drainage Geocomposite Placement. During placement, the CQA monitor will:

- Observe the drainage geocomposite as it is deployed and record defects and disposition of the defects (panel rejected, patch installed, etc.). Repairs are to be made in accordance with the specifications outlined in Section 3.5.4.
- Verify that equipment used does not damage the drainage geocomposite or underlying geomembrane by handling, trafficking, leakage of hydrocarbons, or by other means.
- Verify that people working on the drainage geocomposite do not smoke, wear shoes that could damage the drainage geocomposite, or engage in activities that could damage the drainage geocomposite or underlying geomembrane.
- Verify that the drainage geocomposite is anchored to prevent movement by the wind (the contractor is responsible for any damage resulting to or from windblown drainage geocomposite).

- Verify that the drainage geocomposite remains free of contaminants such as soil, grease, fuel, etc.
- Observe that the drainage geocomposite is laid smooth and free of tension, stress, folds, wrinkles, or creases.
- Observe that equipment or geocomposite complies with Section 3.6.
- Observe that on slopes the drainage geocomposite is secured in the liner anchor trench and then rolled down the side slope.
- Observe that adjacent rolls of drainage geocomposite are overlapped a minimum of six inches, tied, and seamed in accordance with the manufacturer's recommendations.
- Observe that tying is with plastic fasteners in accordance with the manufacturer's recommendations. In the absence of other specifications, the drainage geocomposite panels will be tied approximately every 5 feet along the roll length (edges) and every 1 foot along the roll width (ends).
- Observe that the geotextile component is overlapped and either heat bonded or sewn together.

Table 3-3
Manufacturer Certification Tests and Properties for the Leachate Collection System Drainage Geocomposite

Material	Test	Standard	Required Property[2]	Test Frequency
Geotextile (Before Lamination)	Mass/Unit Area[4]	ASTM D 5261	6 oz/sy	See Note 1
	Apparent Opening Size	ASTM D 4751	0.21 mm	
	Grab Strength	ASTM D 4632	157 lbs	
	Tear Strength	ASTM D 4533	55 lbs	
	Puncture Strength	ASTM D 6241	310 lbs	
	Permitivity	ASTM D 4491	0.2 sec^{-1}	
HDPE Geonet (Before Lamination)	Specific Gravity	ASTM D 1505	0.95 g/cm^3	Per 50,000 lb.
	Thickness	ASTM D 5199	0.25 inch (bottom liner)	Per 50,000 lb.
	Carbon Black	ASTM D 1603	2%	Per 100,000 lb.
	Tensile Strength	ASTM D 5035	45 lb/in	Per 50,000 lb.
Drainage Geocomposite	Transmissivity	ASTM D 4716	See Table 3-4	Per 200,000 lb.
	Ply Adhesion	ASTM D 7005	1.0 lb/in	Per 100,000 lb.

1 Minimum Average Roll Valve (MARV) except Apparent Opening Site (AOS) is Maximum Average Roll Valve (MaxARV) per manufacturer's recommendations.

2 Minimum required property values for the geotextile and HDPE geonet are based on calculations provided in Appendix IIIC-B. The geonet properties are based on values specified in GRI standard GM-13. In addition, each material will be tested prior to construction to verify that it meets the minimum required properties. Actual geonet thickness, if greater than the minimum, will be determined by manufacturer quality control testing and recommendations.

3 Reference to "geocomposite thickness" within the LQCP and in supporting calculations (Appendix IIIC) refers to the thickness of the geonet, not the overall thickness of the geocomposite. The transmissivity values used for the calculations supporting this LQCP may or may not be representative of actual transmissivity values for every geocomposite manufacturer and may require a prospective material supplier to provide a geocomposite that varies in thickness from the geocomposite presented in this LQCP in order to meet the minimum transmissivity criteria set forth in this LQCP.

4 Higher mass/unit area geotextile may be used; however, it will be required to pass all strength requirements and geocomposite transmissivity requirements under varying loading conditions.

Table 3-4
Third-Party Laboratory Transmissivity Conformance Test for the Leachate Collection System Drainage Geocomposite

Material	Standard	Gradient	Test Point	Normal Pressure (PSF)	Leachate Collection System Design Demonstration Values		Required Property[2,4]
					Thickness[4] (In)	Hydraulic Conductivity (cm/s)	Minimum Transmissivity (m^2/s)
Single-Sided Drainage Geocomposite (Floor Grades)	ASTM D 4716	0.025	1	740	0.248	0.27	3.75E-05
			2	2,895	0.236	1.39	2.62E-04
			3	6,045	0.214	2.05	4.42E-04
			4	14,545	0.164	0.87	1.81E-04
			5	14,833	0.162	0.87	2.05E-04
Double-Sided Drainage Geocomposite (Side-Slope Grades)	ASTM D 4716	0.33	1	740	0.248	0.05	6.94E-06
			2	2,895	0.236	0.30	5.65E-05
			3	6,045	0.214	0.20	4.31E-05
			4	14,545	0.164	0.17	3.55E-05
			5	14,833	0.162	0.10	2.36E-05

1 The minimum testing frequency will be one test sample per 100,000 sf. The drainage geocomposite will be single-sided for the floor grades of the bottom liner. The drainage geocomposite will be double-sided for the sideslopes of the bottom liner.

2 As noted in Appendix IIIC, Appendices IIIC-A and IIIC-A.2, the transmissivity of the single-sided and double-sided geocomposite for the undeveloped areas will be measured at the gradient specified above, normal pressures at each test point, boundary conditions consisting of soil/geocomposite/geomembrane with minimum seating time of 100 hours and will be performed for the first 100,000 sf of liner construction. For each additional 100,000 sf of geocomposite placement area, one additional transmissivity test will be performed under the maximum normal stress (i.e., 14,833 psf) or higher with all the same assumptions. The transmissivity shall be greater than specified above.

3 Minimum required property values for the drainage geocomposite transmissivity are based on calculations provided in Appendix IIIC-A. The geonet properties are based on values specified in GRI standard GM-13. In addition, each material will be tested prior to construction to verify that it meets the minimum required properties. Actual geonet thickness, if greater than the minimum, will be determined by manufacturer quality control testing and recommendations.

4 Reference to "geocomposite thickness" within this LQCP and in supporting calculations (Appendix IIIC) refers to the thickness of the geonet, not the overall thickness of the geocomposite. The transmissivity values used for the calculations supporting this LQCP may or may not be representative of actual transmissivity values for every geocomposite manufacturer and may require a prospective material supplier to provide a geocomposite that varies in thickness from the geocomposite presented in this LQCP in order to meet the minimum transmissivity criteria set forth in this LQCP.

Table 3-5
Manufacturer Quality Control Tests and Properties for the Dewatering System Geocomposite

Material	Test	Standard	Required Property	Test Frequency
Geotextile (Before Lamination)	Unit Weight	ASTM D 5261	6 oz/sy	See Note 1
	Apparent Opening Size	ASTM D 4751	0.21 min	
	Grab Strength	ASTM D 4632	157 lbs	
	Tear Strength	ASTM D 4533	55 lbs	
	Puncture Strength	ASTM D 6241	310 lbs	
	Permittivity	ASTM D 4491	0.2 sec^{-1}	
HDPE Geonet (Before Lamination)	Specific Gravity	ASTM D 1505	0.95 g/cm^3	Per 50,000 lb.
	Thickness	ASTM D 5199	0.20 inch	Per 50,000 lb.
	Carbon Black	ASTM D 1603	2%	Per 100,000 lb.
	Tensile Strength	ASTM D 5035	45 lb/in	Per 50,000 lb.
Drainage Geocomposite	Transmissivity[2]	ASTM D 4716	2.5 x 10^{-5} m^2/s	Per 200,000 lb.
	Peel Adhesion	ASTM D 7005	1.0 lb/in	Per 100,000 lb.

1 Minimum Average Roll Valve (MARV) except Apparent Opening Site (AOS) is Maximum Average Roll Valve (MaxARV) per manufacturer's recommendations.

2 As noted in Appendix IIID-C, the transmissivity of the dewatering system geocomposite will be measured at a minimum gradient of 0.33 (sideslope) under a minimum normal pressure of 5,600 psf with a minimum seating time of 100 hours. Testing shall be performed under soil/geocomposite/plate configuration. Third party testing of underdrain geocomposite will not be required.

3 Minimum required property values for the geotextile and drainage geocomposite transmissivity are based on calculations provided in Appendix IIID-C. The geonet properties are based on values specified in GRI standard GM-13. In addition, each material will be tested prior to construction to verify that it meets the minimum required properties. Actual geonet thickness, if greater than the minimum, will be determined by manufacturer quality control testing and recommendations.

4 Reference to "geocomposite thickness" within this LQCP and in supporting calculations (Appendix IIID-C) refers to the thickness of the geonet, not the overall thickness of the geocomposite. The transmissivity values used for the calculations supporting this LQCP may or may not be representative of actual transmissivity values for every geocomposite manufacturer and may require a prospective material supplier to provide a geocomposite that varies in thickness from the geocomposite presented in this LQCP to meet the minimum transmissivity criteria set forth in this LQCP.

Table 3-6
Manufacturer Certification Tests and Properties for the
Leachate Collection System and Dewatering System Chimney Drain Geotextile

Material	Test	Standard	Required Property[2]	Test Frequency
Geotextile	Mass/Unit Area[3]	ASTM D 5261	6 oz/sy	See Note 1
	Apparent Opening Size	ASTM D 4751	0.25 mm	
	Grab Strength	ASTM D 4632	157 lbs	
	Tear Strength	ASTM D 4533	55 lbs	
	Puncture Strength	ASTM D 6241	310 lbs	
	Permittivity	ASTM D 4491	0.2 sec^{-1}	

[1] Minimum Average Roll Valve (MARV) except Apparent Opening Site (AOS) is Maximum Average Roll Valve (MaxARV) per manufacturer's recommendations.

[2] Minimum required property values for the geotextile are based on calculations provided in Appendix IIIC-B. The geotextile properties are based on values specified in GRI standard GM-13.

[3] Higher mass/unit area geotextile may be used; however, it will be required to pass all strength requirements and geocomposite transmissivity requirements under varying loading conditions.

3.5.4 Repairs

Repair procedures include:

- Holes or tears in the drainage geocomposite will be repaired by placing a patch extending 2 feet beyond the edges of the hole or tear.
- Secure patch to the originally installed drainage geocomposite by tying every 6 inches.
- Where the hole or tear width across the roll is more than 50 percent of the roll width the damaged area will be cut out across the entire roll and the two portions of the drainage geocomposite will be jointed.

3.6 Equipment on Geosynthetic Materials

Construction equipment on the bottom liner system will be minimized to reduce the potential for liner puncture. The CQA monitor will verify that small equipment such as generators are placed on scrap liner material (rub sheets) above geosynthetic materials in the liner system. Aggregate drainage layers and/or protective cover will be placed using low ground pressure equipment. The CQA monitor will verify that the geosynthetics are not displaced while the soil layers are being placed.

Unless otherwise specified by the POR, all lifts of protective soil material placed over geosynthetics will conform with the following guidelines.

Equipment Ground Pressure (psi)	Minimum Lift Thickness (in)
<5.0	12
5.1 – 8.0	18
8.1 – 16.0	24
>16.0	36

No equipment will be left running and unattended over the lined area.

3.7 Reporting

The POR will submit to the TCEQ a GLER for approval of the flexible membrane liner, leachate collection system and protective cover. Section 8 describes the documentation requirements.

4 CONSTRUCTION QUALITY ASSURANCE FOR GEOSYNTHETIC CLAY LINER

4.1 Introduction

GCL may be used in lieu of soil liner in the composite liner system. The GCL will be covered with geomembrane, drainage geocomposite, and a minimum 24-inch-thick protective cover. Material properties based on Geosynthetic Research Institute recommendations as described in GRI-GCL3 have been included in Table 4-1 – Required Testing for GCL Materials. The GCL will meet or exceed the required properties.

4.2 Material Requirements

1. A reinforced GCL which consists of bentonite encapsulated between two geotextiles, one nonwoven and one woven, which are needle punched together will be used. The GCL materials and its components will be tested in accordance with Table 4-1 by the supplier/GCL manufacturer and a third-party independent laboratory and will have the required values listed in Table 4-2. A certificate of analysis for each GCL panel will be submitted as part of the quality control documentation. The GCL permeability will be certified by the manufacturer and will be tested by an independent laboratory at frequencies included in Table 4-1. The manufacturer will provide recommended seaming procedures and supporting test data (flow box or other suitable device). The manufacturer will provide documentation showing the GCL seams are no more permeable than the GCL itself at a confining pressure anticipated in the field. The nonwoven side of the GCL will be in contact with the geomembrane. Table 4-2 includes further details for the GCL material.
2. The GCL will be shipped in rolls, which are wrapped individually in relatively impermeable and opaque protective covers. GCL rolls will be offloaded with equipment that will not damage the GCL rolls. The roll may be stacked only as allowed by manufacturer's recommendations. The GCL rolls must be stored above ground (i.e., wooden pallets) and covered with a waterproof tarpaulin.

3. GCL testing will be performed by the manufacturer and a third-party independent laboratory. The POR will review the manufacturer's certification (quality control certificate) and verify that the GCL meets the values given in the plan or specifications for those tests listed in Table 4-1. Required quality control documentation will be submitted to the POR a minimum of 7 days prior to deployment of any GCL. Requirements for GCL materials are listed in Table 4-2.
4. The POR will perform verification testing as required by additional detailed construction specifications or as required by the POR.

4.3 GCL Installation

Installation of GCL will have continuous on-site monitoring during construction by the POR or his designated representative. The installer will provide a panel layout plan, which will be reviewed by the POR prior to any material deployment. The POR must review field conditions and approve a revised panel layout plan if the field conditions vary from the original plan layout.

4.3.1 Subgrade Preparation

The surface of subgrade for the GCL installation will be stable. It will be smooth and free of foreign and organic material, sharp objects, exposed soil or aggregate particles greater than 3/4 inch (or less if recommended by the manufacturer), or other deleterious materials. Standing water or excessive water on the subgrade will not be allowed. If standing water is encountered it will be removed and soils with excessive moisture will be excavated and replaced with suitable borrowed soils to provide a firm, smooth-surfaced base for GCL placement. The POR will verify that the subgrade does not contain excessive moisture, and that soft soil is removed from the area. A firm, smooth-surfaced base grade will be established before GCL placement. The POR may require additional compaction and grading that will result in a smooth surface (e.g., proof rolling), as necessary.

Prior to GCL installation, the POR will verify the following:

- The grades below the GCL have been verified and accepted by the GCL contractor.
- Required documentation for subgrade preparation below the GCL have been completed and are acceptable.
- The supporting surface has been rolled to provide a smooth surface and does not contain materials, which could damage the GCL or adjacent layer. The subgrade will be rolled with a smooth-drum compactor. Protrusions extending more than 3/4 inches (or less if recommended by the manufacturer) from the base grade surface will be either removed or pushed into the surface with a smooth-drum compactor.

4.3.2 Deployment

Equipment used to deploy GCL over soil must not cause excessive rutting of the GCL subgrade. Deployed GCL panels should contain no folds or excessive slack. Generators, gasoline or solvent cans, tools, or supplies must not be stored directly on GCL. Installation personnel must not smoke or wear damaging shoes when working on GCL.

GCL seams will be constructed overlapping their adjacent edges a minimum of 12 inches. GCL seams will be constructed per manufacturer's directions. The CQA monitor will verify that steps are taken to minimize the presence of loose soil or other debris within the overlap zone.

GCL on sideslopes must not be unrolled in a direction perpendicular to the direction of the slope. GCL should be anchored temporarily (e.g., sandbags) at the top of the slope and then unrolled working from the top of the slope so as to keep the material free of wrinkles and folds, and GCL should be anchored at the bottom of the slope.

Horizontal seams will only be allowed on the slopes under one of the following conditions:

- 2 feet of overlap with horizontal seams being staggered.
- 1 foot of overlap with the underlying panel having a 1-foot runout anchored with 6 inches of subgrade.

Manufacturer hydraulic conductivity testing of GCL seams must be performed by using a flow box or other suitable device per adjoining material and type. Hydraulic conductivity value must be equal to or less than the specified hydraulic conductivity value for the GCL ($5x10^{-9}$ cm/s).

The POR or his designated representative will observe the GCL as it is deployed for even bentonite distribution, thin spots, or other panel defects. Defects and the disposition of the defects (panel rejected, patch installed, etc.) will be recorded. Repairs are to be made in accordance with the specifications at the discretion of the POR. The POR will verify that only panels that can be covered on the same day with an FML are deployed and that the GCL panels are not placed during wet, rainy weather. In accordance with the construction specifications, the POR will also verify the following:

- Proper GCL deployment techniques.
- Proper overlap during deployment.
- Seams between GCL panels are constructed per manufacturer's recommendations.
- The bentonite does not exceed the specified amount of hydration prior to covering.

- Defects are patched and overlapped properly.
- On sideslopes, the GCL is anchored at the top and then unrolled.
- Observe that no debris is trapped beneath or within the GCL.
- Observe that broken needle pieces do not exist within needle-punched GCL.
- Observe that wind speed is less than 40 miles per hour unless a lower wind speed is recommended by the manufacturer. At a minimum, a hand-held anemometer will be used, and readings will be taken at least once a day during GCL deployment to verify that the wind speed is less than 40 miles per hour.

The POR will observe the GCL for premature hydration visually and by walking over the GCL to locate soft spots. GCL that has prematurely hydrated according to the specifications will be removed and replaced with new GCL. These observations will be documented in the GCLER.

4.3.3 GCL Anchor Trench

The GCL anchor trench will be left open to allow installation of FML. Temporary anchoring will be provided until the placement of FML by using sandbags as discussed in Section 4.3.2. Slightly rounded corners will be provided in anchor trenches where the GCL enters the trench to avoid sharp bends in the GCL. No loose soil (e.g., excessive water content) will be allowed to underlie the anchored components of the liner system. Backfilling of soil will be in accordance with Section 2.3.7.

4.3.4 Patching

Torn or otherwise damaged GCL (with no loss of bentonite from the GCL) must be patched with the same type of GCL. The GCL patch must extend at least 12 inches beyond the damaged area and must be bonded to the main GCL to avoid shifting during backfilling. If the GCL damage includes loss of bentonite, the patch must consist of full GCL extending at least 12 inches beyond the damaged area. Lapping procedures must be the same as specified for original laps of GCL panels.

4.4 GCL Protection

Protection of GCL will be verified from production to deployment using the procedures discussed in this section. The manufacturer will provide inspection reports demonstrating that needle-punched nonwoven geotextile was inspected using metal detectors for the presence of broken needles and were found to be needle free. GCL must be rolled by the manufacturer in a fashion to prevent collapse during transit. Rolls will be labeled and bagged in a packaging that is resistant to water.

Visual inspection of each GCL roll will be made during unloading to identify any packaging that has been damaged. Rolls with damaged packaging will be marked and set aside for further inspection. The packaging will be repaired, for acceptable GCL rolls, prior to being placed in storage. If necessary, the party responsible for unloading the GCL will contact the manufacturer prior to shipment to ascertain the suitability of the proposed unloading methods and equipment.

The GCL-installing contractor will be responsible for the storage of GCL material. A dedicated storage area will be selected at the job site or at an alternate off-site area per owner's direction. The selected area will be level, dry, and well drained. Rolls will be stored in a manner that prevents sliding or rolling from the stacks. Rolls should be stacked no higher than three rolls to protect the integrity of roll cores and ensure safe material handling. Stored GCL materials will be covered with a plastic sheet or tarpaulin until it is installed. The integrity and legibility of the labels will be preserved during storage.

Construction equipment (other than low contact pressure rubber-tired vehicles such as ATVs or golf carts) on the GCL will not be allowed. The CQA monitor will verify that small equipment such as generators is placed on scrap FML material (rub sheets). The protective cover will be placed (using low ground pressure equipment as discussed under Section 2.3.6) as soon as possible after installation of FML and drainage layer. Refer to Section 3.6 for equipment operating requirements over geosynthetic materials.

The CQA monitor will verify that GCL (or overlying geosynthetics) are not displaced or damaged while overlying materials are being placed.

4.5 Reporting

The POR will submit to the TCEQ a GCLER for approval of the GCL. Section 8 describes the documentation requirements.

Table 4-1
Required Testing for GCL Materials

Responsible Party	Test	Type of Test	Standard Test Method	Frequency of Testing
Supplier or GCL Manufacturer	Bentonite[1]	Free Swell	ASTM D 5890	per 50 tons (minimum of 1 test for each construction event)
		Fluid Loss	ASTM D 5891	
	Geotextile	Mass/Unit Area	ASTM D 5261	per 25,000 sy
		Grab Tensile Strength	ASTM D 4632	
GCL Manufacturer	GCL Product	Clay Mass/Unit Area	ASTM D 5993	per 5,000 sy
		Bentonite Moisture Content	ASTM D 5993	
		Tensile Strength	ASTM D 6768	per 25,000 sy
		Peel Strength	ASTM D 6496	per 5,000 sy
		Permeability [2]	ASTM D 5887	per 30,000 sy
		Lap Joint Permeability	Flow box or other suitable device	per GCL adjoining material and lap type[3]
Independent Laboratory (Conformance Testing)	GCL Product	Clay Mass/Unit Area	ASTM D 5993	per 100,000 sf
		Permeability	ASTM D 5887	
		Direct Shear [4]	ASTM D 6243	One per GCL/adjoining material type

1 Tests to be performed on bentonite before incorporation into GCL.
2 Report last 20 permeability values, ending on production date of supplied GCL.
3 May also be performed by an independent laboratory as part of conformance testing.
4 Not applicable for slopes of 4 percent or flatter. Testing must be on material in hydrated states and must use strain rates, confining pressures, and other parameters, which simulate field conditions. Only reinforced GCL (bentonite encapsulated between two geotextiles, one nonwoven and one woven, which are needle punched together) will be used. The nonwoven side of the GCL will be in contact with the geomembrane. Refer to Appendix IIIE – Geotechnical Report for the stability analysis.

Table 4-2
Required Properties for Reinforced GCL Materials

Property	Required Values[1]
Free Swell (milliliters)	24 (minimum)
Fluid Loss (milliliters)	18 (maximum)
Bentonite Mass per Unit Area[2] (lb/sf)	0.75 (minimum)
Tensile Strength[3] (lb/in)	23 (minimum)
Peel Strength (lb/in)	2.1 (minimum)
GCL Permeability[4] (cm/s)	$5x10^{-9}$ (maximum)
Lab Joint Permeability[5, 6] (cm/s)	$5x10^{-9}$ (maximum)

1 Manufacturer will demonstrate that the above listed values will be met prior to shipment in accordance with Table 4-1.
2 Bentonite mass per unit area of GCL must be reported at zero percent moisture content for the finished product.
3 Value is required for GCL and geotextile.
4 Permeability is listed for the finished product at a gradient of 1.0.
5 Minimum overlap is 12 inches. The values listed are minimum dry bentonite amount for 12 inches of overlap. Manufacturer-specified value will be used if it is higher.
6 Manufacturer will provide certification that seams are no more permeable than the GCL material under similar normal stress conditions.

5 QUALITY ASSURANCE FOR PIPING

5.1 Introduction

This section describes CQA procedures for the installation of HDPE pipe for the leachate collection system used for the composite liner. This plan stresses careful documentation during the quality assurance process, from the selection of materials through installation.

The goal of the pipe quality assurance program is to assure that proper construction techniques and procedures are used, and that the project is built in accordance with the project construction drawings and specifications that will be developed in accordance with this LQCP for each liner construction. The following specifications apply to the leachate collection system piping:

- Minimum internal diameter = 5.845 inches for leachate collection pipe and nominal diameter of 18 inches for riser pipe
- Standard dimension ratio = 17
- Perforation hole diameter = 0.5 inches (if slotted pipe is used, standard slot width = 0.125 inches)
- Young's modulus for pipe material = 33,000 psi
- For LCS design/requirements regarding chemical resistance, refer to Appendix IIIC.

The quality assurance program is intended to identify and define problems that may occur during construction and to observe that these problems are corrected before construction is complete. A construction report, prepared after project completion, will document that the constructed facility meets design standards and specifications.

5.2 Pipe and Fittings

5.2.1 General

Construction must be conducted in accordance with the project construction drawings and specifications for each liner constructed. Piping design and specifications are provided in Appendix IIIC – Leachate and Contaminated Water Management Plan. To monitor compliance, a quality assurance program will be implemented that includes: (1) a review of the manufacturer's quality control testing, (2) material conformance testing, and (3) construction monitoring. Conformance testing refers to testing by an independent third-party laboratory that will take place prior to material installation on materials delivered to the site.

5.2.2 Delivery

The CQA monitor will observe:

- That upon delivery, the pipe and pipe fittings are in compliance with the requirements of the construction specifications that will be developed in accordance with this LQCP for each liner construction.
- That a storage location is selected in which the pipe and pipe fittings are protected from excessive heat, cold, construction traffic, hazardous chemicals, and solvents. If the pipe and pipe fittings are stored at a location where other construction materials are present, the CQA monitor will assure that stacking or insertion of the other construction materials onto or into the pipe and pipe fitting is prohibited. The CQA monitor will periodically examine the storage area to observe that the pipe fittings are undamaged and have been protected.
- That upon transporting pipe and fittings from the storage location to the construction site the contractor will use pliable straps, slings, or rope to lift the pipe. Steel cables or chains will not be allowed to transport or lift the pipe.
- That the contractor will provide that a pipe greater than 20 feet in length will be lifted with at least two support points. The contractor will not drop, impact, or bump into the pipe, particularly at the pipe ends. Pipe and fitting ends must be cleaned of all dirt, debris, oil, or any other contaminant which may prohibit making a sound joint.

The CQA monitor will document all activities associated with the handling and storage of this material to maintain compliance with this portion of the CQA plan.

5.2.3 Conformance Testing

Prior to the installation of pipe, the pipe manufacturer will provide the Operator and the POR a quality control certificate for each lot or batch of pipe provided. The quality control certificate will be signed by a responsible party employed by the pipe manufacturer, such as the quality control manager. The quality control certificate and documentation will include:

- A description of the pipe delivered to the project, including but not limited to the strength classification, diameter, perforations, and production lot.
- Properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the specifications that will be developed in accordance with this LQCP for each liner construction, or equivalent.
- A certification that property values given in the properties sheet are minimum values and are guaranteed by the pipe manufacturer.
- A list of quantities and descriptions of materials other than the base resin which comprise the pipe.
- The sampling procedure and results of testing for actual samples manufactured in the same lot as the pipe delivered to the project.

The CQA monitor will observe that:

- The property values certified by the pipe manufacturer meet all of the specifications that will be developed in accordance with this LQCP for each liner construction.
- The measurements of properties by the pipe manufacturer are properly documented and the test methods used are acceptable.
- Verification that the quality control certificates have been provided at the specified frequency for all lots or batches of pipe, and that each certificate identifies the pipe lot/batch related to it.
- The certified properties meet the specifications that will be developed in accordance with this LQCP for each liner construction.

5.2.4 Pipe and Fitting Installation

Surface Preparation. Prior to pipe installation, the CQA monitor must observe the following:

- All lines and grades have been verified by the contractor and project surveyor.
- The pipe trenches are swept clean of any deleterious material which may damage the pipe or geomembrane or may clog the pipe.

- Pipe perforations for leachate collection system are drilled in the pipe outside of the drainage trench where the pipe is to be laid. The drill cuttings must be completely removed from the pipe prior to being placed in the drainage trench.
- Pipe perforations are to the correct size and spacing according to the project specifications that will be developed in accordance with this LQCP for each liner construction. Perforations can be either factory installed slots or factory predrilled holes or field drilled holes.

Pipe and Fitting Placement. During pipe and fitting installation, the CQA monitor will:

- Observe all pipe, pipe fittings, and joints as the pipe is being laid. The CQA monitor will observe that pipes and fittings are not broken, cracked, or otherwise damaged or unsatisfactory. Prior to fusing (if fusion welding is utilized), the pipe installer will provide a fusion surface area which is clean and free of moisture, dust, dirt, debris of any kind, and foreign material.
- If fusion welding is utilized, verify welder credentials and that the procedure is consistent with the pipe manufacturer's recommendations.
- Observe that the pipe and fittings are being constructed in accordance with specifications that will be developed in accordance with this LQCP for each liner construction and accepted practices.
- Observe that the people and equipment utilized to install the pipe do not damage the pipe or any other component of the liner system.

6 LINERS CONSTRUCTED BELOW THE HIGHEST GROUNDWATER LEVEL

6.1 Introduction

Liners constructed below the groundwater surface could potentially experience uplift due to hydrostatic pressure acting on the geomembrane liner. This section of the LQCP describes procedures for short term and long-term protection of the liner system due to hydrostatic pressure uplift that may result from liner construction below the groundwater table.

Long-term liner stability will be provided in the form of ballast that will be created by the weight of protective cover, solid waste, and final cover as applicable. Ballast calculations are included in Appendix IIID-B – Example Ballast Thickness Calculations. Ballast has been and will be provided for the entire area that has a composite liner that is below the estimated groundwater elevation. The highest measured groundwater surface used in determining the required ballasting is included in Appendix IIID-A.

6.2 Highest Measured Groundwater Levels

Groundwater is present within four distinct site-specific aquifers (Aquifer A, Aquifer B, Aquifer C, and Aquifer D). Groundwater at the facility has been evaluated using historical water-level data from the facility's former (pre-2023) and existing groundwater monitor wells and piezometers, which are mostly screened within Aquifer A and Aquifer B sediments. These aquifers affect primarily the previously constructed portions of the landfill and are generally demonstrated to not influence the proposed future construction in Cells 10 thru 12 within the expansion area.

Groundwater elevations from the currently approved Subtitle D groundwater monitor wells are provided in Table 4-1 (Appendix G – Geology Report) and were measured during monitoring events dating back to March of 1995. These data were obtained from the facility's Subtitle D groundwater database, which is maintained by Hydrex Environmental, Inc. (Hydrex). In addition, Weaver Consultants Group began conducting monthly water level readings from the facility's existing groundwater monitor wells and 12 newly installed groundwater piezometers in August 2023, which are summarized in Table 4-2.

Groundwater potentiometric surface contour maps prepared from the 2023 WCG water level data are presented on Figures IIIG-D-2A through IIIG-D-2E (for site-specific aquifers A and B) and IIIG-D-3A through IIIG-D-3E (for site-specific aquifer C and D) in Appendix IIIG-D (all within the Appendix G – Geology Report). Additionally, a Highest Measured Groundwater Map has been prepared and is included as Appendix IIID-A of this LQCP.

As each new cell is designed, the highest measured water levels will be adjusted upward for possible higher well level data and the highest measured groundwater potentiometric contours for that cell will be used for design of ballast (based on measured groundwater levels after construction of the perimeter surface drainage features). Any temporary hydrostatic relief system design different than the one presented in Appendix IIID-C will be submitted under the provisions of §305.70(j) to the TCEQ for approval as a modification to the LQCP.

6.3 Temporary Dewatering System

The site will have a temporary underdrain dewatering system installed for the undeveloped areas, specifically including Cells 10, 11 and 12 as shown on Figure 1 (Appendix IIID-C). As described in the attached demonstration, the temporary underdrain installation will be limited to the future cell sideslopes as shown on Figure IIID-C-2. The underdrain system has been designed to collect groundwater from Aquifers B and C, as described in detail in Appendix IIIG – Geology Report. As discussed in Appendix IIIG, Aquifer A is generally at an elevation above the future cells, or are cut off by previous landfill construction, and Aquifer D is at a depth well below the excavation grades of the future cells. Based on this information, installation of temporary underdrains in the cell floor was deemed unnecessary.

The dewatering system will be comprised of a double-sided geocomposite groundwater collection layer, collection trenches and a collection sump (in cells 10 and 11) which will intercept and divert waters potentially contacting the bottom liner system. Groundwater seepage will drain into the geocomposite and will then discharge into the drainage trenches and perforated 4-inch-diameter high density polyethylene (HDPE) piping installed at the toe of the excavated sideslope, then drain within the trenches and piping to the respective collection sumps. Water from the sumps will be pumped to the surface by submersible pumps installed in 18-inch-diameter sideslope risers located at each sump. A site plan of the underdrain system installed into Cells 10-12 is presented as Figure 1 in Appendix IIID-C. Details of the underdrain dewatering system are presented in Appendix IIIA.

Water collected in the sumps and removed by submersible pump will drain into on-site stormwater management systems and then be discharged from the site consistent with the TPDES Stormwater Permit for the landfill. The pumps will be activated upon installation of the dewatering systems and will remain operational until the BER is approved. The pumps will be operated automatically by pressure transducers. Control levels for the automatic pump will be set to maintain sump liquid levels below the top of the sump.

The temporary dewatering systems will remain operational until enough ballast is placed in the form of protective cover and solid waste over the impacted area. Once sufficient ballast is in place and with the written approval of TCEQ, the dewatering system will be decommissioned.

A different hydrostatic pressure relief system may be used at the site if it is designed using the same methodology as the design included in Appendix IIID-C (e.g., relieve potential hydrostatic uplift pressure that may develop on the geomembrane liner) and approved by TCEQ through a permit modification. If during future cell design, the conditions are such that a different system (e.g., collector trenches, diversion channels adjacent to the sector, or a combination of options) is considered more efficient, the system will be designed and submitted to the TCEQ as a permit modification as described in Section 6.2, above.

6.4 Control of Seepage During Construction

Seepage from the other minor geological layers is not expected but may occur in localized areas. The temporary dewatering system is discussed in Section 6.3 and Appendix IIID-C. During liner construction, the subgrade must be maintained in a firm and unyielding condition to provide a satisfactory foundation for construction of the soil liner. If unexpected seepage is encountered, the POR will inspect the seeps and delineate the area. Per the POR's direction, the wet soils will be over-excavated and replaced with compacted general fill to seal off the seepage. Soft areas will be undercut to firm material and backfilled with suitable compacted general fill. The fill will be free from organics, foreign objects, and other deleterious matter. The fill will also be compacted sufficiently to provide a firm base for soil liner placement, as detailed in Section 2.

6.5 Temporary Dewatering System Materials

6.5.1 Dewatering System Drainage Aggregate

The drainage aggregate for the dewatering trench will have a hydraulic conductivity of at least 1 cm/s and a gradation as specified in Section 2.3.5 of this LQCP. The coarse aggregate will be tested for gradation (ASTM D 448) prior to delivery of granular material to the site. Gradation testing will be performed at a minimum frequency of 1 test per 3,000 cubic yards or per specific liner project if granular material used is less than this amount. The aggregate will be free of organic and foreign objects. Calcium carbonate content testing will not be required due to: (1) the dewatering system will be operational for a relatively short period of time (i.e., until enough waste-as-ballast is in place), and (2) water pH is expected to be neutral. The physical characteristics of the aggregate will be evaluated through visual observation and laboratory classification testing before construction and visual observation during construction. During installation, a CQA monitor will observe that granular material is free of organics and foreign objects. The test results for the coarse aggregate will be included in the SLER.

6.5.2 Dewatering System Piping

Typical total perforation will be 1 square inch per 1 foot of pipe length. Perforation sizes (hole diameter or slot width) will be in accordance with the gradation versus perforation requirements outlined in Section 6.5.1. Refer to Appendix IIID-C for slot and perforation sizing. Prior to installation of dewatering trench pipe, the CQA monitor must observe the following:

- Installation lines and grades have been verified by the contractor and project surveyor.
- The pipe trench is clean of any deleterious material which may damage the pipe or geofabric or may clog the pipe.
- Pipe perforations are drilled outside of the underdrain trench. The drill cuttings will be completely removed from the pipe prior to being placed in the drainage trench.
- Pipe perforations are to the correct size and spacing according to the project specifications that will be developed in accordance with this LQCP for each liner construction. Perforations can be either factory predrilled holes or field drilled holes.
- Observe all pipe, pipe fittings, and joints as the pipe is being laid. The CQA monitor will observe that pipes and fittings are not broken, cracked, or otherwise damaged or unsatisfactory. Prior to fusing, (if fusion welding is utilized) the pipe installer will provide a fusion surface area which is clean and free of moisture, dust, dirt, debris of any kind, and foreign material.
- If fusion welding is utilized, verify welder credentials and that the procedure is consistent with the pipe manufacturer's recommendations.
- Observe that the pipe and fittings are being constructed in accordance with specifications that will be developed in accordance with this LQCP for each liner construction and accepted practices.
- Observe that geotextile wrapping around the pipes and trench complies with project specifications outlined in Section 3.4.
- Observe that the people and equipment utilized to install the pipe do not damage the pipe or any other component of the dewatering system.
- Pipe grades will be established prior to pipe placement by grading the bottom of the trench.

6.5.3 Geotextile

The non-woven geotextile will be wrapped around the drainage stone and the collection pipe in the temporary dewatering trench. Required material properties shall meet the minimum requirements specified Table 3-5 of this LQCP. There will not be any direct contact between the geotextile and any compaction equipment.

6.5.4 Drainage Geocomposite

A drainage geocomposite will be used for the dewatering layer. The drainage geocomposite will meet the requirements set forth in Appendix IIID-C and Table 3-5 of this LQCP and will also meet the requirements of the construction drawings and specifications for each specific liner construction. Design flow capacity for the drainage geocomposite is estimated in Appendix IIID-C. The POR will ensure that the flow capacity of drainage geocomposite is equivalent to the required capacity estimated in Appendix IIID-C under similar loading conditions. Delivery, testing, installation, and repairs shall be consistent with Section 3.5 of this LQCP.

6.5.5 Documentation

Dewatering system installation will be incorporated into the SLER for each cell in accordance with Section 8. The installed dewatering system will be operated until a BER prepared in accordance with Section 8.3 is approved by the TCEQ.

6.5.6 Dewatering System Operation

When pumps are used for the dewatering system, regardless of its location, they will be inspected on a weekly basis to monitor and verify groundwater discharge at the pump outlet pipe. The pumps will be equipped with pressure transducers to control pump operation. All information generated associated with groundwater dewatering operation will be kept in the site operating record. The dewatering pipes will be cleaned out if it is determined that they are clogged. The determination may be based upon an unexpected decrease in flow of groundwater to the dewatering sump. Each groundwater dewatering system installed will be operational until a ballast evaluation report is approved by the TCEQ.

6.6 Liner System Ballast

Ballasting is required to protect the liner system from hydrostatic uplift in areas of the landfill excavation which have been identified to exist below the highest measured groundwater potentiometric surface as defined in Section 6.2. The protective cover soil above the liner system, as well as additional waste placed above the liner system will provide the necessary ballast (weight) for protection of the liner system from hydrostatic uplift.

The factor of safety against hydrostatic uplift must be calculated for those portions of the liner where the liner is below the estimated groundwater potentiometric surface. The calculated factor of safety against uplift at the liner (using the weight of the protective cover and waste) must be 1.5. The thickness of ballast required to ballast the uplift force must be calculated and submitted with the SLER or GLER. Procedures for calculating the anticipated hydrostatic uplift forces, factor of safety against uplift, and required thickness of ballast are included in Appendix IIID-B. Additionally, example ballast calculations are included in Appendix IIID-B. The estimated post-construction groundwater data as described in Section 6.2 will be used for ballast demonstration. The ballast demonstration included in Appendix IIID-B must be updated each time a dewatering system is installed to account for possible higher hydrostatic head measurements.

6.6.1 Waste-As-Ballast Placement Record

When waste is used for ballast, landfill personnel working under the supervision of the site manager will be on site full-time during the placement of the first 5 feet of waste over the liner system. The site operator will verify and document on a daily basis that this lower 5 feet of waste does not contain large bulky items or brush, which cannot be compacted to the required density. The site operator will also document on a daily basis that the waste used for ballast has been properly compacted with compaction equipment, which weighs in excess of 40,000 pounds. When waste is used as ballast, the factor of safety against hydrostatic pressure uplift at the geomembrane liner will be 1.5. This documentation will be placed in the site operating record.

Additionally, the Site Manager will complete and sign a waste-as-ballast placement record that will be attached to the BER (see Section 8 for BER required documentation). The form to be used by the Site Manager is included in Appendix IIID-D. One form will be required for each area (or combination of areas) described by approved liner evaluation reports.

6.7 Liner Performance Verification

Title 30 TAC §330.337(b) requires that the owner demonstrate that the liner system will not undergo uplift from hydrostatic forces during construction. Areas of liner requiring underdrains due to potential uplift from hydrostatic forces will be constructed in a manner that protects the subsequent liner installation from potential uplift, including inspection of the subgrade for wet or pumping areas and the installation of the underdrain geocomposite and piping prior to the placement of geomembrane. Additionally, calculations presented in this section demonstrate that the ballasting will comply with the requirements of Title 30 TAC 330.337(b), and that the ballasting and dewatering systems will be operated and maintained until the executive director determines that such systems are no longer needed.

When ballast is required for a liner, the POR or his representative will verify that the ballast meets the established criteria and uplift of the liner system did not occur during construction. In compliance with Title 30 TAC §330.337(f), the ballasting verification, including but not limited to inspections, compaction, weight, density of material, thickness of waste placed over the liner, and top elevations, will be documented in the BER, which will be submitted to the TCEQ for approval (see Section 8). In the event that uplift occurs, the POR will develop a corrective action to remediate the uplift. The POR will immediately contact the TCEQ and implement initial procedures as soon as the uplift is detected.

6.7.1 Observations for Indications of Seepage

The POR or his representative will observe the liner subgrade for the presence of seepage during construction. To aid in the documentation that short-term uplift has not occurred during ballast placement, the POR will provide a summary of where seepage, if any, was observed, the methods and procedures used to control the seepage, and observations that all seepage has been controlled.

6.7.2 Surveying During Construction

To document that short-term uplift has not occurred during construction of the liner, the POR will verify that the elevations of the geomembrane liner are consistent with the geomembrane liner elevations shown on the construction drawings. The POR will also verify that the protective cover elevations have not increased from those submitted with the GLER. The protective cover elevations will be taken once between the GLER approval and waste placement to document no short-term uplift has occurred. Survey measurements to check against uplift will be taken at a minimum frequency of one measurement per 10,000 square feet by a third-party surveyor.

6.8 Documentation

Documentation for issues related to construction below the high-water table will be included in the SLER, GLER, and BER. These documents are discussed in detail in Section 8. Documentation specifically related to liners constructed below the highest measured groundwater potentiometric surface will include:

- A current estimated potentiometric surface map and recent water-level information (Section 6.2).
- A discussion addressing the areas (if any) where the bottom of compacted clay liner extends below the highest estimated potentiometric level.
- A discussion identifying the groundwater condition.
- Uplift and ballast calculations for liners with an installed dewatering system.

- A discussion addressing any seepage that may have been encountered.
- Description of the dewatering system installed.
- The BER will contain the documentation substantiating that the appropriate depth of ballast has been placed over the liner system and that the liner did not experience hydrostatic uplift.

7 GEOTECHNICAL STRENGTH TESTING REQUIREMENTS

This section of the LQCP addresses the geotechnical strength requirements for the Subtitle D bottom liner. Each component of the Subtitle D bottom liner system is subject to the material testing requirements outlined in Sections 2 through 6 of this LQCP, as applicable. Prior to each Subtitle D bottom liner construction event, the geotechnical testing outlined in Table 7-1 will be performed using actual materials to verify that the Subtitle D bottom liner meets the material strength requirements set forth in Appendix IIIE-A-5 during shear strength conformance testing. A geotechnical analysis of the landfill is presented in Appendix IIIE.

The testing outlined in Table 7-1 and Appendix IIIE-A-5 will be performed under the supervision of the POR by a third-party independent geotechnical laboratory. The POR will ensure that (1) the strength values set forth in Appendix IIIE-A-5 are met or (2) provide an updated geotechnical analysis in the GLER that will be submitted to TCEQ after each liner construction event. If the geotechnical analysis is updated, the resulting factor of safety values must meet the recommended minimum factor of safety values established in Appendix IIIE.

Table 7-1
Recommended Strength for Various Parameters for Subtitle D Bottom Liner Components[1,2]

Interface Description	Peak Strength		Residual Strength	
	Adhesion (psf)	Friction Angle (degree)	Adhesion (psf)	Friction Angle (degree)
Liner System Component Interface				
Protective Cover/Double-sided Geocomposite Interface	200	20	270	15
Geocomposite/Textured HDPE Geomembrane Interface	200	19	120	10
Textured HDPE Geomembrane/Clay Liner Interface	210	18	50	14
Clay Liner (Internal)	100	18	80	13
Clay Liner/Underdrain Geocomposite Interface	200	18	80	10
Underdrain Geocomposite/Subgrade Interface	200	20	270	15
Protective Cover/Single-sided Geocomposite-Geotextile Interface	200	20	270	15
Single-sided Geocomposite-Geonet/Textured HDPE Geomembrane Interface	0	13	0	10
Textured HDPE Geomembrane/Clay Liner Interface	210	18	50	14
Alternative Liner System Component Interface				
Textured HDPE Geomembrane/Reinforced GCL Interface	850	25	400	10
Reinforced GCL (Internal)	800	18	380	11
Reinforced GCL/Subgrade Interface	100	18	--	--

1 The adhesion and interface friction angle of liner components will be determined using ASTM D5321 by a third-party verified geotechnical laboratory to verify they meet the values used in the slope stability analysis included in Appendix IIIE-A. Refer to Appendix IIIE-A for detailed strength information and procedures for determining acceptable shear strength parameters during conformance testing.

2 Interface and material peak and residual strength values in above table are typical values only. Actual shear strength values may vary. The adequacy of the interface and material shear strength values will be evaluated in accordance with the Appendix IIIE-A-5 Interface Shear Strength Conformance Testing Requirements.

8 DOCUMENTATION

The quality assurance plan depends on thorough monitoring and documentation of all construction activities. Therefore, the POR and CQA monitor will document that all quality assurance requirements have been addressed and satisfied. Documentation will consist of daily recordkeeping, testing and installation reports, nonconformance reports (if necessary), progress reports, photographic records, and design and specification revisions. The appropriate documentation will be included in the SLER, GCLER, GLER, and BER (if required). Standard report forms will be provided by the POR prior to construction.

8.1 Preparation of SLER, GCLER, and GLER

The POR will submit to the TCEQ a SLER for review and acceptance for each soil liner portion of the composite liner. After construction of the geosynthetics portion of the liner, the POR will submit a GCLER and a GLER to the TCEQ for review and acceptance. The GCLER and the GLER may be submitted as a single document. All of these reports will be approved by TCEQ prior to placement of solid waste over the specified constructed area.

Testing, evaluation, and submission of the SLERs, GCLERs, and GLERs for the composite liner system will be in accordance with this LQCP. The construction methods and test procedures documented in the SLERs, GCLERs, and GLERs will be consistent with this LQCP, the TCEQ MSWR, and specifications outlined in this LQCP.

At a minimum, the SLER, GCLER, and GLER will contain:

- A summary of all construction activities.
- A summary of all laboratory and field test results.
- Sampling and testing location drawings.
- A description of significant construction problems and the resolution of these problems.
- As-built record drawings signed and sealed by a Texas registered surveyor or professional engineer.
- A statement of compliance with the permit LQCP and construction plans.

- The reports will be signed and stamped by a professional engineer(s) licensed to practice in the state of Texas.

The as-built record drawings will accurately identify the constructed location of all work items, including the piping and anchor trenches. The POR will review and verify that as-built drawings are correct. As-built drawings will be included in the SLER, GCLER, and GLER as appropriate.

8.2 Reporting Requirements

The SLER, GCLER, and GLER will be signed and sealed by the POR and signed by an authorized representative and submitted in triplicate (including all attachments) to the MSW Permits Section of the Waste Permits Division of the TCEQ for review and acceptance. If the Executive Director provides no response, either written or verbal, within 14 days of receipt, the owner or operator may continue facility construction or operation. Any notice of deficiency received from the TCEQ will be promptly addressed and incorporated into the SLER/GCLER/GLER report. No solid waste will be placed over the constructed liner areas until the final acceptance is obtained from the TCEQ. Additionally, upon approval of this application if a new liner area is developed, prior to accepting any solid waste to the newly developed liner area, a pre-opening inspection will be requested. The TCEQ staff will conduct a pre-opening inspection within 14 days of the request. If the TCEQ does not provide a written or verbal response 14 days after conducting the pre-opening inspection, the newly developed liner area will be considered acceptable for solid waste placement, given that the SLER, GCLER, and GLER for the area are also submitted to the TCEQ in accordance with this section.

If a layer of waste is not placed over the top of the protective cover in the dewatering system installation area within 6 months, then the POR will visually observe that the liner is not damaged (e.g., excessive erosion) due to prolonged exposure of the surface of the protective cover. Repairs will be done promptly, and the POR will report findings and measures taken to repair damage in a letter report to the executive director for review and acceptance.

8.3 Ballast Evaluation Report

Existing and future dewatering system BERs will be submitted in accordance with this section. A BER will be completed and filed with the TCEQ documenting that enough ballast has been placed in a lined area to offset the potential hydrostatic uplift forces which may exist below the liner system. At a minimum, the information listed below will be included as applicable with the BER.

The top of protective cover elevations immediately after construction compared to the elevations obtained between SLER approval and waste placement, to document the liner did not undergo uplift prior to placement of waste (whether waste ballast is required or not).

- If waste is used for ballast, verification from the Site Manager that the weight of the compaction equipment being used to compact the waste ballast is no less than 40,000 pounds, and that this compaction equipment was utilized during the entire period of placing waste ballast.
- If waste is used for ballast, documentation of the observations that the initial 5 feet of waste used for ballast on the liner system is free of brush and large bulky items, which may not be compacted to the required density.
- A waste-as-ballast placement record (Appendix IIID-D) completed and signed by the Site Manager.
- Survey of the top of waste to document that the required waste ballast thickness has been placed.
- Water-level measurements taken in the site monitor well/piezometer system adjacent to the liner construction area to verify that the groundwater level has not exceeded the design high water level.
- Final ballast thickness calculation using procedures included in Appendix IIID-B and the as-built minimum densities and thicknesses for each component as well as updated groundwater levels.
- A BER will be prepared and signed and sealed by a professional engineer licensed to practice in Texas.

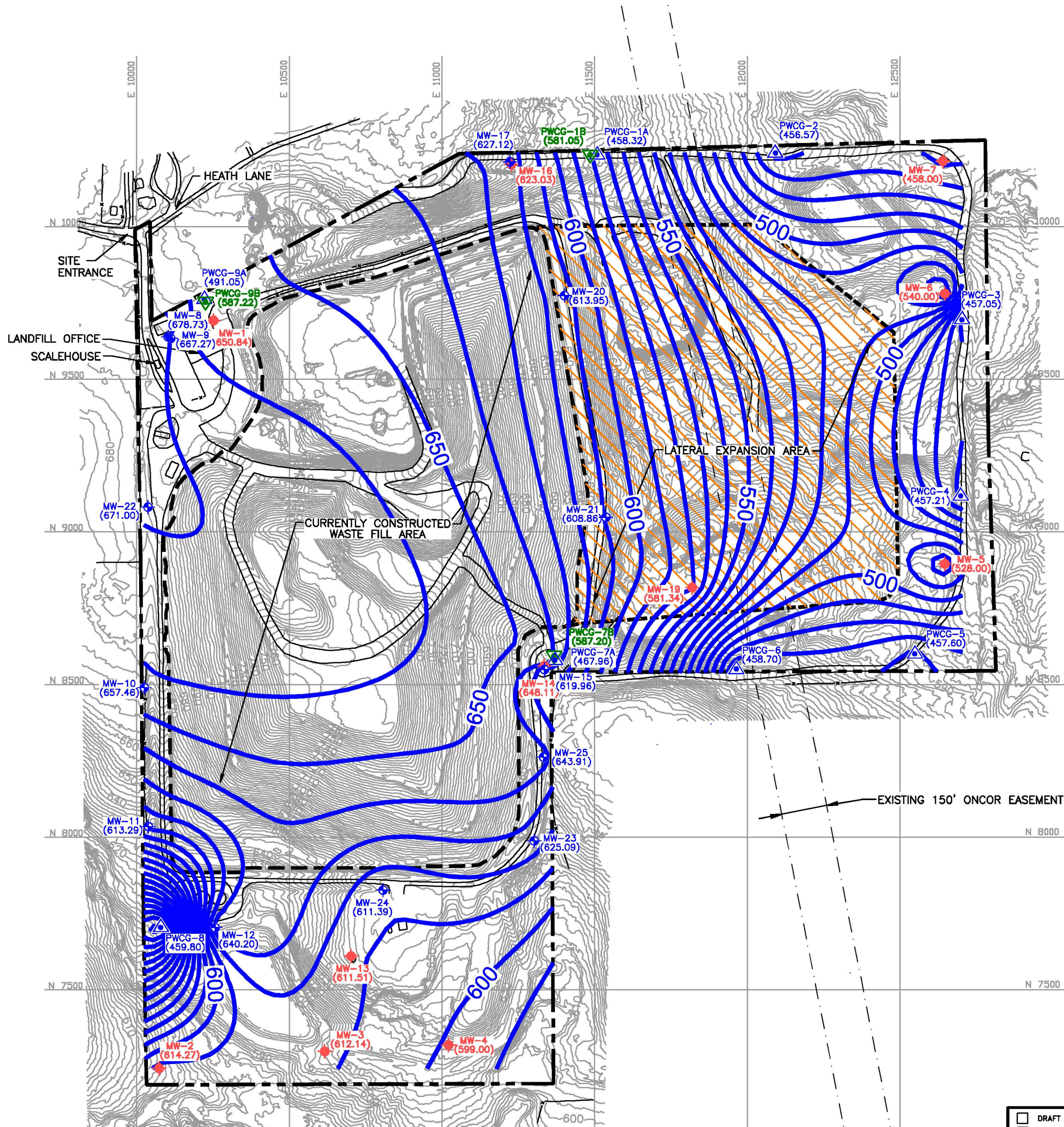
APPENDIX IIID-A

HIGHEST MEASURED GROUNDWATER INFORMATION

Includes Drawing IIID-A-1

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024



N

0 200 400
SCALE IN FEET

STATE OF TEXAS
DAVID E. POE
81734
LICENSED PROFESSIONAL ENGINEER
05/20/2024

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMITS OF WASTE
- PROPOSED LIMITS OF WASTE
- 630 EXISTING CONTOUR
- N 9500 SITE GRID COORDINATE
- PROPOSED LATERAL EXPANSION AREA
- MW-1 (661.0) FORMER GROUNDWATER MONITOR WELL WITH HIGHEST MEASURED GROUNDWATER ELEVATION POSTED IN FT-MSL
- MW-25 (663.2) EXISTING GROUNDWATER PIEZOMETER WITH HIGHEST MEASURED GROUNDWATER ELEVATION POSTED IN FT-MSL
- PWCG-1B (620.2) 2023 AQUIFER C EXPANSION PIEZOMETER WITH HIGHEST MEASURED GROUNDWATER ELEVATION POSTED IN FT-MSL
- PWCG-1A (618.3) 2023 AQUIFER D PIEZOMETER WITH HIGHEST MEASURED GROUNDWATER ELEVATION POSTED IN FT-MSL
- 650 HIGHEST MEASURED GROUNDWATER POTENTIOMETRIC SURFACE IN FT-MSL

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY COOPER AERIAL SURVEYS, CO. FROM AERIAL PHOTOGRAPHY FLOWN ON 11-10-2022. THE GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NORTH CENTRAL ZONE NAD 1983.
2. EXISTING GROUNDWATER MONITOR WELL LOCATION COORDINATES OBTAINED FROM AUGUST 2023 AS-BUILT SURVEY BY WEAVER CONSULTANTS GROUP.
3. GROUNDWATER ELEVATION DATA OBTAINED FROM THE FACILITY'S SUBTITLE D GROUNDWATER MONITORING DATABASE PROVIDED BY HYDREX IN DECEMBER 2023, FROM PREVIOUS SUBSURFACE EXPLORATION AND INVESTIGATION REPORTS, AND FROM GAUGING CONDUCTED BY WEAVER CONSULTANTS GROUP IN 2023 AND 2024.
4. HIGHEST MEASURED GROUNDWATER ELEVATIONS IN DEEP PIEZOMETERS PWCG-1A, PWCG-7A, PWCG-7B, PWCG-9A, AND PWCG-9B HAVE NOT BEEN CONTOURED DUE TO SIGNIFICANT DIFFERENCE IN GROUNDWATER DEPTH AS COMPARED TO ADJACENT SHALLOWER MONITORING POINTS.
5. HIGHEST MEASURED GROUNDWATER SURFACE CONTOURS CREATED USING THE HIGHEST MEASURED GROUNDWATER ELEVATION RECORDED AT EACH MEASUREMENT POINT AND DO NOT REPRESENT A SINGLE GAUGING EVENT OR ACTUAL GROUNDWATER FLOW.
6. HIGHEST MEASURED GROUNDWATER ELEVATIONS IN DEEP PIEZOMETERS PWCG-1A, PWCG-7A, PWCG-7B, PWCG-9A, AND PWCG-9B HAVE NOT BEEN CONTOURED DUE TO SIGNIFICANT DIFFERENCE IN GROUNDWATER DEPTH AS COMPARED TO ADJACENT SHALLOWER MONITORING POINTS.

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-076-11
CAD: IIID-A-1-GW CONTOUR MAP.DWG

DRAWN BY: RAA
DESIGN BY: DEP
REVIEWED BY: DEP/JP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
HIGHEST MEASURED GROUNDWATER MAP

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

WWW.WCGRP.COM

FIGURE IIID-A-1

O:\0120\76\EXPANSION 2023\PART III\IIID\IIID-A-1-GROUNDWATER CONTOUR MAP.dwg, byoung, 1:2

APPENDIX IIID-B

EXAMPLE BALLAST THICKNESS CALCULATIONS

Includes pages IIID-B-1 through IIID-B-8

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

BALLAST THICKNESS CALCULATIONS

The ballast requirements evaluated in this appendix are based on the estimated maximum groundwater contours shown on Drawings IIID-A-1 and IIID-B-8. As shown on Drawing IIID-B-1, the groundwater contours are projected across the site to facilitate ballast calculations. The required ballast depths shown on Drawing IIID-B-8 is established using the following two-step procedure.

1. The estimated maximum groundwater contours shown on Drawing IIID-A-1 are utilized to estimate the uplift pressures shown for selected analysis points on Drawing IIID-B-8. Note that the underdrain system is limited to Aquifers B and C, and installed on the excavation sideslopes only. Installation of the underdrain system is not required in the floor of Cells 10-12.
2. For areas of sideslope that are higher in elevation than the maximum groundwater contours (Point Nos. 4 and 5) a top of groundwater elevation equal to 20 feet below the top of the excavation grade sideslope was assumed. This is a conservative assumption based on the downstream presence of a stormwater drainage channel which will drain the groundwater to below the top of the excavation sideslope elevation, as well as the drawdown characteristics of Aquifers B and C as discussed in Appendix IIID-C.
3. After Steps 1 and 2 are complete, the actual ballast required to offset the hydraulic uplift pressures on the bottom liner is calculated as shown on Sheet IIID-B-7.

The evaluation points on Drawing IIID-B-8 correspond to the areas where the dewatering system is designed to be installed and ballast is necessary for long-term liner stability. The temporary dewatering system is designed to control groundwater until enough ballast is in place. The design of this underdrain system is presented in Appendix IIID-C.

The actual thickness of ballast required must be calculated and submitted with the Soil Liner Evaluation Report (SLER). A summary of the procedure, which will be used to calculate ballast thickness, is discussed below. Example calculations are also presented on pages IIID-B-5 through IIID-B-7. The lined area may be divided into smaller subareas to determine the ballast requirements. The thickness of ballast required will be calculated using the following methodology:

A. The estimated groundwater potentiometric surface elevations will be determined from the updated (post-construction) water level data as illustrated in Appendix IIID-A.

At each evaluation point assigned to the liner construction area, determine the maximum hydrostatic uplift pressures acting at the geomembrane liner.

At each evaluation point, determine the uplift pressure acting on the geomembrane liner using the unit weight of water times the vertical distance from the geomembrane liner to the highest measured water table.

$$P_{H2O}=\gamma_{H2O}*H$$

where:

γ_{H2O} = unit weight of water (pcf)

H = vertical distance from the bottom of the liner (ft)

P_{H2O} = uplift pressure on the base of the liner (psf)

B. At each evaluation point, determine the resisting pressure for vertical uplift.

Determine the vertical resisting pressure at the evaluation points using the unit weight of the protective cover layer times the thickness of the protective cover layer.

$$\Sigma R_{i,v} = \Sigma(\gamma_i * T_{i,})$$

where:

$T_{i,v}$ = thickness of ballast component (protective cover) in vertical direction

γ_i = unit weight (pcf) of ballast component (protective cover)

$R_{i,v}$ = resisting pressure (psf) provided by each ballast component (protective cover) in vertical direction

C. Evaluate the factor of safety in the vertical direction at each evaluation point as a ratio of the total resisting pressure to uplift pressure.

The factor of safety (FS) against uplift due to the hydrostatic pressure acting at the geomembrane liner in the vertical direction is calculated as the resisting pressure determined in B divided by the uplift pressure determined in A.

$$FS_v = \sum R_{i,v} / P_{H2O}$$

If the factor of safety is less than 1.2, additional ballast will be necessary to offset the hydrostatic forces. See Section D for determining the thickness of additional ballast if necessary.

D. Determine the additional ballast necessary to offset hydrostatic pressures acting at the bottom of the liner in the vertical direction.

If the factor of safety calculated in Section C is less than 1.2, determine the thickness of additional ballast in the form of waste (T_{waste}) in the vertical direction to offset the hydrostatic uplift pressure at the evaluation point.

Use a factor of safety of 1.5 against uplift pressure when utilizing solid waste and protective cover.

Use a unit weight of 1200 lb/cy for in-place solid waste per Title 30 TAC §330.337(h)(2).

Calculate the minimum required waste column thickness that provides additional ballast to offset the hydrostatic uplift pressure with a factor of safety of 1.5 in the vertical direction.

$$R_{waste,v} = \gamma_{waste} * T_{waste,v}$$

where:

$T_{waste,\,v}$	=	waste thickness (ft) in vertical direction
γ_{waste}	=	unit weight of waste (pcf)
$R_{waste,\,v}$	=	resisting pressure of waste (psf) in vertical direction

$$P_{H2O} = \frac{\sum R_{i,v}}{1.5} + \frac{R_{waste,v}}{1.5}$$

Substituting appropriate values and solving for height of waste in the vertical direction:

$$T_{waste,v} = \frac{1.5}{\gamma_{waste}} * \left(P_{H2O} - \frac{\sum R_{i,v}}{1.5} \right)$$

If waste and protective cover do not provide enough ballast against uplift, final cover will be used for ballast with a factor of safety of 1.5.

Required: Provide example calculations to be used to estimate the amount of ballast required for the sidewall of the liner prior to decommissioning the dewatering system. Note that the calculations were performed assuming GCL installation, and do not take advantage of the ballasting provided by the clay bottom liner. this is a conservative assumption for the analysis.

Solution: Estimate the amount of ballast needed for the sidewall of the liner.

An example calculation using Evaluation Point No. 2 (Cell 12) is shown below. A summary of the calculation results for each evaluation point located on the liner side slopes is shown on Sheet IIID-B-6. Sheet IIID-B-7 shows the location of the evaluation points and the top of waste elevation required for ballast at each evaluation point.

Definition of terms/variables:

H = Maximum groundwater head at base of GCL, ft
P_{H20} = Maximum uplift pressure created by groundwater head, psf
$R_{pc,\,v}$ = Counteracting ballast pressure from GCL and protective cover - vertical, psf
$R_{pc,\,n}$ = Counteracting ballast pressure from GCL and protective cover - normal, psf
E_{H20} = Highest potentiometric surface elevation, ft-msl
E_{exc} = Elevation of excavation grade, ft-msl
$E_{waste,\,v}$ = Required top of waste elevation needed for ballast - vertical, ft-msl
$E_{waste,\,n}$ = Required top of waste elevation needed for ballast - normal, ft-msl
γ_{H20} = Unit weight of water, pcf
γ_{pc} = Unit weight of protective cover, pcf
γ_{waste} = Unit weight of waste, lb/cy (Assumed to be 1,200 lb/cy per 30 TAC Section 330.337(h)(2))
$T_{pc,\,v}$ = Thickness of clay liner and protective cover as ballast - vertical, ft
$T_{pc,\,n}$ = Thickness of clay liner and protective cover as ballast - normal, ft
$T_{waste,\,v}$ = Required waste thickness needed for ballast - vertical, ft
$T_{waste,\,n}$ = Required waste thickness needed for ballast - normal, ft
$E_{pc,v}$ = Elevation of top of protective cover - vertical, ft-msl
$E_{pc,n}$ = Elevation of top of protective cover - normal, ft-msl
$FS_{pc,\,v}$ = Calculated factor of safety with GCL and protective cover installed - vertical
$FS_{pc,\,n}$ = Calculated factor of safety with GCL and protective cover installed - normal
$E_{fc,\,v}$ = Design top of final cover elevation - vertical, ft-msl
$E_{fc,\,n}$ = Design top of final cover elevation - normal, ft-msl
$E_{top\ waste,\,v}$ = Design top of waste elevation - vertical, ft-msl
$E_{top\ waste,\,n}$ = Design top of waste elevation - normal, ft-msl
T_{fc} = Approximate thickness of final cover including intermediate cover, ft (note this thickness is assumed the same for the vertical and normal directions)

Example calculation using Evaluation Point No. 2:

Parameters:

E_{H20} =	570.9	ft-msl	γ_{pc} =	120	pcf
E_{exc} =	520.0	ft-msl	γ_{waste} =	1,200	lb/cy
γ_{H20} =	62.4	pcf	$E_{fc,\,v}$ =	722.7	ft-msl
β = side slope angle =	18.43	degrees	$E_{fc,\,n}$ =	722.7	ft-msl
$\cos\beta$ =	0.9487		T_{fc} =	2	ft
$T_{pc,\,v}$ =	2.2	ft ($T_{pc,v}/\cos\beta$)			
$T_{pc,\,n}$ =	2.0	ft			

Calculate the maximum groundwater head at the base of the GCL.

$H = E_{H20} - E_{liner}$
$H =$ 50.9 ft

Calculate the maximum hydrostatic uplift pressure created by the groundwater head.

$P_{H2O} = (\gamma_{H2O} \times H)$
$P_{H2O} =$ 3,176 psf

Calculate the counteracting ballast pressure from the GCL/protective cover in the vertical and normal directions.

$R_{pc,\,v} = (\gamma_{pc} \times T_{pc,\,v})$
$R_{pc,\,v} =$ 264 psf

$R_{pc,\,n} = (\gamma_{pc} \times T_{pc,\,n})$
$R_{pc,\,n} =$ 240 psf

Compare the uplift pressure to the ballast pressure by calculating the factors of safety in the vertical and normal direction with GCL/protective cover as ballast at the evaluation point.

$FS_{pc,\,v} = R_{pc,\,v}/P_{H2O} =$ 0.1

$FS_{pc,\,n} = R_{pc,\,n}/P_{H2O} =$ 0.1

The minimum required factor of safety for GCL/protective cover as ballast is 1.2. Since the factor of safety against uplift is less than 1.2 additional ballast (in the form of waste) will be necessary to counteract the hydrostatic uplift pressure acting at the top of geomembrane. If the factor of safety against uplift was 1.2 or greater, then no additional ballast would be necessary indicating that the protective cover provides enough ballast to counteract the hydrostatic uplift pressure acting at the top of GCL. When solid waste is necessary as ballast, a factor of safety of 1.5 is used for protective cover and solid waste.

Determine amount of additional ballast in the form of waste necessary to offset the hydrostatic pressure acting at the top of GCL (geomembrane) in the vertical and normal direction. Use a factor of safety of 1.5 for protective cover and solid waste.

$T_{waste,\,v} = [(1.5 \times P_{H2O}) - R_{pc,\,v}]/\gamma_{waste}$
$T_{waste,\,v} =$ 101.3 ft

$E_{waste,\,v} = E_{exc} + T_{pc,\,v} + T_{waste,\,v}$
$E_{waste,\,v} =$ 623.5 ft-msl

$T_{waste,\,n} = [(1.5 \times P_{H2O}) - R_{pc,\,n}]/\gamma_{waste}$
$T_{waste,\,n} =$ 101.8 ft

$E_{waste,\,n} = E_{exc} + T_{pc,\,n} + T_{waste,\,n}$
$E_{waste,\,n} =$ 623.8 ft-msl

Check to verify that the required top of waste elevation is less than the design top of waste elevation in the vertical and normal direction.

$E_{top\ waste,\,v} = E_{fc,\,v} - T_{fc}$
$E_{top\ waste,\,v} =$ 720.7 ft-msl

$E_{top\ waste,\,v} > E_{waste,\,v}$
$720.7 > 623.5$

$E_{top\ waste,\,n} = E_{fc,\,n} - T_{fc}$
$E_{top\ waste,\,n} =$ 720.7 ft-msl

$E_{top\ waste,\,n} > E_{waste,\,n}$
$720.7 > 623.8$

The required top of waste elevation needed as ballast is less than the design top of waste elevation in the vertical and normal directions. Therefore, the design top of waste elevation allows for the required top of waste elevation needed for ballast in the vertical and normal directions. If the top of waste elevation did not provide enough ballast, then the final cover is used to provide additional ballast against uplift using a factor of safety of 1.5.

ROYAL OAKS LANDFILL
APPENDIX IIID-B
EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDESLOPE LINER UPLIFT

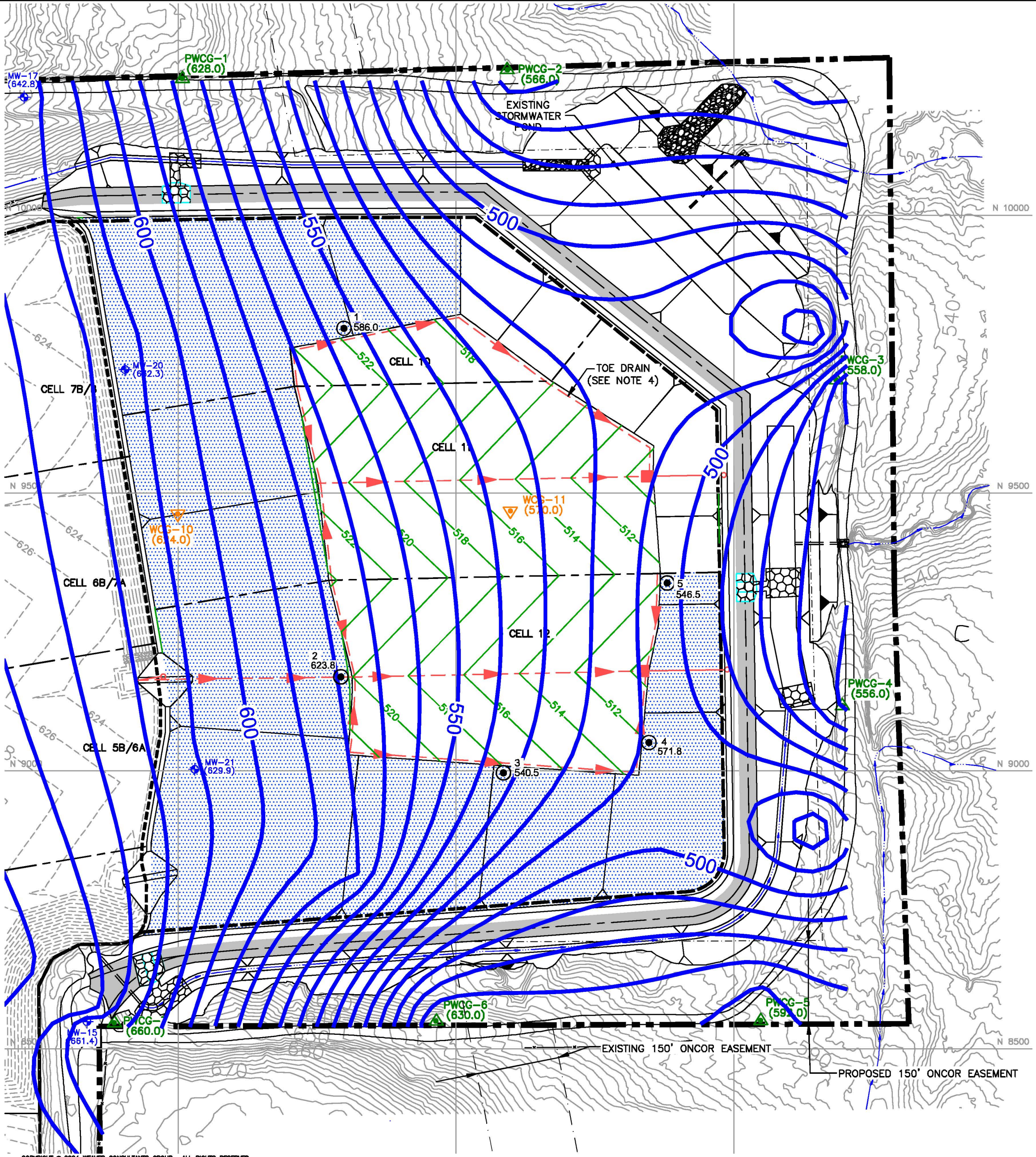
Unit Weight of Water =	62.4	pcf		Thickness of Protective Cover - Vertical =	2.2	ft
Unit Weight Protective Cover =	120	pcf		Thickness of Protective Cover - Normal =	2.0	ft
Unit Weight of Waste =	1200	pcy		Thickness of Final Cover/Int Cover =	2.0	ft
Unit Weight of Final Cover =	120	pcf				

Evaluation Point	Highest Potentiometric Surface Elevation E_{H2O} (ft-msl)	Excavation Grade E_{liner} (ft-msl)	Maximum Groundwater Head at Base of GCL[2] (Toe of Sideslope) H (ft)	Maximum Uplift Pressure Created by Groundwater Head P_{H2O} (psf)	Elevation of Top of GCL/ Protective Cover - Vertical $E_{pc,v}$ (ft-msl)	Elevation of Top of Protective Cover - Normal $E_{pc,n}$ (ft-msl)	Counteracting Ballast Pressure from GCL/ Protective Cover - Vertical $R_{pc,v}$ (psf)	Counteracting Ballast Pressure from GCL/ Protective Cover - Normal $R_{pc,n}$ (psf)	Calculated Factor of Safety with GCL/ Protective Cover Installed - Vertical	Calculated Factor of Safety with GCL/ Protective Cover Installed - Normal	Factor of Safety - Vertical > 1.2?	Factor of Safety - Normal > 1.2?	Required Waste Thickness Needed for Ballast - Vertical $T_{wb,v}$ (ft)[1]	Required Waste Thickness Needed for Ballast - Normal $T_{wb,n}$ (ft)[1]	Required Top of Waste Elevation Needed for Ballast - Vertical $E_{wb,v}$ (ft-msl)	**Required Top of Waste Elevation Needed for Ballast - Normal $E_{wb,n}$ (ft-msl)**	Design Top of Waste Elevation - Vertical $E_{top\ waste,v}$ (ft-msl)	Design Top of Waste Elevation - Normal $E_{top\ waste,n}$ (ft-msl)	Required Waste Needed for Ballast Elevation < Design Top of Waste Elevation - Vertical?	Required Waste Needed for Ballast Elevation < Design Top of Waste Elevation - Normal?	Counteracting Ballast Pressure from Protective Cover, Waste, and Final Cover - Vertical $Rf_{c,v}$ (psf)	Counteracting Ballast Pressure from Protective Cover, Waste, and Final Cover - Normal $Rf_{c,n}$ (psf)	Calculated Factor of Safety with Final Cover Installed - Vertical	Calculated Factor of Safety with Final Cover Installed - Normal	Factor of Safety - Vertical > 1.5?	Factor of Safety - Normal > 1.5?
1	554.0	522.0	32.0	1,997	524.2	524.0	264	240	0.1	0.1	NO	NO	61.5	62.0	585.7	**586.0**	646.9	647.1	YES	YES	5,957	5,975	3.0	3.0	YES	YES
2	570.9	520.0	50.9	3,176	522.2	522.0	264	240	0.1	0.1	NO	NO	101.3	101.8	623.5	**623.8**	720.7	720.9	YES	YES	9,326	9,344	2.9	2.9	YES	YES
3	530.3	518.0	12.3	768	520.2	520.0	264	240	0.3	0.3	NO	NO	20.0	20.5	540.2	**540.5**	663.9	664.1	YES	YES	6,891	6,884	9.0	9.0	YES	YES
4[3]	542.0	512.0	30.0	1,872	514.2	514.0	264	240	0.1	0.1	NO	NO	57.2	57.8	571.4	**571.8**	600.0	600.2	YES	YES	4,317	4,311	2.3	2.3	YES	YES
5[3]	530.0	512.0	18.0	1,123	514.2	514.0	264	240	0.2	0.2	NO	NO	32.0	32.5	546.2	**546.5**	577.3	577.5	YES	YES	3,308	3,302	2.9	2.9	YES	YES

[1] Refer to Sheet IIID-B-7 for the highest measured groundwater contours.

[2] Analysis conservatively performed assuming groundwater uplift acting on bottom of GCL layer. Ballasting of clay liner alternative not considered in calculations. Analysis conservatively assumes that peak uplift occuring in geocomposite layer at toe of sideslope.

[3] Highest measured groundwater elevation for analysis points 4 and 5 adjusted to height of 20 feet (vertically) below the top of sideslope excavation grades (approximate) to account for groundwater drainage into or from adjacent stormwater management channel (at elevation below top of slope elevation) into underdrain system, and drawdown (as demonstrated in Appendix IIID-C) assumed for the medium to high permeability Aquifers B and C waterbearing formations.



N

0 100 200
SCALE IN FEET

LEGEND

LANDFILL PERMIT BOUNDARY

LIMIT OF WASTE

610 EXISTING CONTOUR

520 EXCAVATION GRADES

EXISTING EASEMENT

PROPOSED EASEMENT

630 PROPOSED TOP OF LINER CONTOUR

MW-25 (663.2) EXISTING GROUNDWATER MONITORING WELL LOCATION

PWCG-1 (628.0) PROPOSED EXPANSION BOREHOLE & PIEZOMETER LOCATION

WCG-10 (624.0) PROPOSED EXPANSION BOREHOLE LOCATION

LIMITS OF SIDESLOPE UNDERDRAIN

UNDERDRAIN SUMP

UNDERDRAIN PIPING

650 HIGHEST MEASURED GROUNDWATER POTENTIOMETRIC SURFACE IN FT-MSL

1 BALLAST EVALUATION POINT

639.3 TOP OF WASTE REQUIRED FOR BALLAST

NOTES:

1. EXISTING CONTOURS AND PHOTO DEVELOPED BY COOPER AERIAL SURVEYS, CO. FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 20, 2021.
2. VERTICAL LIMITS OF UNDERDRAIN ESTABLISHED BASED ON HIGHEST MEASURED GROUNDWATER LEVEL. TOP OF SIDESLOPE LOCATION MAY VARY BASED ON FIELD OBSERVATIONS BY POR OF SEEPAGE FROM SIDESLOPE EXPOSED DURING EXCAVATIONS. LIMITS OF TOE DRAIN MAY ALSO BE EXTENDED INTO THE NORTHEAST CORNER OF CELL 12 IN THE EVENT SEEPAGE IS OBSERVED IN THE SIDESLOPE EXCAVATIONS.
3. IN AREAS THAT TOP OF SIDESLOPE IS BELOW ELEV. 585 FT-MSL, THE UNDERDRAIN WILL BE INSTALLED TO THE TOP OF SLOPE. UNDERDRAIN GEOCOMPOSITE WILL NOT BE INSTALLED INTO THE ANCHOR TRENCH AT THE TOP OF SIDESLOPE BUT INSTEAD WILL TERMINATE A MINIMUM 1 FOOT (3 FEET ALONG SIDESLOPE) BELOW TOP OF SIDESLOPE TO PREVENT MIGRATION OF LANDFILL GAS INTO ANCHOR TRENCH.
4. GEOCOMPOSITE UNDERDRAIN WILL BE INSTALLED CONSISTENT WITH THE REQUIREMENTS OF APPENDIX IIID-LINER QUALITY CONTROL PLAN.
5. SIDESLOPE TOE DRAIN IN AREAS WITHOUT GEOCOMPOSITE WILL NOT BE PERFORATED.

05/20/2024

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CAD: IIID-B-8-BALLAST POINTS.DWG

DRAWN BY: RAA
DESIGN BY: DEP
REVIEWED BY: DEP/BY

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS		
NO.	DATE	DESCRIPTION

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MAJOR PERMIT AMENDMENT
BALLAST EVALUATION PLAN

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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SHEET IIID-B-8

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APPENDIX IIID-C

TEMPORARY DEWATERING SYSTEM DESIGN

Includes pages IIID-C-1 through IIID-C-41

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

TEMPORARY DEWATERING SYSTEM DESIGN

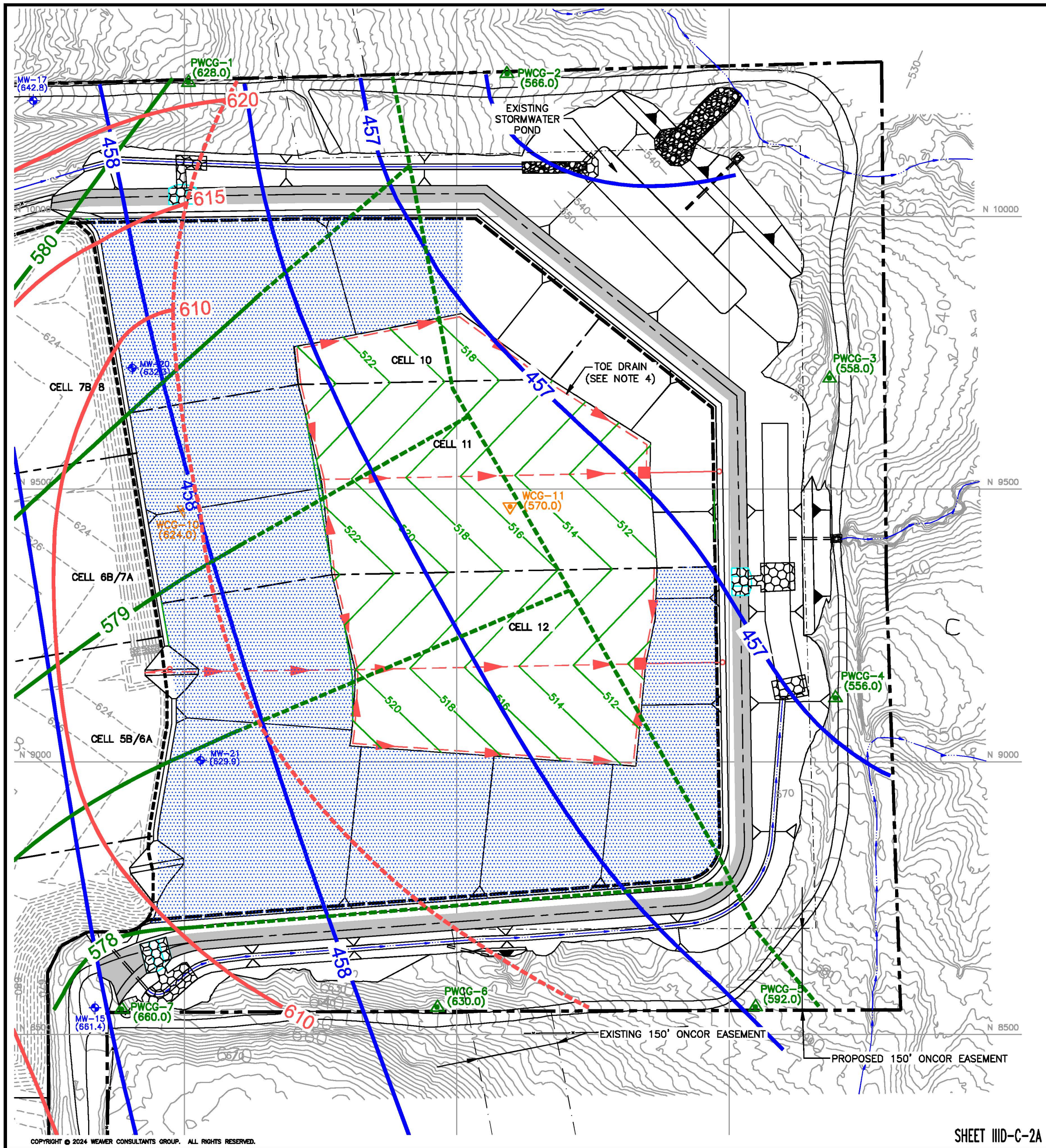
The site will have a temporary underdrain dewatering system installed for the undeveloped areas, specifically including Cells 10, 11 and 12 as shown on Figure 1. As described in the attached demonstration, the temporary underdrain installation will be limited to the future cell sideslopes as shown on Figure 1. The underdrain system has been designed to collect groundwater from Aquifers B and C, as described in detail in Appendix IIIG – Geology Report. As discussed in Appendix IIIG, Aquifer A is above the affected cells, or is cut off by previous landfill construction, and Aquifer D is at a depth well below the excavation grades of the remaining Cells 10-12. Based on this information, installation of temporary underdrains in the cell bottom was deemed unnecessary.

The dewatering system will be comprised of a double-sided geocomposite groundwater collection layer, collection trenches and collection sumps (in cells 11 and 12) which will intercept and divert waters potentially contacting the bottom liner system. Groundwater seepage will drain into the geocomposite and will then discharge into the drainage trenches and perforated 4-inch-diameter high density polyethylene (HDPE) piping installed at the toe of the excavated sideslope, then drain within the trenches and piping to the respective collection sumps. Water from the sumps will be pumped to the surface by submersible pumps installed in 18-inch-diameter sideslope risers located at each sump. A site plan of the underdrain system installed into Cells 10-12 is presented as Figure 1. Details of the underdrain dewatering system are presented in Appendix IIIA.

Water collected in the sumps and removed by submersible pump will drain into on-site stormwater management systems and then be discharged from the site consistent with the TPDES Stormwater Permit for the landfill. The pumps will be activated upon installation of the dewatering systems and will remain operational until the BER is approved. The pumps will be operated automatically by pressure transducers. Control levels for the automatic pump will be set to maintain sump liquid levels below the top of the sump.

The temporary dewatering system will remain operational until enough ballast is placed in the form of protective cover and solid waste over the impacted area. Once sufficient ballast is in place and with the written approval of TCEQ, the dewatering system will be decommissioned. A plan of the required waste ballast depths (based on groundwater surface captures) across the future lined areas is provided as Figure 2.

A different hydrostatic pressure relief system may be used at the site if it is designed using the same methodology as the design included in Appendix IIID-C (e.g., relieve of potential hydrostatic uplift pressure that may develop on the geomembrane liner) and approved by TCEQ through a permit modification. If during future cell design, the conditions are such that a different system (e.g., collector trenches, diversion channels adjacent to the sector, removal of the underdrain system, or a combination of options) is considered more efficient, the system design will be submitted to TCEQ for approval as a permit modification to the LQCP prior to implementation.



SHEET IIID-C-2A

0 100 200
SCALE IN FEET

LEGEND

- LANDFILL PERMIT BOUNDARY
- LIMIT OF WASTE
- EXISTING CONTOUR
- EXCAVATION GRADES
- PROPOSED 100' ONCOR ROW
- PROPOSED TOP OF LINER CONTOUR
- MW-25 (663.2) EXISTING GROUNDWATER MONITORING WELL LOCATION
- PWCG-1 (628.0) PROPOSED EXPANSION BOREHOLE & PIEZOMETER LOCATION
- WCG-10 (624.0) PROPOSED EXPANSION BOREHOLE LOCATION
- LIMITS OF SIDESLOPE UNDERDRAIN
- UNDERDRAIN SUMP
- UNDERDRAIN PIPING
- 458 AQUIFER D GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL
- 579 AQUIFER C GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL
- 620 AQUIFER B GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL
- AQUIFER C APPROXIMATE OUTCROP BOUNDARY
- AQUIFER B APPROXIMATE OUTCROP BOUNDARY

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NOTES:

1. EXISTING CONTOURS AND PHOTO DEVELOPED BY COOPER AERIAL SURVEYS, CO. FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 20, 2022.
2. VERTICAL LIMITS OF UNDERDRAIN ESTABLISHED BASED ON HIGHEST MEASURED GROUNDWATER LEVEL. TOP OF SIDESLOPE LOCATION MAY VARY BASED ON FIELD OBSERVATIONS BY POR OF SEEPAGE FROM SIDESLOPE EXPOSED DURING EXCAVATIONS. LIMITS OF TOE DRAIN MAY ALSO BE EXTENDED INTO THE NORTHEAST CORNER OF CELL 12 IN THE EVENT SEEPAGE IS OBSERVED IN THE SIDESLOPE EXCAVATIONS.
3. UNDERDRAIN GEOCOMPOSITE WILL NOT BE INSTALLED INTO THE ANCHOR TRENCH AT THE TOP OF SIDESLOPE BUT INSTEAD WILL TERMINATE A MINIMUM 1 FOOT (3 FEET ALONG SIDESLOPE) BELOW TOP OF SIDESLOPE TO PREVENT MIGRATION OF LANDFILL GAS INTO ANCHOR TRENCH.
4. GEOCOMPOSITE UNDERDRAIN WILL BE INSTALLED CONSISTENT WITH THE REQUIREMENTS OF APPENDIX IIID–LINER QUALITY CONTROL PLAN.
5. SIDESLOPE TOE DRAIN IN AREAS WITHOUT GEOCOMPOSITE WILL NOT BE PERFORATED.

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FILE: 0120–76–11	DESIGN BY: DEP
CAD: FIG 1–UNDERDRAIN PLAN.DWG	REVIEWED BY: DEP/JP

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REVISIONS

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MAJOR PERMIT AMENDMENT UNDERDRAIN PLAN

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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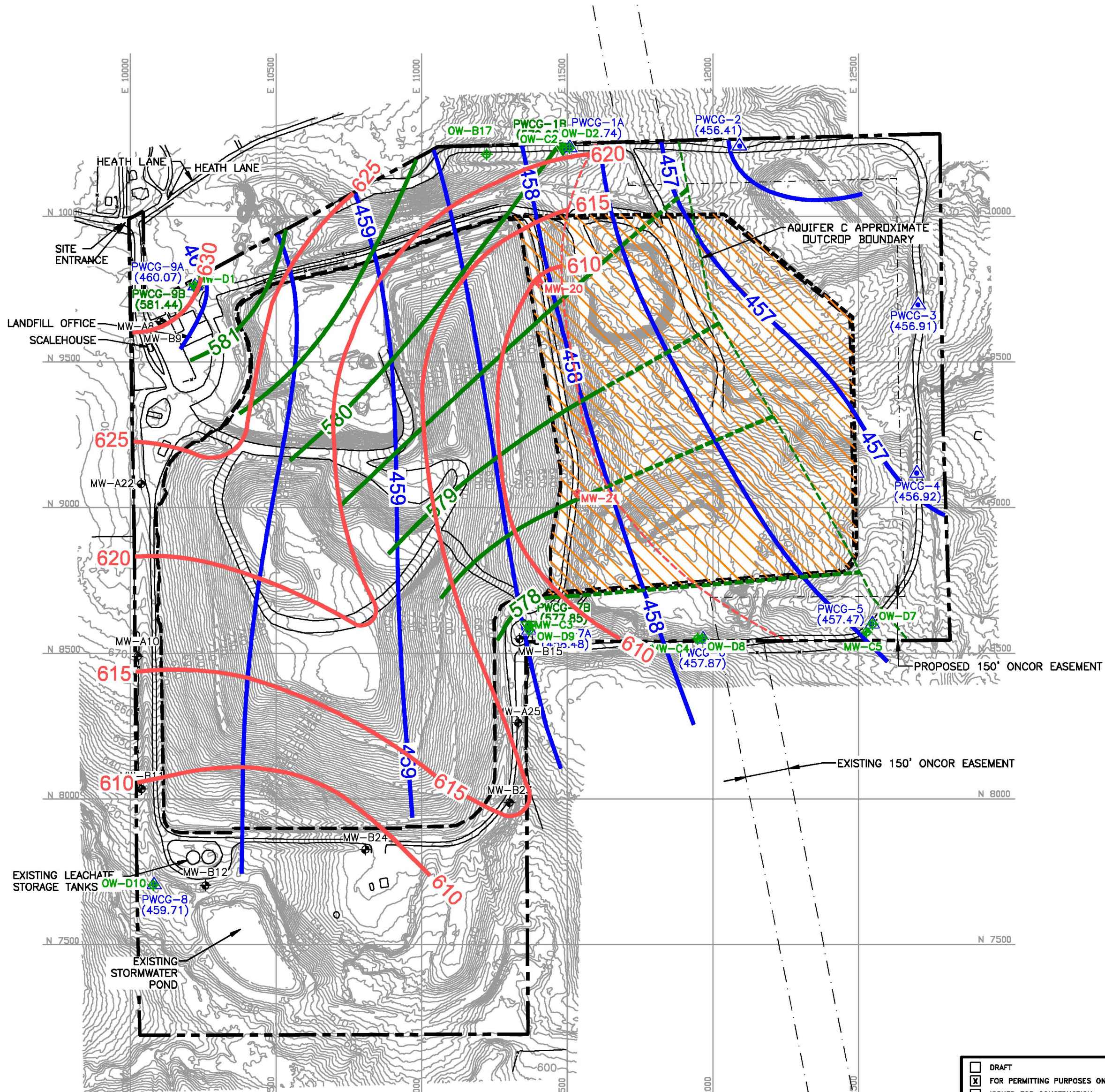
FIGURE 1

O:\0120\76\EXPANSION 2023\PART III\IIID\FIG 1–UNDERDRAIN PLAN.dwg, byoung, 1:2



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0 200 400

SCALE IN FEET

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DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

LEGEND

PERMIT BOUNDARY

PERMITTED LIMITS OF WASTE

PROPOSED LIMITS OF WASTE

EXISTING EASEMENT

PROPOSED EASEMENT

630 EXISTING CONTOUR

N 9500 STATE PLANE COORDINATE

PROPOSED LATERAL EXPANSION AREA

PWCG-1A (457.74) 2023 AQUIFER D EXPANSION PIEZOMETER WITH GROUNDWATER ELEVATION POSTED IN FT-MSL

PWCG-1B (579.98) 2023 AQUIFER C EXPANSION PIEZOMETER WITH GROUNDWATER ELEVATION POSTED IN FT-MSL

456 AQUIFER D GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL

581 AQUIFER C GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL

620 AQUIFER B GROUNDWATER POTENTIOMETRIC SURFACE CONTOUR IN FT-MSL

AQUIFER C APPROXIMATE OUTCROP BOUNDARY

AQUIFER B APPROXIMATE OUTCROP BOUNDARY

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY COOPER AERIAL SURVEYS, CO. FROM AERIAL PHOTOGRAPHY FLOWN ON 11-10-2022. THE GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NORTH CENTRAL ZONE NAD 1983.
2. PIEZOMETER LOCATION COORDINATES OBTAINED FROM AUGUST 2023 AS-BUILT SURVEY BY WEAVER CONSULTANTS GROUP.
3. GROUNDWATER ELEVATIONS GAUGED BY WCG IN SEPTEMBER 2023 AND POSTED IN FT-MSL BY EACH LOCATION.

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CAD: FIG 2-GW CONTOUR MAP.DWG

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DESIGN BY: DEP
REVIEWED BY: DEP/JP

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REVISIONS		
NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
UNDERDRAIN SYSTEM LAYOUT
GROUNDWATER CONTOUR MAP

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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FIGURE 2

SHEET IIID-C-2B

O:\0120\76\EXPANSION 2023\PART III\IIID\FIG 2-GROUNDWATER CONTOUR MAP.dwg, byoung, 1:2

TEMPORARY DEWATERING SYSTEM DESIGN

Required

The purpose of these calculations is to demonstrate the adequacy of the cell sideslope temporary underdrain dewatering system proposed for Cells 10 thru 12.

The underdrain system is designed to provide hydrostatic pressure relief below the bottom liner system for the areas excavated below the groundwater tables (Aquifers B and C) as shown on Figures 1 and 2.

Details of the underdrain system are presented in Appendix IIIA of this application.

Assumptions - Sideslopes

For the 3H:1V cell sideslopes the calculations were performed assuming the sideslope excavation shown on Figure 1 will intersect both Aquifers B and C, as described in Appendix G - Geology Report. These calculations assume that Aquifer A is cut off from the expansion area, and that Aquifer D is well below the excavation grades for the expansion area cells (as presented in Appendix IIIG). An 11-foot-thick groundwater table was assumed for Aquifer B, and a 22-foot-thick groundwater table was assumed for Aquifer C. Note that actual measured groundwater tables in borings installed adjacent to the future landfill sideslopes indicate water tables generally less than assumed for this anlaysis. Note also that the northeast corner of Cells 10 thru 12 was assumed to be downgradient for both Aquifers B and C, and therefore the underdrain system does not extend into this area of the cells.

Testing of the Aquifers B and C sands demonstrates an average horizontal permeability of approximately $2.07E^{-4}$ cm/sec and $5.95E^{-4}$ cm/sec, respectively. This value was used in estimating the flow of groundwater from each aquifer into the underdrain system. Due to the relatively high permeability of the sands within the Aquifers B and C strata it is reasonably assumed that the water table will draw down once the slope is excavated and aquifers are exposed to gravity drainage during excavation. Drawdown curves were approximated for both Aquifer B and C (shown on Sheets IIID-C-7A and IIID-C-7B) as representative of the gravity drainage of waters from the aquifers into the underdrain geocomposite. The values calculated on Sheets IIID-C-7A and IIID-C-7B were used for the geocomposite, piping and sump demonstrations presented in these calculations.

The overburden pressure causing compression of the geocomposite layer for the slideslope analysis was limited to approximaely 1.5 times the hydraulic uplift from the bottom of the cell (approximate elevation 520 ft-msl along the western sideslope toe in Cell 11) to the top of the highest measured groundwater contour above the western sideslope toe (approximate elevation 580 ft-msl from Figure IIID-A-1) for a required overburden pressure (from waste ballast) of approximately 5,600 psf. Additional compression of the geocomposite resulting from overburden pressure greater than the required ballasting pressure was not considered for the demonstration as the underdrain system will be abandoned after demonstration of ballasting for Cells 10 thru 12.

Method

1. Estimate the hydraulic conductivity of the foundation soils based on strata identified in the Appendix IIIG - Geology Report.
2. Develop flow nets that demonstrate the combined rate of flow of groundwater (from Aquifers B and C) into the geocomposite drainage layer.
3. Determine the flow capacity of the geocomposite drainage layer.
4. Compare geocomposite flow capacity with inflow to determine suitability of selected geocomposite.
5. Estimate the flow into the dewatering pipe installed at the toe of the sideslope.
6. Demonstrate the flow capacity of the dewatering pipe is sufficient.
7. Determine required pipe perforation based on characteristics of the surrounding drainage

media.

8. Evaluate the storage capacity and pump cycling for the sump design.

<u>References</u>

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems,* Pollution Technology Review No. 138, Noyles Data Corporation, 1986.
2. Texas Natural Resource Conservation Commission, *Leachate Collection System Handbook* , 30 TAC 330.201, 1993.
3. Koerner, R.M., *Designing with Geosynthetics* , second edition, Prentice Hall, Inc., 1990.
4. GSE Drainage Design Manual, May 2004.
5. *Dewatering and Groundwater Control,* TM5-818-5, November 1983.
6. Phillips 66 Driscopipe, System Design, 1991.
7. Acar, Yalcin B.& Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics, Volume 2, American Society of Civil Engineers,* 1995.
8. Gray, Donald H., Koerner, Robert M., Qian, Xuede, <u>Geotechnical Aspects of Landfill Design and Construction</u>, 2002.
9. Geosynthetic Institute, GRI Standard GC-8, 2001.
10. Cedergren, Harry R, *Seepage, Drainage, and Flow Nets* , 2nd Ed., 1977.

Solution

1. Estimate the flow into the geocomposite drainage layer - Cells 10-12

In order to develop an estimate of groundwater flow into the underdrain geocomposite drainage layer, flow nets representing the Aquifers B and C water-bearing formations were developed (see Sheet IIID-C-7A and IIID-C-7B). The flow nets were developed based on an assumed 3H:1V cut slope (cell sideslope), aquifer thicknesses of 11 feet and 22 feet for Aquifers B and C, respectively. Hydraulic conductivity values of 2.07E^{-4} and 5.95E^{-4} cm/sec were assumed for Aquifers B and C, respectively. Flow estimates shown on Sheets IIID-C-7A and IIID-C-7B estimate the flow per linear foot of slope (assumed as measured at the toe drain). Refer to the flownet figures for calculations.

$Q_{\text{Aquifer B, Sideslope}}$ = 7.47E-06 cf/sec-ft of sideslope (as measured at the sideslope toe)
$Q_{\text{Aquifer C, Sideslope}}$ = 4.29E-05 cf/sec-ft of sideslope (as measured at the sideslope toe)

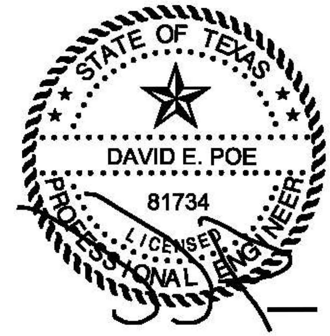
$Q_{\text{max, sideslope}}$ = 5.04E-05 cf/sec-ft of sideslope (as measured at the sideslope toe)

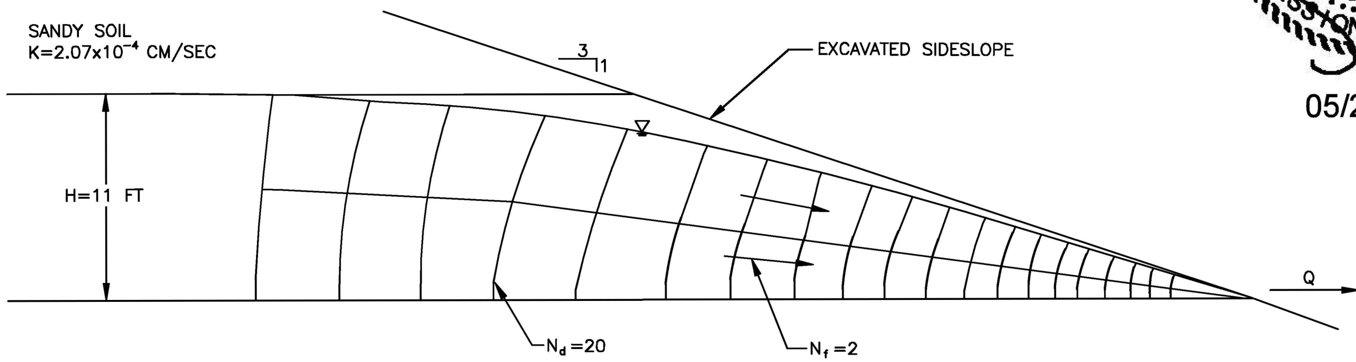
Geocomposite design will be evaluated incorporating the combined Aquifers B and C flow above.

2. Determine the flow capacity of the geocomposite drainage layer - Landfill Sideslope (Cells 10 thru 12)

Assume the geocomposite leachate collection layer will undergo compression due to the weight of liner, protective cover, and waste.

Unloaded Geocomposite Thickness (200 mil) =	0.20	in
Unit Weight of Soil =	120	pcf





N.T.S.

AQUIFER B SIDESLOPE DRAINAGE CALCULATIONS
K = 2.07E10^{-4} CM/SEC (FROM IIIG - GEOLOGY)
H = 11 FT (ESTIMATED FROM CROSS-SECTIONS, TYPICAL)
Nf = NO. FLOW LINES = 2 (FROM DIAGRAM)
Nd = NO. OF HYDRAULIC PRESSURE DROPS = 20 (FROM DIAGRAM)
Nf / Nd = HYDRAULIC GRADIENT = 0.10 FT/FT
Q = (K) (H) (Nf/Nd) =
(2.07E10^{-4} CM/SEC) (11 FT) (2/20) (0.0328 FT/CM)
= 7.47E10^{-6} CF/SEC-FT (OF TOE DRAIN)
CALCULATED FLOW RATE FROM AQUIFER B
INTO UNDERDRAIN = 7.47E^{-6} CF/ SEC-FT OF TOE DRAIN

NOTES:

1. GRADING VALUE OF 0.10 FT/FT IS CONSERVATIVE. GRADIENT REPORTED IN APPENDIX IIIG–GEOLOGY AS 0.013 FT/FT.

CELLS 10–12 SIDESLOPE
AQUIFER B FLOWNET

ROYAL OAKS LANDFILL
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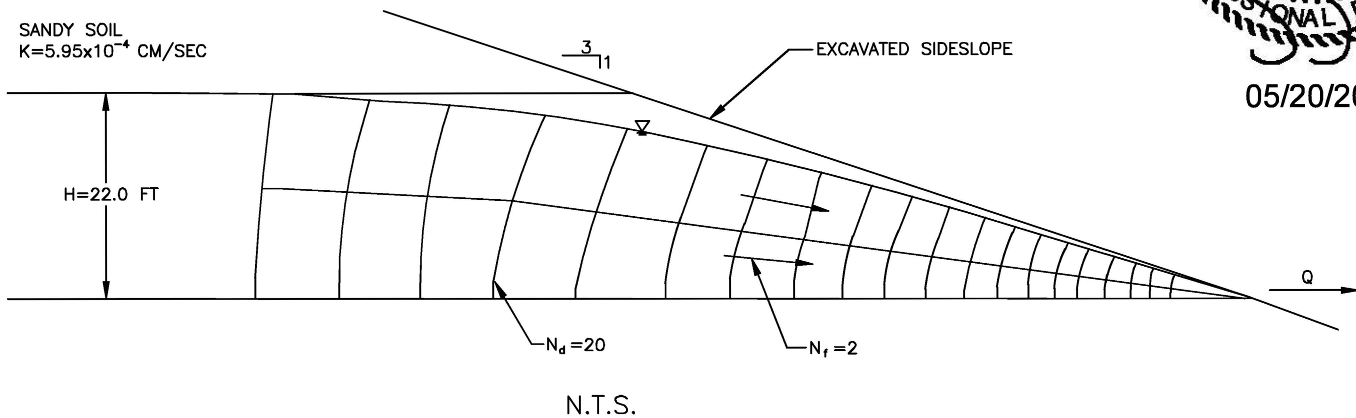

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05/20/2024

AQUIFER C SIDESLOPE DRAINAGE CALCULATIONS
K = 5.95E10^{-4} CM/SEC (FROM IIIG - GEOLOGY)
H = 22 FT (ESTIMATED FROM CROSS-SECTIONS, TYPICAL)
Nf = NO. FLOW LINES = 2 (FROM DIAGRAM)
Nd = NO. OF HYDRAULIC PRESSURE DROPS = 20 (FROM DIAGRAM)
Nf / Nd = HYDRAULIC GRADIENT = 0.10 FT/FT
Q = (K) (H) (Nf/Nd) =
(5.95E10^{-4} CM/SEC) (22 FT) (2/20) (0.0328 FT/CM)
= 4.29E10^{-5} CF/SEC-FT (OF TOE DRAIN)
CALCULATED FLOW RATE FROM AQUIFER C
INTO UNDERDRAIN = 4.29E^{-5} CF/ SEC-FT OF TOE DRAIN

NOTES:

1. GRADING VALUE OF 0.10 FT/FT IS CONSERVATIVE. GRADIENT REPORTED IN APPENDIX IIIG–GEOLOGY AS 0.002 FT/FT.

CELLS 10–12 SIDESLOPE
AQUIFER C FLOWNET

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

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REVIEWED BY: DEP	CAD: IIID-C-7B.DWG	SHEET IIID-C-7B

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Table 1 - Geocomposite Thickness

Fill Condition	d_W[1] (ft)	d_S[2] (ft)	γ[3] (pcf)	P[4] (psf)	t[5] (in)	t[5] (m)
Grading, Liner and LCS Layers Installed	0	4	120	480	0.199	0.0051
Waste Thickness - Sideslope (see Note 4 below)	115.2	4	44	5,600	0.170	0.0043

[1] d_W is the depth of waste and daily cover soil above the geocomposite underdrain collection layer. Depth of waste estimated as 1.5 times the maximum depth from top of Highest Measured Groundwater Contour underdrain (west sideslope toe of Cell 11).

[2] d_S is the depth of soil (protective cover, intermediate cover) above the geocomposite underdrain collection layer.

[3] The unit weight of waste/soil is selected at the midpoint of the waste column thickness and based on the 1,200 pcf unit weight required during ballasting demonstration for discontinuation of pumping from the underdrain.

[4] P is the pressure on the geocomposite underdrain collection layer under the load calculated in Table 1, above.

[5] t is the thickness of the geocomposite underdrain collection layer after being subjected to compression based on the chart below adapted from Reference 7.

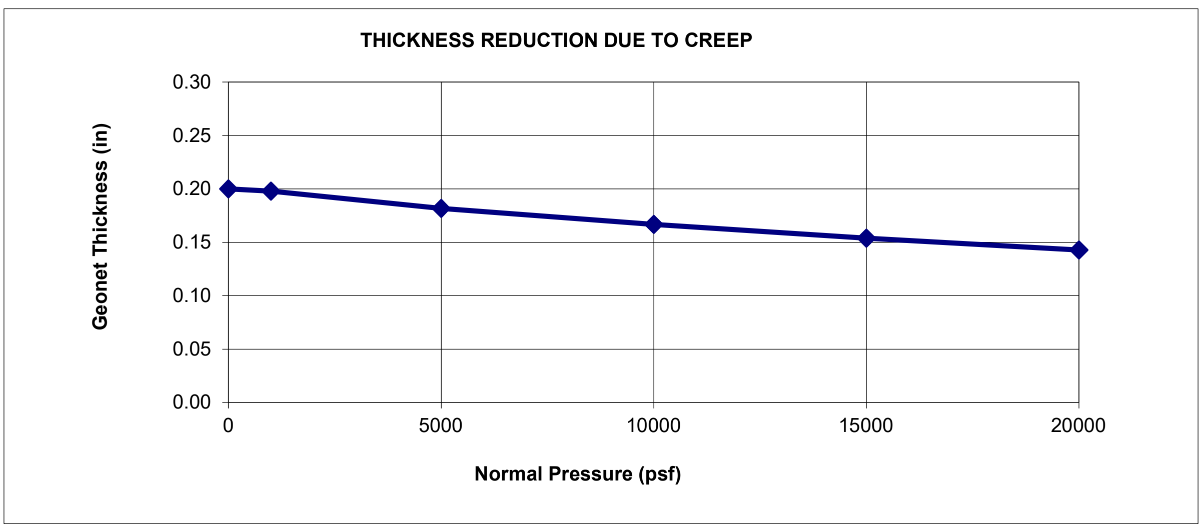


Table 2 - Reduction Factors and Factor of Safety (Sideslopes)

Reduction Factors[1]		Fill Condition: Liner Protective Cover Installed	Fill Condition: Maximum Waste Column in Place
RF_{IN}	Delayed Intrusion	1.0	1.2
RF_{CC}	Chemical Clogging	1.0	1.1
RF_{BC}	Biological Clogging	1.0	1.0
Total Reduction Factor[2]		1.00	1.32
Overall Factor of Safety to Account For Uncertainties		2.0	2.0
FS Factor[3]		2.00	2.64

[1] Values are obtained from References 3, 8, and 9.

[2] The Total Reduction Factors are a product of all the reduction factors for each fill condition.

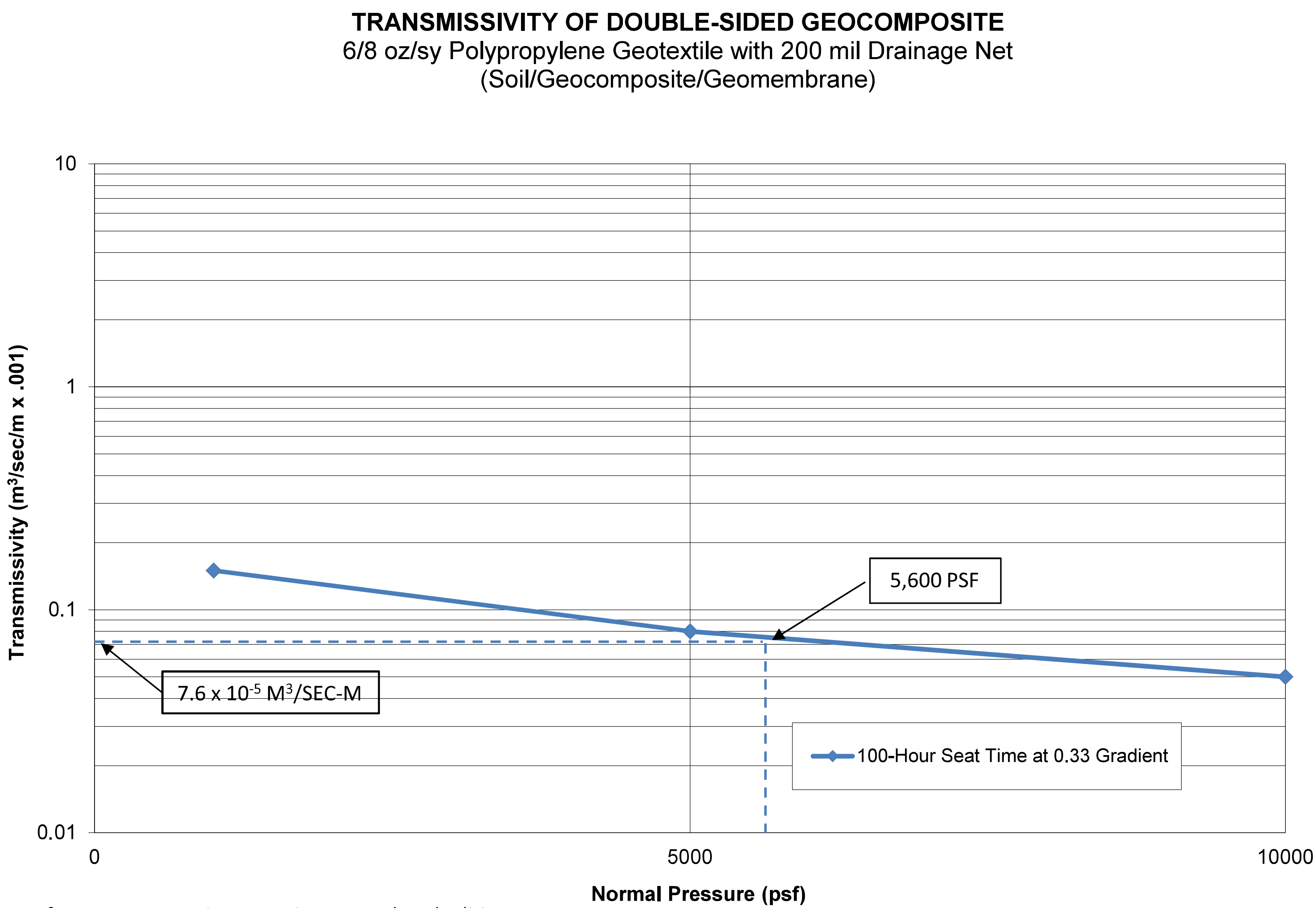
[3] The FS Factor is a product of the Total Reduction Factor and Overall Factor of Safety to Account For Uncertainties for each fill condition.

[4] Chemical and biological clogging are assumed neglible due to short time underdrain utilized prior to ballasting. Some minor chemical clogging may occur over time due to groundwater mineralization. Underdrain will not be exposed to biological leachate.

Manufacturer's Transmissivity Data

The required minimum transmissivity for the 200-mil-thick double-sided geocomposite with a 6 oz/sy geotextile is shown in table below. These values are developed based on engineering judgment and experience with similar geocomposite products at numerous MSW sites evaluated by WCG in the US.

Compute the design transmissivity (T) of the geocomposite.



Reference: GSE Drainage Design Manual, 3rd Edition, 2004

Table 3 - Design Transmissivity (Sideslopes)

Fill Condition	t^1 (in)	T^2 (m^2/s)	FS Factor[3]	$T_{DES}{}^4$ (m^2/s)	T_{DES} (cf/sec-ft)
Liner and Protective Cover Installed	0.199	1.50E-04	2.00	7.50E-05	8.07E-04
Maximum Waste Thickness as Ballast	0.170	7.60E-05	2.64	2.88E-05	3.10E-04

[1] t is the calculated geocomposite thickness from Table 1.
[2] T is the transmissivity values obtained from review of representative geocomposite products similar to proposed for project. Representative transmissivity values for 200-mil geocomposite shown on Sheet IIID-C-10.
[3] FS Factor is the product of the factors of safety from Table 2.
[4] T_{DES} is the design transmissivity value calculated using the following equation:

$$T_{DES} = T / (\text{FS Factor})$$

Design Flow Capacity

Unit Width of Geocomposite in dewatering: 1 ft

From Tables 3A and 3B above, the minimum design transmissivity of the geocomposite drainage layer is:

$Q_{design,\ sideslope}$ = **3.10E-04 cf/sec-ft**

$Q_{max,\ sideslope}$ = **5.04E-05 cf/sec-ft**

$Q_{design,sideslope} / Q_{max,sideslope}$ = **6.15**

The flow capacity of the 200 mil geocomposite ($Q_{design,\ sideslope}$) is greater than the estimated flow of groundwater into the geocomposite ($Q_{max,\ sideslope}$) by a factor exceeding 6.1. Therefore the design use of 200-mil geocomposite for the sideslope underdrain installation is acceptable.

3. Estimate the flow into the dewatering pipe - Sideslope (Cells 10 thru 12)

$Q_{\text{toe drain pipe,max}} = (Q_{\text{design, sideslope}}) \times (L)$

where: $(Q_{\text{design, sideslope}})$ = Maximum flov cf/sec-ft of sideslope (as measured at the sideslope toe)
L= Longest length of toe drain collecting groundwater (ft)

$(Q_{\text{design, sideslope}})$ = 5.04E-05 cf/sec-ft of sideslope (as measured at the sideslope toe)
L= 700 ft (along southern and eastern edge of Cell 12)

$Q_{\text{toe drain pipe,max}}$ = 3.53E-02 cf/sec

4. Determine the flow capacity of the dewatering pipe (Cells 10 thru 12 Sideslope Toe Drain)

$$Q_{full} = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Where: A = Cross-sectional area of pipe, with d representing the inside diameter in feet
R = Hydraulic radius of pipe in feet under full flow conditions

Using a 4-inch SDR 17 pipe: ID = 3.97 in
= 0.331 ft

$A = \frac{(\pi \times d^2)}{4}$ A = 0.086 sq ft

R = d / 4 R = 0.083 ft

S = Design slope of pipe (1.8% approx.) S = 0.018 ft / ft
n = Manning's number n = 0.009 from Ref. 6

Q_{full} = 0.36 cfs
= 162 gpm

$Q_{\text{max, toe drain piping}}$ = 3.53E-02 cfs (from Step 3)
= 15.8 gpm

The flow capacity of the 4-inch-diameter pipe (162 gpm) is significantly larger than the maximum calculated flow from the geocomposite (15.8 gpm) into the toe underdrain dewatering pipe. Note also that these calculations do not account for the future dewatering of Aquifers B or C or absence of Aquifers B or C in the excavated sideslope, which may greatly reduce flow into the underdrain system.

ROYAL OAKS LANDFILL
APPENDIX IIID-C
TEMPORARY UNDERDRAIN DEWATERING SYSTEM
UNDERDRAIN SUMP DESIGN

REQUIRED: Size underdrain collection sumps and demonstrate capacity provides storage under peak conditions determined from groundwater inflow calculations.

METHOD: Use groundwater production rates from sideslope underdrain calculations and the estimated linear length of slope being drained for Cell 11 (approximately 1,150 lf) as representative of the future groundwater underdrain sump requirements for Cells 10 thru 12. Underdrain piping and sump details are provided on drawings in Appendix IIIA.

REFERENCES:

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems* , Pollution Technology Review No. 138, Noyles Data Corporation, 1986.
2. Phillips 66 Driscopipe, *System Design* , 1991.
3. Heisler, Sanford I., P.E., Wiley Engineer's Desk Reference, John Wiley & Sons, Inc., New York, 1998.

SOLUTION:

A. Average Groundwater Flow Rate into Sump

Determine the per acre flow rate for a typical leachate collection sump.

The following table presents an estimate of the flow into a sump based on the caculations presented in this appendix.

Calculations performed for Cell 11 is considered worst case for Cells 10 thru 12.

Condition	Underdrain Collection Area		Total Flow to Sump		
	Total Length (ft)	Underdrain Seepage (cf/sec-ft)[1]	cf/day	gpm	gpd
Cell 11 Toe Drains	1,150	5.04E-05	5007.7	26.0	37,458

[1]Underdrain seepage calculations presented on Sheet IIID-C-6.

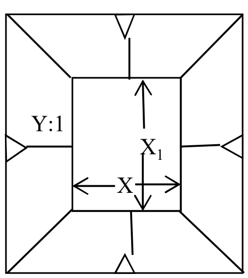
B. Storage Capacity of Sump

Total sump volume:

$$V_{TOT} = 1/3\left(A_1 + A_2 + \sqrt{(A_1 \cdot A_2)}\right)h \qquad \text{(Ref. 4, page 17)}$$

Where:

A_1 = Area of bottom of sump
A_2 = Area of top of sump
h = Depth of sump



Y = Slope of sump side walls
$A_1 = X_1 * X_2$
$A_2 = (X_1 + 2(h*Y))*(X_2 + 2(h*Y))$

ROYAL OAKS LANDFILL
APPENDIX IIID-C
TEMPORARY UNDERDRAIN DEWATERING SYSTEM
UNDERDRAIN SUMP DESIGN

X_1 (ft)	X_2 (ft)	Y (ft)	h (ft)	A_1 (ft^2)	A_2 (ft^2)	V_{TOT} (ft^3)
16	16	1	3	256	484	1,092

Assumed porosity of sump drainage stone (P) = 0.35

$\mathbf{V_{EFF} = V_{TOT} \times P}$

V_{TOT} (ft^3)	V_{EFF} (ft^3)	V_{EFF} (gal)
1,092	382	2,859

Compute the number of days storage provided for the following:

$$\text{STORAGE (Detention Time)} = \frac{V_{TOT}}{V_{\text{Daily Inflow}}}$$

$V_{\text{Daily Inflow}}$ (gpd)[1]	V_{EFF} (gal)	Storage (hours)
37,458	2,859	1.8

C. Estimated Rate of Underdrain Groundwater Removal

Submersible pump capacity = 50 gpm

Groundwater Production (gpd)	Pump Rate (gpm)	Average Pump Time (min/day)	Average Pump Time (hr/day)
37,457.9	50	749.2	12.5

Average pump time is less than 24 hours per day, therefore the design is acceptable. A pump with less capacity may also be used if it can be demonstrated (based on field records) that the actual underdrain groundwater flow rate is less than the design flow.

REQUIRED: Analyze structural stability of the 4-inch-diameter groundwater dewatering system pipe.

METHOD:

A. Determine the critical load and calculate stress under the following two conditions:

1. Construction loading
2. Overburden loading (conservatively calculated based on a waste unit weight of 72 pcf versus 44.4 used for ballasting demonstration)

B. Use the critical loading pressure to analyze pipe stability under the following three possible failure conditions:

1. Wall crushing
2. Wall buckling
3. Ring deflection

NOTE: The groundwater dewatering system details shown on Sheets IIID-C-27 and IIID-C-28 are for illustration purposes only to show parameters used in the following calculations. Groundwater dewatering system details can be found in Appendix IIIA.

REFERENCES:

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems* , Pollution Technology Review No. 138, Noyles Data Corporation, 1986.

2. Texas Natural Resource Conservation Commission, *Leachate Collection System Handbook* , 30 TAC 330.201, 1993.

3. Phillips 66 Driscopipe, *System Design* , 1991.

4. Landfill Design Series, *Leachate Gas Management Systems Design, Volume 5, Leachate Management and Storage,* Appendix A, 1993.

5. Caterpillar Tractor Company*, Caterpillar Performance Handbook,* Edition 27, October 1996.

6. Quian, Xuede, R.M. Koerner, D. H. Gray, "Geotechnical Aspects of Landfill Design and Construction." Prentice-Hall, Inc., New Jersey, 2002.

SOLUTION:

A. Determine the critical load and stress:

A.1. Maximum construction loading:

Assume: CAT 637E Series II scraper with an even load distribution

Loaded weight =	190,500	lb
Tire pressure =	80	psi
Number of tires =	4	

For a circular tire imprint:

$$F = \frac{\text{Loaded Weight}}{\text{Number of Tires}}$$

Where: F = Force exerted by one tire (lb)

F =	47,625	lb

Determine area of contact for circular tire imprint:

$$r = (F/\pi p)^{1/2}$$

Where:
r = Radius of contact (in)
F = Force exerted by one tire (lb)
p = Tire pressure (psi)

r =	13.8	in

Use Boussinesq's solution to find the stress at a point below a uniformly loaded circular area:

$$y = p\,(1 - ((r/z)^2+1)^{-3/2})$$

Where:
y = Change in vertical stress (psi)
p = Tire pressure (psi)
r = Radius of contact (in)
z = Protective cover thickness (in)

z =	24	in

y =	27.8	psi

Assume only one wheel load on pipe and add 50% for impact loading:

P_L = 1.5y

Where: P_L = Maximum live load (psi)

P_L =	41.7	psi

P_D = (zw)/1728

Where:
P_D = Maximum dead load (psi)
z = Protective cover thickness (in)
w = Unit weight of protective cover (pcf)

z = 24 in
w = 120 pcf

P_D =	1.67	psi

P_T = $P_L + P_D$

Where: P_T = Maximum construction load (psi)

P_T =	43.3	psi

A.2. Overburden loading (pre-ballast loading):

For maximum fill load on pipe:

2.0	ft protective cover @	120	pcf =	240	psf	
2.0	GCL & prot. cover @	120	pcf =	240	psf	
115.2	ft solid waste/soil @	72	pcf =	8,294	psf	Highest waste column thickness
			Σ =	8,774	psf	over a 4" LCS pipe.

P_T =	60.9	psi

Determine critical loading condition:

Construction loading:	P_T =	43.3	psi
Overburden loading:	P_T =	60.9	psi

Overburden loading is most critical to the structural stability of the pipe and will be used to determine the design pipe stress.

Determine design stress:

1. Adjust critical stress to account for loss of strength in the pipe due to perforations:

$$P_{DES1} = 12P_T / (12 - l_p)$$

Where:

l_p = Cumulative length of perforations per foot of pipe
P_T = Critical pipe stress (psi)
P_{DES1} = Pipe stress adjusted for loss of strength (psi)

6 holes / foot
0.5 in / hole

l_p = 3.0 in/ft

From determination of critical loading:

P_T = 60.9 psi

P_{DES1} = 81.2 psi

Adjust pipe stress determined above to account for effects of soil arching:

2. The design pipe stress is estimated by accounting for the soil structure interaction between the buried groundwater dewatering system pipe and its backfill to obtain a realistic loading condition on the pipe.

2a. For the burial conditions shown on Figure 1 (Sheet IIID-C-27), the pipe may be classified as a positive projecting conduit.

2b. Because the pipe is flexible and will deflect in the vertical plane as shown on Figure 2 (Sheet IIID-C-28), the pipe will experience a reduction in loading due to soil arching. Soil arching is present when the soil column over the pipe settles and creates shear stresses in the surrounding soil. Those shear stresses will support the soil column, thereby reducing the load experienced by the pipe (see Figure 3, Sheet IIID-C-28).

2c. The load on the pipe will be estimated using Marston's Formula:

$$W_c = \gamma C_c B_c^{\,2} \qquad (1)$$

$$C_c = \frac{e^{\pm 2k\mu(H_e/B_c)} - 1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2k\mu(H_e/B_c)} \qquad (2)$$

Where:

	W_c	= Load per unit length of conduit (lb/ft)
	γ	= Unit weight of soil above conduit (pcf)
	B_c	= Outer diameter of conduit (ft)
	H	= Height of fill above conduit (ft)
	H_e	= Height of plane of equal settlement above critical plane (ft)
	k	= Lateral pressure ratio (earth pressure coefficient)
	μ	= tan ϕ
	ϕ	= Angle of internal friction of pipe-zone backfill (PZB) (degrees)

$$H_e = \pm r_{sd}\, p\left(\frac{H}{B_c}\right) \qquad (3)$$

Where:

	r_{sd}	= Settlement ratio
	p	= Ratio of the conduit projection above the compacted soil liner to its diameter

$$r_{sd} = \frac{(S_m + S_g) - (S_f + dc)}{S_m} \qquad (4)$$

Where:

	S_m	= Compression deformation of soil column adjacent to conduit
	S_g	= Settlement of natural ground adjacent to conduit
	S_f	= Settlement of conduit into foundation material
	dc	= Vertical deflection of the conduit

It is assumed that for a groundwater dewatering system pipe S_g and S_f are equivalent. The equation settlement ratio, therefore reduces to the following:

$$r_{sd} = \frac{S_m - dc}{S_m} \qquad (5)$$

Since the trench aggregate (PZB) is much stiffer than the pipe, dc is larger than S_m implying that r_{sd} will be negative. Because r_{sd} is negative, the pipe is categorized as an incomplete ditch as specified by Marston. Note that in the above equations, where a + and a - sign are used together, the upper sign corresponds to a positive r_{sd} and a the lower sign to a negative r_{sd}.

2d. Load analysis solution by trial and error

Step 1: Assume a value for the settlement ratio, r_{sd}.

r_{sd} = -0.52

Step 2: Calculate S_m based on the estimated vertical stress at the level of the pipe and the deformation modulus E of the PZB.

$S_m = P_{DES1}\, D / E_s$

Where:
- P_{DES1} = Pipe stress adjusted for loss of strength (psi)
- D = Pipe diameter (in)
- E_s = PZB soil modulus (psi)

P_{DES1} =	81.2	psi
D =	4.5	in
E_s =	3,000	psi

S_m =	0.122	in

Step 3: Calculate dc using Equation (5):

$dc = S_m\,(1 - r_{sd})$

dc =	0.185	in

Step 4: Use the Iowa Formula (provided below) to calculate load per unit length (W_c).

$$W_c = \frac{dc}{(DL)k}\left(\frac{EI}{r^3} + 0.061E'\right)$$

Where:
- DL = Deflection lag factor
- k = Bedding factor
- E = Young's modulus for pipe material (psi)
- I = Moment of inertia for pipe wall = $t^3/12$ (in^4/in)
- r = Pipe radius (in)
- E' = Modulus of soil reaction (psi)

DL =	2.5	(Ref 6)
k =	0.1	(Ref 6)
E =	33,000	psi (refer to chart 25 on Sheet IIID-C-30, based on P_{DES1} above)
t =	0.390	in (SDR 17 pipe)
I =	0.005	in^4/in
r =	2.3	in
E' =	3,000	psi

$\boxed{W_c = 146 \text{ lb/in}}$

Step 5: Calculate C_c using Equation 1:

$$C_c = \frac{W_c}{\gamma B_c^{\,2}}$$

Composite unit weight for waste and soil:

4.0	ft soil @	120	pcf =	480	psf
115.2	ft waste @	72	pcf =	8,294	psf
			Total =	8,774	psf

γ =	73.61	pcf (weighted average based on above table)
B_c =	4.5	in

$\boxed{C_c = 169.2 \text{ (unitless)}}$

Step 6: Solve for H_e/B_c using Equation 2 in an iterative manner:

H =	115	ft
H/B_c =	307.2	

Assume: H_e/B_c = 1.94

$k\mu$ =	0.13	(Ref 4)
$e^{-2k\mu(He/Bc)}-1$ =	-0.40	
$-2k\mu$ =	-0.26	
$(H/B_c - H_e/B_c)$ =	305.3	
$e^{-2k\mu(He/Bc)}$ =	0.60	

Left-hand-side of equation (LHS) = 169
Right-hand-side of equation (RHS) = 186

Step 7: Substitute H_e/B_c into equation given below to determine if proper value for r_{sd} was used.

$$\left[\frac{1}{2k\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}\,p}{3}\right]\frac{e^{\pm 2k\mu(H_e/B_c)} - 1}{\pm 2k\mu} \pm \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$
$$\pm \frac{r_{sd}\,p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)e^{\pm 2k\mu(H_e/B_c)} - \frac{1}{2k\mu}\left(\frac{H_e}{B_c}\right) \mp \left(\frac{H}{B_c}\right)\left(\frac{H_e}{B_c}\right) = \pm r_{sd}\,p\left(\frac{H}{B_c}\right)$$

Because r_{sd} is negative for the incomplete ditch condition, the lower signs in the above equation are used.

p =	1
kμ =	0.13
H/B_c =	307.2
H_e/B_c =	1.94
r_{sd} =	-0.52
LHS =	159
RHS =	159

If LHS is not approximately equal to RHS, adjust value for r_{sd} in Step 1 and repeat solution procedure.

2e. Once the solutions to the above equations are determined, the design pipe stress may be calculated and the deflection of the pipe determined.

$P_{DES2} = W_c / D$

Where: P_{DES2} = Load on pipe adjusted to account for effects of soil arching (psi)

W_c =	146	lb/in
D =	4.5	in
P_{DES2} =	32	psi

A summary table for the structural stability analysis is provided on Sheet IIID-C-26 for the 4-inch-diameter groundwater dewatering system pipe. A pipe will be selected from this table for use in the groundwater dewatering system based on the calculated factors of safety for each possible failure condition. An example calculation is provided below that outlines the procedures used to determine the factors of safety for all pipe SDR sizes shown in the summary table.

B. Use the critical loading pressure to analyze pipe stability:

Example pipe structural stability calculations:

SDR	= Standard dimension ratio	=	17	
S_Y	= compressive yield strength	=	1,500	psi
RD_{all}	= allowable ring deflection	=	4.2	%

1. Wall crushing (Ref 3)

S_A = P_{DES2} (SDR - 1) / 2   FS = S_Y / S_A

Where:
- S_A = Actual compressive stress (psi)
- SDR = Standard dimension ratio
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- S_Y = Compressive yield strength (psi)
- FS = Factor of safety against wall crushing

P_{DES2} = 32 psi

S_A =	259.5	psi
FS =	5.8	

Compare calculated and suggested factor of safety:	5.8	> 1.0

2. Wall buckling (Ref 3)

P_{cb} = 0.8 (E' (2.32E / SDR^3))$^{1/2}$ FS = P_{cb} / P_{DES2}

Where:
- P_{cb} = Critical buckling pressure at top of pipe (psi)
- E' = Soil modulus (psi)
- E = Stress/time dependent tensile modulus for design loading conditions (psi)
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- FS = Factor of safety against wall buckling

E' =	3,000	psi
E =	27,000	psi for 50 years based on S_A above (see chart Sheet IIID-C-30)
P_{DES2} =	32	psi

P_{cb} =	156.5	psi
FS =	4.8	

Compare calculated and suggested factor of safety:	4.8	> 1.0

3. Ring deflection (Ref 3)

$E_S = P_{DES2} / E'$

Where:
E_S = Soil strain (%)
P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
E' = Soil modulus (psi)

P_{DES2} =	32	psi
E' =	3,000	psi
E_S =	1.1	%

Ring deflection for buried HDPE pipe is conservatively the same (no more than) the vertical compression of the soil envelope around the pipe. Therefore, assumed actual ring deflection (RD_{act}) is equal to soil strain.

RD_{act} = 1.1 %

Allowable ring deflection, RD_{all} = 4.20 %

$RD_{act} < RD_{all}$, design is acceptable

ROYAL OAKS LANDFILL
APPENDIX IIID-C
TEMPORARY UNDERDRAIN DEWATERING SYSTEM
PIPE STRUCTURAL STABILITY - 4" DIA PIPE

Adjusted load to account for soil arching = 32 psi

	Wall Crushing			Wall Buckling				Ring Deflection			
SDR	S_Y	S_A	FS_{WC}	E^2	E'	P_{cb}	FS_{WB}	RD_{all}	E'	RD_{act}	FS_{RD}
32.5	1,500	511.0	2.9	20,000	3,000	50.9	1.6	8.1	3,000	1.1	7.5
26.0	1,500	405.5	3.7	22,000	3,000	74.7	2.3	6.5	3,000	1.1	6.0
21.0	1,500	324.4	4.6	25,000	3,000	109.7	3.4	5.2	3,000	1.1	4.8
19.0	1,500	292.0	5.1	26,000	3,000	129.9	4.0	4.7	3,000	1.1	4.3
17.0 [1]	1,500	259.5	5.8	27,000	3,000	156.5	4.8	4.2	3,000	1.1	3.9
15.5	1,500	235.2	6.4	28,000	3,000	183.0	5.6	3.9	3,000	1.1	3.6
13.5	1,500	202.9	7.4	29,000	3,000	228.9	7.1	3.4	3,000	1.1	3.1
11.0	1,500	162.2	9.2	30,000	3,000	316.9	9.8	2.7	3,000	1.1	2.5

denotes standard size

[1] Select 4-inch-diameter HDPE SDR 17.0 pipe for use in the groundwater dewatering system based on the calculated factors of safety.

[2] Values for the modulus of elasticity were selected from the attached chart (page IIID-C-30), Reference 3, using the calculated stress in the pipe wall (S_A under the wall crushing heading in the above table) for a 50 year duration (maximum loading is the overburden load on the pipe).

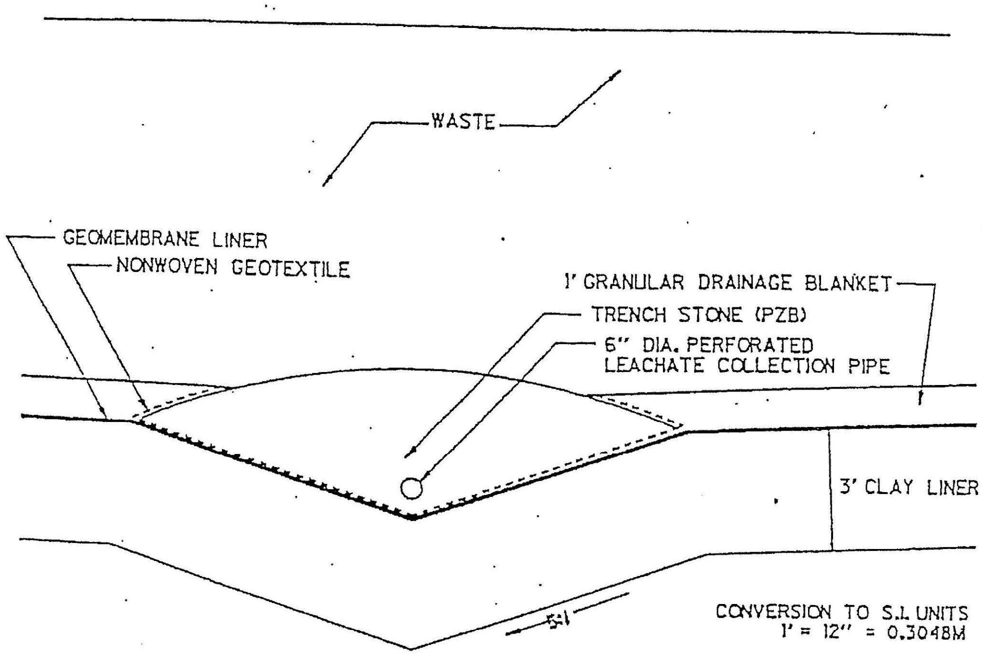


FIGURE 1: TYPICAL V-SHAPED TRENCH FOR LANDFILLS WITH COMPOSITE LINERS

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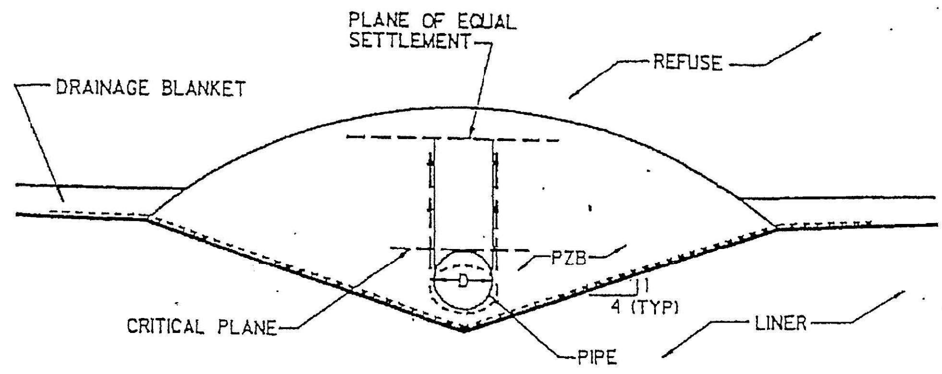

FIGURE 2: SETTLEMENT OF LEACHATE PIPE INDUCING SHEAR STRESSES IN PZB

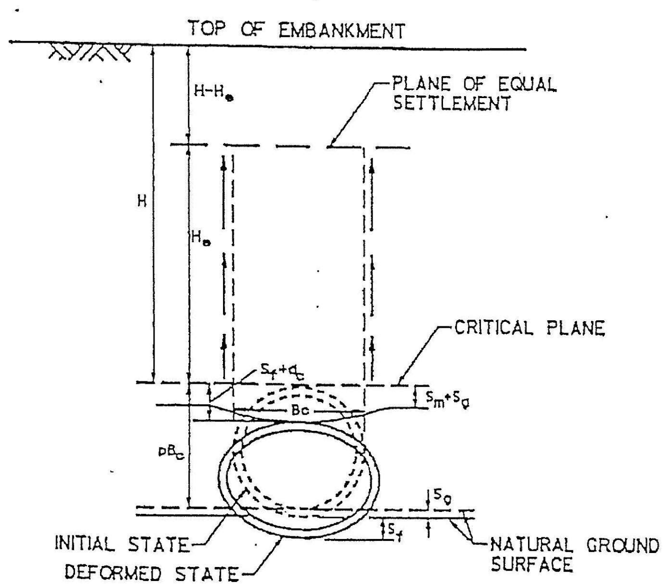

FIGURE 3: CASE OF AN INCOMPLETE DITCH CONDITION FOR A POSITIVE PROJECTING CONDUIT

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here: S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = External Pressure, psi

Safety Factor = 1500 psi ÷ S_A where 1500 psi is the Compressive Yield Strength of Driscopipe.

Design by Wall Buckling: Local wall buckling is a longitudinal wrinkling of the pipe wall. Tests of *non-pressurized* Driscopipe show that buckling and collapse do not occur when the soil envelope is in full contact with the pipe and is compacted to a dense state. However, it can be forced to occur over the long term in *non-pressurized* pipe if the total external soil pressure, P_t, is allowed to exceed the pipe-soil system's critical buckling pressure, P_{cb}. If $P_t > P_{cb}$, gradual collapse may occur over the long term. A calculated, conservative value for the critical buckling pressure may be obtained by the following approximate formula. All pipe diameters with the same SDR in the same burial situation have the same critical collapse and critical buckling endurance

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

Where:

P_t = Total vertical soil pressure at the top of the pipe, psi

P_{cb} = Critical buckling soil pressure at the top of the pipe, psi

E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to vertical soil strain at a specified density

P_c = Hydrostatic, critical-collapse differential pressure, psi

$$P_c = \frac{2E\,(t/D)^3\,(D_{MIN}/D_{MAX})^3}{(1-\mu^2)}$$

$$P_c = \frac{2.32\,E}{(SDR)^3}$$

Where: (D_{MIN}/D_{MAX}) = .95
μ = Poission's Ratio
μ = .45 for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Design by Wall Buckling Guidelines:
Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_t < P_{cb}$.

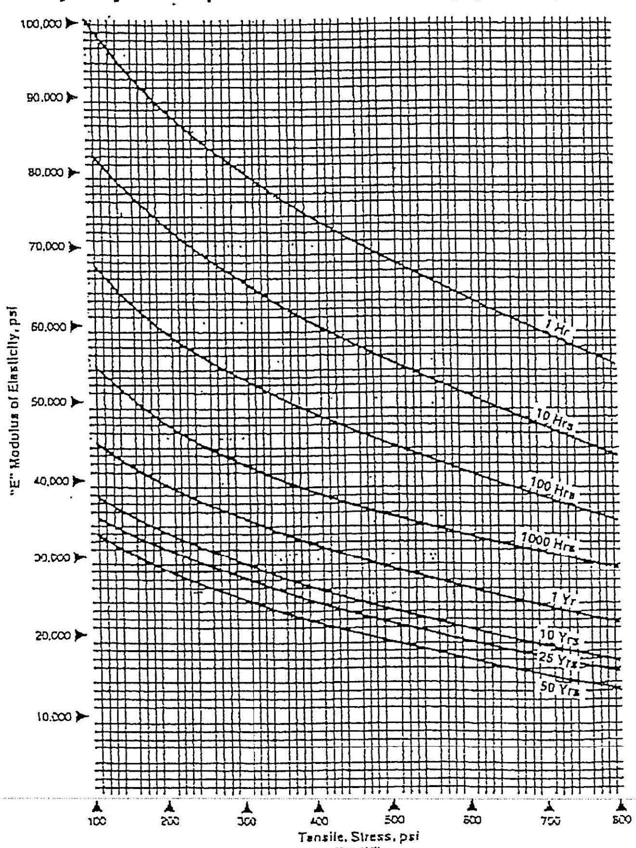
1. Calculate or estimate the total soil pressure, P_t, at the top of the pipe.
2. Calculate the stress "S_A" in the pipe wall according to the formula:

$$S_A = \frac{(SDR - 1)\,P_t}{2}$$

3. Based upon the stress "S_A" and the estimated time duration of non-pressurization, use Chart 25 to find the value of the pipe's modulus of elasticity, E, in psi.

Chart 25

Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

DRISCOPIPE

Simplified Burial Design: A conservative estimate of the ability of Driscopipe pipelines to perform in a buried environment is found in Chart 24. It is based on a minimum 2:1 safety factor and 50 year design service life. A detailed burial design starts on page 37. The detailed design should be used for critical or marginal applications or whenever a more precise solution is desired.

Detailed Burial Design:
Design by Wall Crushing: Wall crushing would theoretically occur when the stress in a pipe wall, due to the external vertical pressure, exceeded the long-term compressive strength of the pipe material. To ensure that the Driscopipe wall is strong enough to endure the external pressure the following check should be made:

$$S_A = \frac{(SDR - 1)}{2} P_T$$

Values of E′

Based on Soil Type (ASTM D2321) and Degree of Compaction

Soil Type of Initial Backfill Embedment Material	Description	E′ (psi) for Degree of Compaction (Proctor Density, %) Loose	Slight (70-85%)	Moderate (85-95%)	High (95%)
I	Manufactured angular, granular materials (crushed stone or rock, broken coral, cinders, etc.)	1,000	3,000	3,000	3,000
II	Coarse grained soils with little or no fines	N.R.	1,000	2,000	3,000
III	Coarse grained soils with fines	N.R.	N.R.	1,000	2,000
IV	Fine-grained soils	N.R.	N.R.	N.R.	N.R.
V	Organic soils (peat, muck, clay, etc.)	N.R.	N.R.	N.R.	N.R.

N.R. = Not Recommended for use by ASTM D2321 for pipe wall support

Chart 24

SDR	Maximum Burial Depth, ft. in dry soil of 100 lbs/cu. ft. Soil Modulus, psi* 1000	2000	3000	Maximum External Pressure psi Soil Modulus, psi* 1000	2000	3000	Maximum Deflection, % after installation Soil Modulus, psi* 1000	2000	3000
32.5	25	32	37	17	22	26	1.7	0.9	0.6
26	33	45	52	23	31	36	2.3	1.2	0.8
21	46	61	71	32	42	49	3.2	1.6	1.1
19	52	69	81	36	48	56	3.6	1.8	1.2
17	61	121	181	42	84	126	4.2	2.1	1.4
15.5	56	112	168	39	78	117	3.9	2.0	1.3
13.5	49	98	147	34	68	102	3.4	1.7	1.1
11	39	78	117	27	54	81	2.7	1.4	0.9
9.3	33	68	101	23	47	70	2.3	1.2	0.8
8.3	30	61	89	21	42	62	2.1	1.1	0.7
7.3	26	52	79	18	36	55	1.8	0.9	0.6

*assumes no external loads

REQUIRED: Analyze structural stability of the 18-inch-diameter groundwater dewatering system sideslope riser pipe.

METHOD:

A. Determine the critical load and calculate stress under the following two conditions:

1. Construction loading
2. Overburden loading (conservatively calculated based on a waste unit weight of 72 pcf versus 44.4 used for ballasting demonstration)

B. Use the critical loading pressure to analyze pipe stability under the following three possible failure conditions:

1. Wall crushing
2. Wall buckling
3. Ring deflection

NOTE: The groundwater dewatering system details shown on Sheets IIID-C-27 and IIID-C-28 are for illustration purposes only to show parameters used in the following calculations. Groundwater dewatering system details can be found in Appendix IIIA.

REFERENCES:

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems* , Pollution Technology Review No. 138, Noyles Data Corporation, 1986.

2. Texas Natural Resource Conservation Commission, *Leachate Collection System Handbook* , 30 TAC 330.201, 1993.

3. Phillips 66 Driscopipe, *System Design* , 1991.

4. Landfill Design Series, *Leachate Gas Management Systems Design, Volume 5, Leachate Management and Storage,* Appendix A, 1993.

5. Caterpillar Tractor Company*, Caterpillar Performance Handbook,* Edition 27, October 1996.

6. Quian, Xuede, R.M. Koerner, D. H. Gray, "Geotechnical Aspects of Landfill Design and Construction." Prentice-Hall, Inc., New Jersey, 2002.

SOLUTION:

A. Determine the critical load and stress:

A.1. Maximum construction loading

Assume: CAT 637E Series II scraper with an even load distribution

Loaded weight =	190,500	lb
Tire pressure =	80	psi
Number of tires =	4	

For a circular tire imprint:

$$F = \frac{\text{Loaded Weight}}{\text{Number of Tires}}$$

Where: F = Force exerted by one tire (lb)

F =	47,625	lb

Determine area of contact for circular tire imprint:

$$r = (F/\pi p)^{1/2}$$

Where:
- r = Radius of contact (in)
- F = Force exerted by one tire (lb)
- p = Tire pressure (psi)

r =	13.8	in

Use Boussinesq's solution to find the stress at a point below a uniformly loaded circular area:

$$y = p\,(1 - ((r/z)^2+1)^{-3/2})$$

Where:
- y = Change in vertical stress (psi)
- p = Tire pressure (psi)
- r = Radius of contact (in)
- z = Protective cover thickness (in)

z =	24	in

y =	27.8	psi

Assume only one wheel load on pipe and add 50% for impact loading:

P_L = 1.5y

Where: P_L = Maximum live load (psi)

P_L =	41.7	psi

P_D = (zw)/1728

Where: P_D = Maximum dead load (psi)
z = Protective cover thickness (in)
w = Unit weight of protective cover (pcf)

z = 24 in
w = 120 pcf

P_D =	1.67	psi

P_T = $P_L + P_D$

Where: P_T = Maximum construction load (psi)

P_T =	43.3	psi

A.2. Overburden loading (postclosure load):

For maximum fill load on pipe (at deepest sump location):

4.0	ft protective cover @	120	pcf =	480	psf
6.0	bottom and final cover@	120	pcf =	720	psf
115.2	ft solid waste/soil @	72	pcf =	8,294	psf
			Σ =	9,494	psf

P_T =	65.9	psi

Determine critical loading condition:

Construction loading:	P_T =	43.3	psi
Overburden loading:	P_T =	65.9	psi

Overburden loading is most critical to the structural stability of the pipe and will be used to determine the design pipe stress.

Determine Design Stress:

1. Adjust critical stress to account for loss of strength in the pipe due to perforations:

$$P_{DES1} = 12P_T / (12 - l_p)$$

Where:

l_p	=	Cumulative length of perforations per foot of pipe
P_T	=	Critical pipe stress (psi)
P_{DES1}	=	Pipe stress adjusted for loss of strength (psi)

6	holes / foot
0.5	in / hole

l_p =	3.0	in/ft

From determination of critical loading:

P_T =	65.9	psi

P_{DES1} =	87.9	psi

Adjust pipe stress determined above to account for effects of soil arching:

2. The design pipe stress is estimated by accounting for the soil structure interaction between the groundwater dewatering system pipe and its backfill to obtain a realistic loading condition on the pipe.

2a. For the burial conditions shown on Figure 1 (Sheet IIID-C-27), the pipe may be classified as a positive projecting conduit.

2b. Because the pipe is flexible and will deflect in the vertical plane as shown on Figure 2 (Sheet IIID-C-28), the pipe will experience a reduction in loading due to soil arching. Soil arching is present when the soil column over the pipe settles and creates shear stresses in the surrounding soil. Those shear stresses will support the soil column, thereby reducing the load experienced by the pipe (see Figure 3, Sheet IIID-C-28).

2c. The load on the pipe will be estimated using Marston's Formula:

$$W_c = \gamma C_c B_c^{\,2} \tag{1}$$

$$C_c = \frac{e^{\pm 2k\mu\left(H_e/B_c\right)} - 1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2k\mu\left(H_e/B_c\right)} \tag{2}$$

Where:	W_c	= Load per unit length of conduit (lb/ft)
	γ	= Unit weight of soil above conduit (pcf)
	B_c	= Outer diameter of conduit (ft)
	H	= Height of fill above conduit (ft)
	H_e	= Height of plane of equal settlement above critical plane (ft)
	k	= Lateral pressure ratio (earth pressure coefficient)
	μ	= tan ϕ
	ϕ	= Angle of internal friction of pipe-zone backfill (PZB) (degrees)

$$H_e = \pm r_{sd}\, p\left(\frac{H}{B_c}\right) \tag{3}$$

Where:	r_{sd}	= Settlement ratio
	p	= Ratio of the conduit projection above the compacted soil liner to its diameter

$$r_{sd} = \frac{\left(S_m + S_g\right) - \left(S_f + dc\right)}{S_m} \tag{4}$$

Where:	S_m	= Compression deformation of soil column adjacent to conduit
	S_g	= Settlement of natural ground adjacent to conduit
	S_f	= Settlement of conduit into foundation material
	dc	= Vertical deflection of the conduit

It is assumed that for a groundwater dewatering system pipe S_g and S_f are equivalent. The equation settlement ratio, therefore, reduces to the following:

$$r_{sd} = \frac{S_m - dc}{S_m} \tag{5}$$

Since the trench aggregate (PZB) is much stiffer than the pipe, dc is larger than S_m implying that r_{sd} will be negative. Because r_{sd} is negative, the pipe is categorized as an incomplete ditch as specified by Marston. Note that in the above equations, where a + and a - sign are used together, the upper sign corresponds to a positive r_{sd} and a the lower sign to a negative r_{sd}.

2d. Load analysis solution by trial and error

Step 1: Assume a value for the settlement ratio, r_{sd}.

r_{sd} = -0.68

Step 2: Calculate S_m based on the estimated vertical stress at the level of the pipe and the deformation modulus E of the PZB.

$S_m = P_{DES1} D / E_s$

Where:

	P_{DES1}	= Pipe stress adjusted for loss of strength (psi)
	D	= Pipe diameter (in)
	E_s	= PZB soil modulus (psi)

P_{DES1} =	87.9	psi
D =	18	in
E_s =	3,000	psi

S_m =	0.527	in

Step 3: Calculate dc using Equation (5):

$dc = S_m (1 - r_{sd})$

dc =	0.886	in

Step 4: Use the Iowa Formula (provided below) to calculate load per unit length (W_c).

$$W_c = \frac{dc}{(DL)k}\left(\frac{EI}{r^3} + 0.061E'\right)$$

Where:

DL	= Deflection lag factor
k	= Bedding factor
E	= Young's modulus for pipe material (psi)
I	= Moment of inertia for pipe wall = $t^3/12$ (in^4/in)
r	= Pipe radius (in)
E'	= Modulus of soil reaction (psi)

DL =	2.5	(Ref 6)
k =	0.1	(Ref 6)
E =	33,000	psi (refer to chart 25 on Sheet IIID-C-30, based on P_{DES1} above)
t =	1.059	in (SDR 17 pipe)
I =	0.099	in^4/in
r =	9.0	in
E' =	3,000	psi

W_c =	664	lb/in

Step 5: Calculate C_c using Equation 1:

$$C_c = \frac{W_c}{\gamma B_c^{\,2}}$$

Composite unit weight for waste and soil:

10.0	ft soil @	120	pcf =	1,200	psf
115.2	ft waste/soil @	72	pcf =	8,294	psf
			Total =	9,494	psf

γ =	75.8	pcf (weighted average based on above table)
B_c =	18	in

C_c =	46.7	(unitless)

Step 6: Solve for H_e/B_c using Equation 2 in an iterative manner:

	H =	125	ft
	H/B_c =	83.5	
Assume:	H_e/B_c =	2.28	
	kμ =	0.13	(Ref 4)
	$e^{-2k\mu(He/Bc)}-1$ =	-0.45	
	-2kμ =	-0.26	
	$(H/B_c - H_e/B_c)$ =	81.2	
	$e^{-2k\mu(He/Bc)}$ =	0.55	

Left-hand-side of equation (LHS) = 47
Right-hand-side of equation (RHS) = 47

Step 7: Substitute H_e/B_c into equation given below to determine if proper value for r_{sd} was used.

$$\left[\frac{1}{2k\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}\,p}{3}\right]\frac{e^{\pm 2k\mu(H_e/B_c)} - 1}{\pm 2k\mu} \pm \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$
$$\pm \frac{r_{sd}\,p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)e^{\pm 2k\mu(H_e/B_c)} - \frac{1}{2k\mu}\left(\frac{H_e}{B_c}\right) \mp \left(\frac{H}{B_c}\right)\left(\frac{H_e}{B_c}\right) = \pm r_{sd}\,p\left(\frac{H}{B_c}\right)$$

Because r_{sd} is negative for the incomplete ditch condition, the lower signs in the above equation are used.

p =	1
kμ =	0.13
H/B_c =	83.5
H_e/B_c =	2.28
r_{sd} =	-0.68
LHS =	56
RHS =	57

If LHS is not approximately equal to RHS, adjust value for r_{sd} in Step 1 and repeat solution procedure.

2e. Once the solutions to the above equations are determined, the design pipe stress may be calculated and the deflection of the pipe determined.

$P_{DES2} = W_c / D$

Where: P_{DES2} = Load on pipe adjusted to account for effects of soil arching (psi)

W_c =	664	lb/in
D =	18.0	in
P_{DES2} =	37	psi

A summary table for the structural stability analysis is provided on Sheet IIID-C-41 for the 18-inch-diameter groundwater dewatering system pipe. A pipe will be selected from this table for use in the groundwater dewatering system based on the calculated factors of safety for each possible failure condition. An example calculation is provided below that outlines the procedures used to determine the factors of safety for all pipe SDR sizes shown in the summary table.

B. Use the critical loading pressure to analyze pipe stability:

Example pipe structural stability calculations:

SDR	= Standard dimension ratio	=	17	
S_Y	= compressive yield strength	=	1,500	psi
RD_{all}	= allowable ring deflection	=	4.2	%

1. Wall crushing (Ref 3)

$S_A = P_{DES2}\,(SDR - 1) / 2$ FS $= S_Y / S_A$

Where:
- S_A = Actual compressive stress (psi)
- SDR = Standard dimension ratio
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- S_Y = Compressive yield strength (psi)
- FS = Factor of safety against wall crushing

P_{DES2} = 37 psi

S_A =	295.2	psi
FS =	5.1	

Compare calculated and suggested factor of safety:	5.1	> 1.0

2. Wall buckling (Ref 3)

$P_{cb} = 0.8\,(E'\,(2.32E / SDR^3))^{1/2}$ FS $= P_{cb} / P_{DES2}$

Where:
- P_{cb} = Critical buckling pressure at top of pipe (psi)
- E' = Soil modulus (psi)
- E = Stress/time dependent tensile modulus for design loading conditions (psi)
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- FS = Factor of safety against wall buckling

E' =	3,000	psi
E =	26,000	psi for 50 years based on S_A above (see chart Sheet IIID-C-30)
P_{DES2} =	37	psi

P_{cb} =	153.5	psi
FS =	4.2	

Compare calculated and suggested factor of safety:	4.2	> 1.0

3. Ring deflection (Ref 3)

$E_S = P_{DES2} / E'$

Where:
E_S = Soil strain (%)
P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
E' = Soil modulus (psi)

P_{DES2} = 37 psi
E' = 3,000 psi

E_S = 1.2 %

Ring deflection for buried HDPE pipe is conservatively the same (no more than) the vertical compression of the soil envelope around the pipe. Therefore, assumed actual ring deflection (RDact) is equal to soil strain.

RD_{act} = 1.2 %

Allowable ring deflection, RD_{all} = 4.20 %

$RD_{act} < RD_{all}$, design is acceptable

ROYAL OAKS LANDFILL
APPENDIX IIID-C
TEMP. UNDERDRAIN DEWATERING SYSTEM
PIPE STRUCTURAL STABILITY - 18"-DIA PIPE

Adjusted load to account for soil arching = 37 psi

	Wall Crushing			Wall Buckling				Ring Deflection			
SDR	S_Y	S_A	FS_{WC}	E[2]	E'	P_{cb}	FS_{WB}	RD_{all}	E'	RD_{act}	FS_{RD}
32.5	1,500	581.1	2.6	20,000	3,000	50.9	1.4	8.1	3,000	1.2	6.6
26.0	1,500	461.2	3.3	22,000	3,000	74.7	2.0	6.5	3,000	1.2	5.3
21.0	1,500	369.0	4.1	24,000	3,000	107.4	2.9	5.2	3,000	1.2	4.2
19.0	1,500	332.1	4.5	25,000	3,000	127.4	3.5	4.7	3,000	1.2	3.8
17.0 [1]	1,500	295.2	5.1	26,000	3,000	153.5	4.2	4.2	3,000	1.2	3.4
15.5	1,500	267.5	5.6	27,000	3,000	179.7	4.9	3.9	3,000	1.2	3.2
13.5	1,500	230.8	6.5	28,500	3,000	226.9	6.1	3.4	3,000	1.2	2.8
11.0	1,500	184.5	8.1	30,000	3,000	316.9	8.6	2.7	3,000	1.2	2.2

(shaded) denotes standard size

[1] Select 18-inch-diameter HDPE SDR 17.0 pipe for use in the groundwater dewatering system based on the calculated factors of safety.

[2] Values for the modulus of elasticity were selected from the attached chart (Sheet IIID-C-30), Reference 3, using the calculated stress in the pipe wall (S_A under the wall crushing heading in the above table) for a 50 year duration (maximum loading is the overburden load on the pipe).

APPENDIX IIID-D

WASTE-AS-BALLAST PLACEMENT RECORD

Includes pages IIID-D-1 through IIID-D-2

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

WASTE-AS-BALLAST PLACEMENT RECORD

This form is to be completed by the Site Manager or designated representative for all landfill areas utilizing waste as ballast. One form will be developed for each area (or combination of areas) described by approved liner evaluation reports. This form is to be submitted with the Ballast Evaluation Report (BER) for the evaluated area and may be referenced by the Professional of Record (POR) in order to verify that the placement of ballast is in compliance with the Liner Quality Control Plan (LQCP). The site operator must prepare and sign supporting documentation on a daily basis verifying the area of waste placement, the waste material in the first 5 feet of waste was free of large bulky items, daily operation of the pressure relief/dewatering system, and a wheeled trash compactor having a minimum weight of 40,000 pounds was used.

A. GENERAL INFORMATION

Area documented by this record (provide site grid coordinates of each corner) _____________
____

Soils and Liner Evaluation Report document date(s) and approval date(s) for this area _______

Date of initial waste placement _____________

Date of completion of first 5 feet of waste in place over entire area _______________________

Total required waste-as-ballast thickness for this area (Note: Calculations for determining the required thickness of waste as ballast are included with the LQCP/BER for this area.) ___________________

Date when minimum required thickness of waste was achieved _________________________

B. WASTE EQUIPMENT USED

What type of compaction equipment was used? _______________________________________

Did the compactor have a minimum gross weight of 40,000 pounds? _____________________

Was this compactor used throughout the entire period covered by this record? _____________

If a minimum 40,000-pound wheeled trash compactor was not used throughout the period covered by this record, attach documentation of initial and final survey data (if not previously provided as part of the BER) of the ballasted area and measurements of truck weights at the scalehouse for the time period covered by the BER for use in determining in-place waste density. Is this documentation complete and accurate? _________________________________

C. FIRST WASTE LIFT CONSIDERATIONS

Describe type(s) of waste placed in first 5 feet of waste over the top of the liner protective cover ____

Does the first 5 feet of waste contain any large bulky waste items which would damage the underlying liner system or which cannot be compacted to the required density? ______________________

D. WASTE COMPACTION METHODS

Approximate loose waste layer thickness prior to compaction _______________

Minimum number of compactor passes for each waste layer ________________

Maximum slope of compacted waste layers ______________________________

E. PRESSURE RELIEF/DEWATERING SYSTEM

Was the pressure relief/dewatering system (if required) operated continuously during the period covered by this record? __________ Is the pressure relief/dewatering system presently in operation? ____

SIGNATURE OF PERMITTEE OR OPERATOR

The waste overlying the area described in this record has been placed and compacted as described in this record and in accordance with the Liner quality control plan and Site Operating Plan.

______________________________	Royal Oaks Landfill
(Signature)	(Business Name or Facility)
______________________________	______________________________
(Typed or Printed Name)	
______________________________	______________________________
(Title)	(Address, City, Zip Code)
______________________________	______________________________
(Date Signed)	(Phone No.)

Note: This completed form must be submitted with the BER and placed in the Operating Record and available for review.

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS
TCEQ PERMIT NO. MSW-1614B

MAJOR PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN

APPENDIX IIIE
GEOTECHNICAL REPORT

Prepared for

Pine Hill Farms Landfill TX, LP

May 2024

05/20/2024

Prepared by

Weaver Consultants Group, LLC
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817-735-9770

WCG Project No. 0120-076-11-106

This document intended for permitting purposes only.

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DAVID E. POE
81734
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05/20/2024

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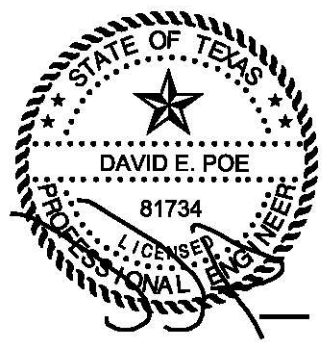

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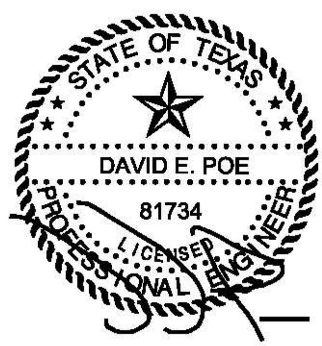
TABLES



05/20/2024

1 INTRODUCTION

This appendix addresses §330.63(e)(5)(A) and (B).

The purpose of this report is to present the geotechnical analysis and design for the proposed major permit amendment for the vertical and lateral expansion of the Royal Oaks Landfill located in Cherokee County. This report is based on the geotechnical testing information obtained during field and laboratory investigations conducted in 2023, as well as the information compiled from earlier geological studies at the landfill as compiled from the subsurface investigations from previous permits.

This report contains a compilation of geotechnical testing and design information, including:

- Presentation of the geotechnical (field and laboratory) and geological information compiled during the 2023 and previous permit applications and incorporated into his amendment.
- Slope stability analyses performed based on the geotechnical testing results and subsurface conditions, including groundwater, for landfill excavations, landfill completion, and sequence of development (interim condition) plans; and
- Settlement and heave analyses, which are also based on the landfill excavation and completion plans.

The stability analyses and settlement and strain analyses considered both developed and undeveloped portions of the landfill, with the primary focus of the analyses being the unconstructed expansion area cells 10 through 12. The analyses also includes evaluation of the leachate piping system incorporated into the bottom liner (in future cells), and the effects of foundation settlement on the design piping slopes and grades (see Appendix IIIE-B).

This report also provides geotechnical recommendations for construction of the landfill components, including bottom liner and final cover systems with soil and geosynthetic materials. The construction quality control and material and construction specifications for the groundwater protection components of the landfill are provided in Appendix IIID–Liner Quality Control Plan (LQCP).

2 LABORATORY TESTING

2.1 Introduction

Numerous geological investigations have been performed at the Royal Oaks Landfill for previous permitting efforts and are discussed in further detail in Appendix IIIG – Geology Report. The information used for the geotechnical studies presented in this appendix were derived primarily from field and laboratory investigations conducted in 2023. Discussion of the investigation findings is presented below.

Geotechnical investigation activities included the sampling and geotechnical testing of samples obtained during the investigations. A brief description of the geological/ geotechnical characteristics for the strata identified at the site is presented in Section 3 of this appendix. Additional geological and hydrogeological discussion is provided in Appendix IIIG–Geology Report of this application.

Laboratory tests were conducted on select samples recovered from the borings to evaluate the physical and engineering properties of the varying strata. Laboratory tests were performed in general accordance with ASTM procedures. Laboratory testing results from the 2023 investigations are provided in Appendix IIIE-C and on boring logs included in Appendix IIIG–Geology Report. The results of laboratory testing are summarized in the material descriptions presented in Section 3 of this appendix. A summary of the laboratory tests performed is given in Table 2-1. A summary table presenting the results of the geotechnical laboratory testing is also included in Appendix IIIE-C.

Table 2-1
Geotechnical Test Methods

Test	Test Method
Sieve Analysis (Passing No. 200)	ASTM D 140
Atterberg Limits (Liquid & Plastic Limit)	ASTM D4318
Moisture Content	ASTM D2216
Unconfined Compression	ASTM D 2166 & Pocket Penetrometer
Triaxial Compression Test	ASTM D4767
Coefficient of Permeability (Hydraulic Conductivity)	Vertical - ASTM D5084 Method F Horizontal – ASTM D4044 and D8084 Method F
Consolidation	ASTM D2435
Hand Penetrometer Testing	ASTM D2573
Standard Proctor	ASTM D698

2.2 Classification Tests

Classification tests consisting of Atterberg limits, percent passing the #200 sieve, moist unit weight, and moisture content were performed on selected soil samples recovered from boreholes. Classification tests were used to characterize the soils according to the Unified Soil Classification System (USCS) and to evaluate physical properties of the soils. The test results for the strata identified at the site are presented in Section 3 of this appendix and summarized in the table included in Appendix IIIE-C.

2.2.1 Material Strength Tests

Material strength tests were performed to provide generalized strength parameters that were used to evaluate the soils at the site. Additionally, triaxial testing was performed to assist developing strength profiles for selected strata. The triaxial testing was performed for consolidated undrained conditions. Note that strength testing of the sand stratum was not possible as undisturbed samples could not be collected. Strength values for the sands as required for stability modeling were developed from review of field logs and WCG experience with similar formations.

2.2.2 Hydraulic Conductivity Tests

Laboratory hydraulic conductivity tests were performed to evaluate the hydrogeological properties of the soils at the site. Additional discussion regarding the hydraulic conductivity testing is presented in Appendix IIIG–Geology Report and has not been reproduced for this appendix.

2.2.3 Consolidation Tests

Consolidation data used for settlement analyses was developed from information obtained during the 2023 investigations. Consolidation properties for sands and sandy soils as required for settlement analysis were assumed based on published information related to non-elastic settlement of granular soils, and based on soil characteristics observed during field investigations.

Consolidation data for the elastic soils encountered during investigations was derived from laboratory test results of field samples obtained during the investigations. Consolidation properties of waste were obtained from cited references. Combined, the above information was used in estimating the settlement and heave characteristics of the landfill and underlying foundation strata.

The results of the consolidation testing performed on elastic soils encountered in the landfill foundations are presented in Appendix IIIE-C. The settlement analyses presented in Appendix IIIE-B incorporate the test results.

2.3 Conclusion of Laboratory Testing

Classification testing along with unit weight, moisture content, and sieve analysis results were used to support field observations during subsurface explorations. Testing results were also used to support the subsurface characterization across the site. Additionally, soil strength and consolidation parameters from both field and laboratory were conservatively selected for use in the stability and settlement analyses, respectively.

3 SITE STATIGRAPHY AND SOIL PROPERTIES

3.1 General

This section of the report includes the generalized stratigraphy for the site, typical properties of subsurface soils, potential uses of materials that may be excavated during construction, and soil material requirements for various components of the landfill.

The laboratory test results for soil samples obtained from the site are summarized in the material descriptions for each subsurface stratum below. Laboratory testing results are presented in Appendix IIIE-C.

3.2 Site Stratigraphy

The site stratigraphy at the site is described in detail in Appendix IIIG – Geology Report of this application, including geological cross-sections presenting both site stratigraphy and the results of soil borings installed at the site. The below is a synopsis of the information presented in Appendix IIIG – Geology Report.

The existing subsurface characterization of the site is supported by data from 88 advanced borings at locations shown Figure IIIG-B-1 in Appendix IIIG – Geology Report. The data from these borings is summarized in Table 3-1 and the individual lithologic logs are provided in Appendix IIIG-B. The borings were advanced during 10 drilling events conducted between 1981 and 2023 and are further discussed in Section 3.3 of Appendix IIIG. To illustrate subsurface conditions, seven geologic cross sections were constructed from the available lithologic and hydrogeologic data obtained from the site-specific lithologic logs (provided in Appendix IIIG-B), local water well logs (provided in Appendix IIIG-A), and information contained in prior investigatory reports. These cross sections are presented in Appendix IIIG-C as Figures IIIG-C-2 through IIIG-C-8.

The subsurface investigation data and geologic cross sections indicate that the facility's geology can be divided into five site-specific stratigraphic units (Surficial Sediments, Stratum A, Stratum, B, Stratum C, and Stratum D) with the lowermost four strata comprised of aquifer and aquiclude subunits. The nomenclature for these site-specific stratigraphic units generally corresponds to the unit designations in the permitted subsurface characterization for Permit No. MSW-1614B.

Site-specific strata and substrata designations are listed below with their corresponding former nomenclature listed in parenthesis/italics:

- Surficial Sediments: (*Sparta Sand* and *Clay Overlying Aquifer*),
- Stratum A:
  - Stratum A1 - Aquifer A (*Aquifer A*)
  - Stratum A2 – Aquiclude A (*Basal Clay A*)
- Stratum B:
  - Stratum B1 - Aquifer B (*Aquifer A*)
  - Stratum B2 – Aquiclude B (*Basal Clay A*)
- Stratum C:
  - Stratum C1 - Aquifer C (*Sediments Below Basal Clay B*)
  - Stratum C2 – Aquiclude C (*Sediments Below Basal Clay B*)
- Stratum D:
  - Stratum D1 - Aquifer D (previously uncharacterized)
  - Stratum D2 – Aquiclude D (previously uncharacterized)

At ground surface in undeveloped areas across the western and southern permit boundary areas lies the Surficial Sediments site-specific stratum which is comprised of Sparta Sand and uppermost Weches formation sediments. The Surficial Sediments are discontinuous across the permit boundary and have been removed from within the constructed limits of waste. The remaining Surficial Sediments are present within the western and southern permit boundary and the existing developed limits of waste at elevations above 640 ft-amsl. The Surficial Sediments do not exist within the eastern half of the permit boundary and proposed expansion area. According to the existing site exploration data, these sediments exhibit a high degree of compositional heterogeneity with interbedding and abrupt sudden to gradational transitions between predominate material composition, and a predevelopment average thickness of approximately 15 feet.

The uppermost Surficial Sediments are present due to the in-situ weathering of Sparta Sand Formation sediments which are composed predominately of unconsolidated dry to moist sand and silty sand, with lesser proportions of sandy silt, silt, and clayey sand, and ferrous interbedding. These uppermost sediments are present within limited areas of the northwestern and western permit boundary, outside the developed limits of waste, generally above elevation of 660 ft-amsl. The lowermost Surficial Sediments are present due to the in-situ weathering of uppermost Weches Formation sediments which are composed predominately of unconsolidated dry to moist glauconitic silty clay, with lesser proportions of sandy clay.

Hydrogeological and groundwater information related to the site stratigraphy is presented in Appendix IIIG – Geology Report of this application.

4 CONSTRUCTION CONSIDERATIONS

4.1 General

This section contains recommendations for excavation of the landfill, soil liner, leachate collection layer and final cover materials and construction. Additionally, operational cover soils, final cover construction, and perimeter embankment construction-related recommendations are included in this section.

The existing 144.3-acre permit boundary will not be changed with this amendment application. The permitted limit of waste will be increased by 28.6 acres, from approximately 54.5 acres to 83.1 acres.

The currently developed Subtitle D liners of the landfill include groundwater dewatering systems for temporary hydrostatic uplift pressure relief below the bottom liner system. The future Cells 10 through 12 will also require temporary groundwater uplift control in portions of the sideslopes of the excavation and as described in Appendix IIID-C of Appendix IIID–LQCP.

4.2 Material Requirements for Landfill Components

Construction of the landfill will require controlled soil placement to provide liner system foundations, perimeter berms and containment structures, and other earthen features. Bottom liner and final cover infiltration layer alternatives include compacted clay and geosynthetic clay liner (GCL).

Soil will also be required for protective cover over the liner and operational cover (daily and intermediate cover). Granular material (i.e., gravel) will be used for the leachate collection sumps, leachate collection chimneys and groundwater dewatering collection trenches. Typical material requirements for various soil fill applications are summarized in Table 4-1. Gradation requirements for granular materials is provided in Appendix IIID-LQCP.

Testing requirements and construction quality control and quality assurance for liner soils are detailed in Appendix IIID-LQCP. Testing requirements and construction quality control and quality assurance for final cover soils are detailed in Appendix IIIJ-Closure Plan and in Appendix IIIJ-A–Final Cover System Quality Control Plan (FCSQCP). Liner and final cover details are presented in Appendix IIIA-A–Landfill Unit Design Information.

Table 4-1
Typical Soil Requirements for Landfill Construction[3]

Landfill Component	Soil Description	Classification	Test Parameters				Material Source
			LL	PI	% – 200	Coefficient of Permeability cm/s	
Soil Liner	clayey sand, sandy clay, or clay	SC, CL, CH	30 min	15 min	30 min	$1x10^{-7}$ max	On site[1]
Final Cover Infiltration Layer	clayey sand, sandy clay, or clay	SC, CL, CH	30 min	15 min	30 min	$1x10^{-5}$ max[2]	On site
Liner Protective Cover	sand, sandy silt or clay, clayey or silty sand, silt and clay	SP-SM, SP, SP-SC, SW, SM or SM-SC, ML, CL, CH	(2)	(2)	(2)	$1x10^{-4}$ min	On site[2]
Final Cover Erosion Layer	clayey sand, sandy clay, or clay	SC, CL, SM	Suitable to support plant growth				On-site
Operational Cover[2] (Daily Cover and Intermediate Cover)	sand, sandy silt or clay, clayey or silty sand, silt and clay	SP-SM, SP, SP-SC, SW, SM or SM-SC, ML, CL, CH	--	--	--	--	On-site
Earth Fill Perimeter Berm and Subgrade Preparation	sand, sandy silt or clay, clayey or silty sand, silt and clay	SP-SM, SP, SP-SC, SW, SM or SM-SC, ML, CL, CH	--	--	--	--	On-site

[1] If on-site materials meeting the required properties do not exist, an off-site material source can be used for liner soil.
[2] If on-site material does not meet the hydraulic conductivity criteria, leachate collection chimney drains will be extended through the protective cover at selected locations and will be exposed adequately for transmission of leachate to the collection system.
[3] Granular material requirements and gradation provided in Appendix IIID-LQCP.

4.3 Landfill Excavation

The excavation for the bottom liner construction will be performed in a manner that will achieve reasonable segregation of liner quality material from soils that are not suitable for liner construction. Soil materials to potentially be used for liner construction will be stockpiled separately, according to construction material properties outlined in Section 4.4 and visual observation during excavation. Alternatively, the operator may elect to not segregate the soils in anticipation of substituting GCL for the compacted clay liner component of the bottom liner system.

Excavation of the soils encountered will be achieved with equipment such as bulldozers and excavators. Localized zones of cemented sands may be encountered intermittently within the excavation. If encountered, these zones can be broken up with an excavator equipped with a hydraulic hammer tool or ripped. The hydraulic hammer may be fitted with a pointed chisel or moil or a blunt tool for harder cemented material. Blasting of hard rock will not be required and will not be used at this site.

Excavation side slopes will be graded no steeper than 3 horizontal to 1 vertical (3H:1V). Temporary slopes during excavation may be steeper. Excavation cut slopes within the future cell construction areas may require erosion protection if an extended period of time occurs between excavation and liner construction. Interim erosion protection can be accomplished by diverting runoff away from the slopes. “Track walking” with a bulldozer up and down the slopes will create the effect of “mini-dikes” with the bulldozer tracks, which will also reduce erosion.

Prior to beginning construction of the liner components, the subgrade area will be stripped to a depth sufficient to remove all loose surface soils or soft zones within the exposed excavation. The liner base grades will be proof-rolled with heavy rubber-tired construction equipment or equivalent to detect soft or pumping areas. Soft or pumping areas will be undercut to firm material and backfilled with suitable compacted clay fill, as discussed in Appendix IIID-LQCP. Preparation of the liner base grades will result in a surface that is stable and that does not exhibit rutting from the construction traffic. The prepared liner base grades will be approved by a Professional of Record (POR), tested to verify that it meets the requirements outlined in Appendix IIID–LQCP, and surveyed to verify grades.

4.4 Soil Liner Construction

The bottom and sides of the landfill excavation may consist of 2-foot-thick compacted clay liner (in instances GCL is not substituted for compacted clay liner). The clay liner will have a maximum hydraulic conductivity of 1×10^{-7} cm/s. Details for the liner system are provided in Appendix IIIA (Appendix IIIA-A). Adequate soil liner material will be available from proposed landfill excavations or on-site borrow areas, or offsite borrow sources. Preconstruction laboratory tests may be performed to verify that a borrow source soil material is adequate to meet the compacted clay liner requirements listed in Title 30 TAC §330.339(c)(5) prior to using any soil borrow source as liner. As previously stated, GCL may be used as a alternative to the compacted clay liner.

The soils used for liner construction will have the minimum soil property values listed in Table 4-2 that will be verified by preconstruction testing in a geotechnical laboratory. The following soil liner properties are also included in Appendix IIID–LQCP.

Table 4-2
Compacted Clay Liner Properties

Test	Specifications
Hydraulic Conductivity of Remolded Soils[1]	1.0x10^{-7} cm/s or less (soil liner)
Plasticity Index[2]	15 minimum
Liquid Limit[2]	30 minimum
Percent Passing No. 200 Mesh Sieve[2]	30 minimum
Percent Passing 1-inch Sieve[2]	100

[1] A hydraulic conductivity test will be performed on soil samples remolded per ASTM D698 in accordance with Appendix IIID-LQCP.
[2] Testing applicable to soil liner only.

Representative preliminary sampling will be performed on the materials that will be used for soil liner construction. Laboratory tests of samples recovered from soil borings or test pits, as well as previous testing conducted during liner construction will demonstrate that soils which will achieve a coefficient of permeability of less than 1x10^{-7} cm/s are present at the site. Prior to construction of each new liner incorporating compacted clay, conformance tests that include Atterberg limits, percent passing the No. 200 sieve, Standard Proctor (ASTM D698) and remolded hydraulic conductivity will be performed. Additional conformance tests will be conducted if there are visual changes in the borrow material or the liquid limit or plasticity index vary by more than 10 points. The soil liner construction and testing procedures are outlined in Appendix IIID-LQCP.

4.5 Drainage Materials

The LCS drainage material will consist of a drainage geocomposite over the entire liner bottom and side slopes. Each cell will have a bottom slope toward an LCS trench (i.e., pipe enveloped in gravel and geotextile) that will collect leachate from the bottom and sideslopes. The leachate collection system details are illustrated in Appendix IIIA (Appendix IIIA-A). The material specifications and construction procedures for the LCS components are presented in Appendix IIID–LQCP. The LCS design and demonstrations are provided in Appendix IIIC–Leachate and Contaminated Water Management Plan.

4.6 Liner Protective Cover

The liner protective cover will be a minimum thickness of 24 inches. The purpose of the protective cover is to protect the geosynthetics (i.e., geomembrane and drainage geocomposite) from solid waste placed over the liner system. To ensure passage of leachate into the leachate collection system, drainage passages (chimney drains) will be constructed through the protective cover (if the protective cover soils have a hydraulic conductivity less than $1x10^{-4}$ cm/sec). The chimney drains will be installed over the LCS collection pipes as shown in Appendix IIIA (Appendix IIIA-A). The protective cover soils will be placed with construction equipment in one lift such that it covers the leachate collection layer completely. The protective cover will be free of solid waste and will not require compaction under the density-controlled construction procedures.

4.7 Operational Cover Soils

Operational cover soils include daily cover (placed over the waste each day) and intermediate cover (placed over waste in areas that will not receive additional fill for at least 6 months). All soils excavated at the site may be used for operational cover.

4.8 Composite Final Cover Construction

4.8.1 Final Cover Infiltration Layer Construction

The final cover infiltration layer is designed to reduce infiltration of surface water into the waste. The infiltration layer of the final cover system will be constructed with clayey soils and will be minimum of 18 inches in thickness overlain by geomembrane. A GCL may be substituted for the clayey soil layer as shown on drawings in Appendix IIIA-A. The clayey soil layer will have a coefficient of permeability equal to or less than $1x10^{-5}$ cm/s. The final cover components material and construction requirements will be in accordance with Appendix IIIJ-A–FCSQCP.

4.8.2 Final Cover Erosion Layer Construction

As shown in Appendix IIIA-A, the composite final cover system will include a 12-inch-thick erosion layer. The erosion layer will protect the infiltration layer and will support vegetative growth. The erosion layer may be spread and placed as a 12-inch-thick lift (with soils that will support vegetation) or with two 6-inch-thick lifts (with the upper 6 inches capable of supporting vegetation) over the entire final cover area as the final cover is constructed. After spreading, each lift will be compacted lightly to reduce future erosion but not to the extent that compaction would inhibit plant growth. The top 6 inches of the erosion layer will consist of

(1) topsoil stockpiled during the excavation process, (2) other on-site excavated soils amended as necessary to be capable of sustaining vegetation, and/or (3) imported soil materials. Whether placed in a single lift or two lifts, the erosion layer (top of final cover) will sustain vegetative growth.

4.9 Perimeter Embankment Construction

Perimeter embankments (berms) previously were constructed at the landfill and will be constructed at future cells as required to prevent surface water flow from entering the landfill excavation. Constructed embankments will have side slopes no steeper than 3H:1V. A sufficient amount of soil is available from the landfill excavations or on-site borrow areas to construct the perimeter embankment and other features that require soil fill material.

Prior to beginning embankment fill, the subgrade area will be stripped to a depth sufficient to remove all topsoil and vegetation. Topsoil will be stockpiled for later use. The subgrade area will be proof-rolled with heavy, rubber-tired construction equipment to detect soft areas. Soft areas will be undercut to firm material and backfilled with suitable compacted clay fill. The subgrade preparation will result in a subgrade surface that is stable and does not exhibit significant rutting from construction equipment traffic.

The embankments will be constructed of soils free of organic or other objectionable materials. As necessary, the outside slope of all embankment construction will be vegetated to minimize erosion and desiccation.

4.10 General Fill Construction

General fill material may be required for subgrade preparation, embankments, haul roads, and other miscellaneous fill. Material availability, compactability, and long-term maintenance requirements will be considered when evaluating the excavated soils for use as earth fill. Most soils that will be excavated for landfill development are suitable for use as earth fill. General fill placement methods are discussed in Section 2.3.3 of the Appendix IIID–LQCP.

5 SLOPE STABILITY ANALYSIS

5.1 General

This slope stability analysis has been developed to analyze excavation slopes, interim slopes, and landfill completion slopes using critical sections for each condition. The computer model SLIDE2 (RocScience, Inc., 2023) was used to analyze the stability of excavation slopes, interim fill slopes, and the final configuration of the landfill. SLIDE2 is an industry standard computer program developed by RocScience, Inc.

SLIDE2 is a two-dimensional slope stability program for evaluating the safety factor or probability of failure of circular and non-circular failure surfaces in soil or rock slopes. SLIDE2 analyzes the stability of slip surfaces using vertical slice or non-vertical slice limit equilibrium methods like Bishop, Janbu, Spencer, and Sarma, among others. Individual slip surfaces can be analyzed, or search methods can be applied to locate the critical slip surface for a given slope. SLIDE2 incorporates a windows-based interface that allows input of analysis sections and geological conditions from AutoCAD design drawings. The input file for the SLIDE2 program includes:

- Slope surface geometry.
- Subsurface information to identify different types of soil materials in horizontal and vertical directions so that each subsurface segment is identified with corresponding soil strength parameters.
- Groundwater information. The program is capable of modeling multiple groundwater surfaces that may be applicable to various subsurface soil components identified in the second bullet.
- Material strength information. Each soil section and geosynthetic interface (horizontal or vertical) is assigned with strength parameters including cohesion and friction angle for both total and effective stresses or peak residual stresses for use in analysis of soil, geosynthetic and soil-geosynthetic interfaces.
- Model control and simulation user interface of the model that allows selection of the method of analysis (e.g., Simplified Bishop) and identifying simulation control parameters.

Automatic failure surface generation functions, that use either initiation/termination ranges of the failure surface or use search boxes to define failure surface location, are used to locate the critical failure surface. The two methods employed for this slope stability analysis are described below.

1. Simplified Janbu Method – This method uses the method of slices to determine the stability of the mass above a failure surface.
2. Simplified Bishop Method – This method uses the method of slices to discretize the soil mass for determining the factor of safety.

In general, the stability of various critical sections was analyzed under static conditions for short-term (excavation and interim construction) and long-term safety. The slope stability analyses are provided in Appendix IIIE-A. The stability of the various liner and final cover configurations with the geosynthetic components were also evaluated using infinite slope stability analysis (refer to Appendix IIIE-A-4).

The stability analyses developed for this project demonstrate that the forces resisting slope movement (referred to as the resisting forces) are higher than the forces potentially creating movement for each of the sections analyzed. The ratio of forces resisting movement to the forces potentially creating movement (referred to as driving forces) is defined as the factor of safety (FS). When the FS is equal to or greater than 1.0 it means that the slope is theoretically stable. When conducting slope stability analysis a factor of safety greater than 1.0 is desired. The desired FS value is increased for the increased uncertainty within the system analyzed. A factor of safety of 1.5 has been used for slopes that will stay in place long-term, including final cover configurations. A factor of safety of 1.3 is acceptable for stress conditions that will be applicable for short periods of time, including interim and excavation slopes. A minimum factor of safety of 1.1 is acceptable for residual or large deformation failure conditions (typical of Rankine-Block analyses of critical geosynthetic interfaces).

5.2 Sections Selected for Analysis

Slope stability analyses were performed on critical sections to evaluate the stability of the excavation, interim and final cover slopes. The critical section locations were selected based on review of the proposed excavation and final cover plans, and incorporate bottom liner and final cover grades, drainage and access structures at the outside toe of the side slopes and generalized geological conditions beneath selected section. Figures showing the location of the critical sections developed for slope stability modeling are included in Appendix IIIE-A.

5.3 Configurations Analyzed

The excavation, interim and final cover configurations were modeled to represent critical slope conditions, and the analysis was performed using circular and block failure surfaces. The maximum final (closure) fill slopes will be 4H:1V, while interim slopes, internal liner slopes, and excavation slopes will be no steeper than 3H:1V. These are the slopes incorporated into the slope stability modeling.

A copy of the top of liner plan and final completion plan showing the locations of the cross sections selected for analysis are included as Sheets IIIE-A-7 and IIIE-A-8 in Appendix IIIE-A. Additionally, the configurations analyzed are graphically illustrated in Sheets IIIE-A-9 through IIIE-A-13 in Appendix IIIE-A. The interim condition was analyzed considering a 3H:1V slope with a horizontal length of approximately 450 feet (150 feet vertically). If the horizontal length of actual interim slopes longer than 450 feet is developed during site operations, a permit modification supporting the increased slope length will be submitted.

5.4 Input Parameters

The cross sections for slope stability analysis were developed for each of the conditions analyzed (see Figures IIIE-A-7 through IIIE-A-13). The soil and geosynthetic parameters were selected based on a review of the boring logs and laboratory test results from the 2023 subsurface investigation and upon engineering judgment and experience with similar materials. A summary of material and interface strength values considered for the modeling are presented on page IIIE-A-6 included in Appendix IIIE-A. Table 5-1 summarizes the unit weights and strength parameters used for the stability analyses for the evaluated landfill slopes (excavation, interim and final cover slopes). Note that for analyzing interface failure surfaces (planes) along the bottom liner system a single 2-foot-thick zone was input into the SLIDE2 model to represent the bottom liner system, and the weakest strength parameters from the table included on page IIIE-A-6 was assigned to this zone.

5.4.1 Groundwater

The geological logs for the lateral expansion area as well as the unconstructed footprint of the previously permitted landfill highlight the discontinuity and variability of the multiple perched groundwater zones in the near-surface strata at the site. Review of logs in the vicinity of the expansion area indicate perched groundwater with thicknesses ranging from 8 to 25 feet (approximately) perched on clay and clayey silt stratum located at approximate elevations 545 to 583 ft-msl. Groundwater within the sands underlying the site (Aquifer D) is at an estimated elevation of 457 to 459 ft-msl, at least 40 feet below the excavation grades within the expansion area. As demonstrated within the stability models included in

Appendix IIIE-A, failure surfaces for the modeling do not penetrate into or below this lower Aquifer D. Note also that the upper aquifers are discontinuous, and not clearly represented in all geological logs.

For the stability modeling, a generalized groundwater aquifer was input into the models above elevation 545 ft-msl, with an aquifer thickness of 20 feet. The lower Aquifer D was not considered for the stability modeling. Groundwater elevations considered for the stability analylsis are presented on the Groundwater Contour Map (Figure IIIG-D-4) included in Appendix IIIG-Geology Report.

Lastly, it is worth noting that review of the geological logs and laboratory data indicate that the clay zones upon which the upper groundwater zones are perched (as described above) are not saturated. Additionally, these perched zones are subject to draining and dewatering from both the landfill underdrain system as well as the sideslope excavations further alleviating the influence of groundwater on slope stability. Based on this information, WCG concludes that incorporating the 20-foot-thick groundwater zone into the slope stability modeling is conservative.

Table 5-1
Summary of Material Weight and Strength Parameters Used in the Slope Stability Analysis

Strength Parameters					Comments
Final Cover System					The final cover system includes the erosion layer, drainage geocomposite (single-sided on top slopes and double-sided on 4H:1V sideslopes), geomembrane liner (smooth or textured on topslopes and textured on 4H:1V sideslopes), and compacted clay infiltration layer. An infinite stability analysis was performed to establish the minimum interface strength requirements for each layer of the final cover system. The minimum interface strength requirements specified are used for the veneer stability analysis in Appendix IIIE-A-4. For the rotational global stability analysis, the final cover system is modeled as a single layer and the strength parameters represent the compacted clay infiltration layer and the erosion layer. The two geosynthetic layers (i.e., geomembrane and geocomposite) are not included in the global analysis because they provide a negligible contribution to the forces that are resisting movement. The strength values selected for the final cover system represent strength values typically used in the industry and these same strength values have been used in various permit applications approved by TCEQ. The global stability analysis for rotational failure analysis uses the soil material strength parameters (i.e., cohesion of 100 lb/ft^2 and a friction angle of 16 degrees). The global stability analysis is included in Appendix IIIE-A-3. The interface slope stability analysis for the final cover system was performed using an infinite slope stability analysis procedure by Duncan, Buchianani, and De Wet. The purpose of this analysis was to show that the final cover components that are placed on top of each other, such as a geomembrane and compacted clay layer (or geomembrane and geocomposite), will not experience sliding failure due to the lack of strength between these components. The interface strength parameters shown are based on compacted clay internal on the sideslope and smooth geomembrane and compacted clay on the top deck. The interface strength parameters were developed from Geosynthetic Research Institute (GRI) publications (e.g., "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces" by George R. Koerner, GRI, Folsom, PA, June 14, 2005). Although the strength parameters (i.e., adhesion and interface friction) used for the application were selected based on published data, it should be noted that these strength parameters will also be tested and verified at the time of each final cover construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design (as discussed in Appendix IIIE-A). As noted in Appendix IIIE-A, the strength parameters listed are for the weakest interface (or internal) to provide for a conservative design.
Soil Material Strength Parameters			**Interface Strength Parameters**		
Cohesion (lb/ft^2)	**Friction Angle (degrees)**	**Unit Weight (lb/ft^3)**	**Adhesion (lb/ft^2)**	**Friction Angle (degrees)**	
200	16	108	Refer to Appendix IIIE-A-4 for analysis.		
Solid Waste					As noted in Appendix IIIE-A, the strength parameters for solid waste were based on information contained in the following references: Pagotto and Rimoldi (1987), Landva and Clark (1990), and Richardson and Reynolds (1991) and Kavazanjian, et al. (1995). These sources list cohesion and friction angle values that range from 210 lb/ft^2 to 605 lb/ft^2 and 18° (for residual strength or large displacement for direct shear test which requires a factor of safety of 1.1) to 43°, respectively. The selected strength values are selected to represent peak strength for MSW. The unit weight of waste used for stability analyses is consistent with numerous analyses and permit amendment applications in Texas.
Material Strength Parameters			**Interface Strength Parameters**		
Cohesion (lb/ft^2)	**Friction Angle (degrees)**	**Unit Weight (lb/ft^3)**	**Adhesion[3] (lb/ft^2)**	**Friction Angle (degrees)**	
For φ_p < 625 psf C = 500 psf	0	65	Same as Material Strength Values[2].		
For φ_p > 625 psf C = 0	33				

Table 5-1 (Continued)
Summary of Material Weight and Strength Parameters Used in the Slope Stability Analysis

Strength Parameters				Comments
Liner System				The liner system includes a 2-foot-thick compacted clay layer, 60-mil geomembrane (textured geomembrane on the floor of the landfill and textured on the 3H:1V sideslopes), drainage geocomposite (single-sided on floor grades and double-sided on 3H:IV sideslopes), and a 2-foot-thick protective cover soil layer. Alternatively, a GCL may (likely) be substituted for the 2-foot-thick compacted clay layer. This system is modeled as a single 2-foot-thick layer for the global stability analysis. In addition, both a translational and an infinite stability analysis were performed to establish the minimum interface strength requirements for each layer of the liner system. The minimum interface strength requirements are specified in Appendix IIIE-A-5. For the rotational global stability analysis, the liner system is also modeled as a single layer with a 2-foot thickness. The strength values selected for the liner system represent strength values typically used in the industry and these same strength values have been used in various permit applications approved by TCEQ. Duncan and Wright (2005) provides a comprehensive discussion regarding strength parameters for a liner system. In Chapter 5 – Shear Strengths of Soil and Municipal Solid Waste, a significant amount of data are presented and evaluated for compacted clay liners. The results indicate that the lowest cohesion value for compacted cohesive soils is 9 kPa (187 lb/ft^2) and the lowest reported friction angle value is 19 degrees. Review of the Geosynthetic Research Institute (GRI) publication "Report #30, Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces (Koerner et.al., 2005)" reports interface values for membranes and geocomposite drainage layers and is an accepted technical resource for use in stability modeling. The values presented in GRI Report #30 for textured membrane and single and double-sided geocomposites were determined to be the critical interfaces for stability modeling and were incorporated into the analysis. Soil and geosynthetic properties used in the stability analysis are subject to verification at the time of each liner construction. Section 2.4.3 in Appendix IIID–LQCP and Appendix IIIE-4-A-5 includes the material strength tests required for soil and geosynthetic used for liner construction. The global stability analyses are included in Appendices IIIE-A-2 and IIIE-A-3. The interface slope stability analysis, which is performed using an infinite slope stability analysis procedure by Duncan, Buchianani, and De Wet for the liner system, was developed to show that certain landfill components that are placed on top of each other, such as a geomembrane and compacted clay layer or geomembrane and GCL will not experience sliding failure due to the lack of interface strength between these components. These strength values represent the interfaces with the lowest strength at the sideslopes (refer to Appendix IIIE-A-4 for the complete evaluation of interfaces that will occur for the liner system 3H:1V sideslope and the bottom liner interface strength value is obtained from the document referenced in this paragraph). The strength parameters were developed using information from GRI Report #30. Although the strength parameters (i.e., adhesion and interface friction) used for the application were selected based on published data, it should be noted that these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet the minimum strength parameters from this analysis (refer to Appendix IIIE-A-5). The translational slope stability analysis was performed using simplified Janbu Method using the Rankine Blocks. This analysis is similar to the interface slope stability analysis discussed above. The purpose of this analysis is to test the critical interfaces under a variety of loading conditions (refer to Appendices IIIE-A-2 and IIIE-A-3 for more information – i.e., the loading conditions reflect different landfill configurations). SLIDE2 is also used for this analysis. However, for the translational analysis, the liner system strength parameters are modified to reflect the interface strength parameters. The translational stability analysis uses modified liner system strength parameters to reflect the interface strength parameters. As noted above, these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design.
Material Application		Interface Strength Parameters[1, 3]		
Material	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)	
Liner System (Peak Stress)				
Floor (TGM/SSGC)	108	0	13	
3H:1V Sideslopes (TGM/DSGC)	108	200	19	
Liner System (Residual Stress)				
Floor (TGM/SSGC)	108	0	10	
3H:1V Sideslopes (TGM/DSGC)	108	120	10	

Table 5-1 (Continued)
Summary of Material Weight and Strength Parameters Used in the Slope Stability Analysis

Strength Parameters[3]					Comments
Interbedded Sandy Clays and Silts, Silty Clayey Sands					Refer to Section 3.2 of this appendix for additional discussion regarding site stratigraphy.
Material Strength Parameters			**Interface Strength Parameters**		
Cohesion (lb/ft^2)	**Friction Angle (degrees)**	**Unit Weight (lb/ft^3)**	**Adhesion (lb/ft^2)**	**Friction Angle (degrees)**	
Effective 800 Total 1000	Effective 19 Total 14	115 130 (SAT)	This layer associated with the confining units and non-aquifer bearing zones within the subsurface strata, as determined from review of geological cross-sections developed for the site.		
Sands					Refer to Section 3.2 of this appendix for additional discussion regarding site stratigraphy.
Material Strength Parameters			**Interface Strength Parameters**		
Cohesion (lb/ft^2)	**Friction Angle (degrees)**	**Unit Weight (lb/ft^3)**	**Adhesion (lb/ft^2)**	**Friction Angle (degrees)**	
Effective 200 Total 500	Effective 28 Total 18	120 135 (SAT)	This layer associated with the higher permeability sands and silty and clayey sands generally associated with the water bearing portion of the formation, as determined from review of geological cross-sections developed for the site.		

Notes:
1. Liners on the sideslopes and floor grades are listed separately due to different strength characteristics for textured geomembrane/single-sided geocomposites and textured geomembrane /double-sided geocomposite interfaces.
2. Interface strength values for waste have conservatively been set as same values assumed for material strength. However, interface strength of waste with adjacent soil layers (i.e., protective cover soils or intermediate cover soils only as waste does not come into direct contact with geosynthetic or compacted clay component of liner systems or final cover) vary greatly, but can include the waste blending into or "biting" into the soil during both placement and shoving associated with compaction of the waste, or raveling and mixing of soil and waste at the intermediate cover/waste interface during intermediate cover soil placement.
3. Refer to Table IIIE-A-1 for strength parameters.

5.5 Results of Stability Analysis

5.5.1 Stability Analysis Using SLIDE2

The results of the stability analyses using SLIDE2 computer program indicate that the proposed excavation, interim, and final configuration slopes are stable under the conditions analyzed. Tables 5-2 through 5-4 summarize the results of the stability analyses for the landfill slopes and compares the calculated factor of safety to the recommended minimum factor of safety. The recommended minimum factors of safety for the conditions analyzed were determined using recommendations from the Corps of Engineers "Design and Construction of Levees" manual (EM 1110-2-1913) and the EPA's "Technical Guidance Manual for Design of Solid Waste Disposal Facilities," as 1.3 for short-term slope stability (interior excavation and interim slopes) and 1.5 for long-term slope stability (exterior excavation and final cover slopes). The minimum factor of safety for residual stress analysis is 1.1 (Noting that TCEQ's Draft Technical Guideline No. 3 recommends a factor of safety of 1.0.).

Table 5-2
Summary of Slope Stability Analyses
for the Excavation Configurations

Analyzed Section-Run	Failure Type	Minimum Factor of Safety Generated[1]		Factor of Safety Acceptable
		Effective Stress	Total Stress	
		1.5/1.3[2]	1.3	
Excavation Slope A-1 (Exterior)	Bishop-Circular	1.90	1.62	YES
Excavation Slope B-1 (Interior)	Bishop-Circular	2.40	2.14	YES
Excavation Slope B-1 (Exterior)	Bishop-Circular	4.40	4.58	YES
Excavation Slope C-1 (Interior)	Bishop-Circular	1.87	1.61	YES

1 Recommended Minimum Factor of Safety for long-term stability analysis using effective stress is 1.5 and short-term stability analysis using total stress is 1.3.
2 A minimum factor of safety for interior excavation slopes is 1.3 for both effective and total stress.

Table 5-3
Summary of Slope Stability Analysis for Interim Configuration

Slope Designation	Method of Analysis	Minimum Factor of Safety Generated[1]		Factor of Safety Acceptable	
		Effective/ Peak Stress[2]	Total/Residual Stress[3]	Effective	Total
		1.3	1.3/1.1[4]		
Interim Fill Slope D-1	Bishop-Circular	2.0	1.97	YES	YES
Interim Fill Slope D-2	Rankine-Block	1.40	1.24	YES	YES

[1] Long-term factor of safety for temporary slopes is 1.3.
[2] Peak stress for Rankine-Block only.
[3] Residual stress for Rankine-Block only.
[4] An acceptable Factor of Safety for residual stress is 1.1.

Table 5-4
Summary of Slope Stability Analysis for Final Landfill Configurations

Slope Designation	Method of Analysis	Minimum Factor of Safety Generated[1,2]		Factor of Safety Acceptable	
		Effective/ Peak Stress	Total/Residual Stress	Effective	Total
		1.5	1.3/1.1		
Final Fill Slope E-1	Bishop-Circular	2.40	2.48	YES	YES
Final Fill Slope E-2	Rankine-Block	1.75	1.50	YES	YES
Final Fill Slope F-1	Bishop-Circular	2.61	2.41	YES	YES
Final Fill Slope F-2	Rankine-Block	1.76	1.25	YES	YES

[1] Recommended Minimum Factor of Safety for long-term (final cover) stability analysis using effective stress is 1.5 and short-term (final cover) stability analysis using total stress is 1.3.
[2] Recommended Minimum Factor of Safety for stability analysis using peak stress is 1.5 and residual stress is 1.1.
[3] Residual stress for Rankine-Block only.

Computer-generated slope stability analysis output is included in Appendix IIIE-A. The minimum calculated effective stress factor of safety for the closed condition is 1.75, which is greater than the recommended minimum factor of safety of 1.5 for long-term slope stability.

5.5.2 Infinite Slope Stability Analysis

Infinite slope stability analysis for the bottom liner and final cover systems has been included in this design in addition to the block method analysis discussed in the previous section. These calculations are presented in Appendix IIIE-A-4. The infinite stability analyses address anchor trench design, stability of cover and drainage material on anchored geosynthetics, and shear forces within the liner system. The infinite slope stability analysis for the final cover system presented in Appendix IIIE-A-4 addresses both the prescriptive final cover as well as the components of the Alternative Final Cover addressed in Appendix IIIJ of this application.

The infinite final cover slope stability analysis addresses the shear forces within the final cover system. As demonstrated in Appendix IIIE-A-4, the liner and cover systems are structurally stable using the strength parameters shown, which will be verified during each construction event. Prior to each construction event for liner and final cover, the POR will perform interface strength testing using the actual material that will be used for each construction event to demonstrate the interfaces comply with the minimum values set forth in the Interface Shear Strength Conformance Test Requirement presented in Appendix IIIE-A-5. Alternatively, stack testing may be performed also as described in Appendix IIIE-A-5, and as described in the following section.

5.5.3 Bottom Liner Interface Shear Strength Conformance Testing

Prior to each construction event, interface shear strength conformance testing will be required for the specific geosynthetic and soil liner components to be incorporated into the project. The interface shear strength conformance testing requirements have been established for the project based on stability analyses performed for the expansion. The description of the interface shear strength conformance testing requirements and supporting stability analyses is presented in Appendix IIIE-A-5. As discussed in the appendix, the conformance testing requirements are applicable to both laboratory stack testing and single interface testing results and will be incorporated into the Geosynthetic Liner Evaluation Report (GLER) prepared for the respective construction event.

6 SETTLEMENT, STRAIN, AND HEAVE ANALYSIS

6.1 General

The purpose of the settlement and heave analysis is to demonstrate that the bottom liner system will not be adversely impacted by waste-induced foundation settlement. The settlement analysis also addresses the settlement of the final cover system to demonstrate that the proposed final cover is designed to withstand the potential strain induced by waste settlement and long-term biodegradation.

Settlement of the liner system will occur due to consolidation of the foundation materials from the weight of the landfill components (i.e., protective cover, solid waste and daily cover, and final cover systems). Laboratory consolidation testing was performed on the clay stratum existing at the site, although the bottom of the landfill (in expansion area) was assumed to be underlain by sands, clayey sands and silty sands. Consolidation testing was not performed of the sand strata as the collection of intact sand samples for testing is not reasonably possible.

Settlement of the final cover system will occur primarily due to consolidation within the solid waste. Total consolidation of final cover consists of primary and secondary consolidation of deposited waste. Appendix IIIE-B includes foundation settlement analyses and heave analysis, and final cover settlement analysis.

6.2 Foundation/Bottom Liner Settlement and Strain

The Foundation/Bottom Liner Settlement Analysis is presented in Appendix IIIE-B-1. Foundation settlement potential has been assessed using estimates of consolidation properties for sands and clayey sands, the primary formation underlying the constructed cells.

Settlement calculations were performed using SETTLE3, a computer-based model developed by RocScience, Inc. (2023). Input parameters include surfaces representing the subsurface strata, vertical loads representing the waste placed in the cell, and the settlement characteristics of the subsurface strata (from laboratory consolidation testing and program-embedded assumptions based on material properties). The SETTLE3 model creates an isopach of the settlement of the bottom liner system, which then can be used to calculate strain within the bottom liner system components.

The analysis is performed by creating a horizontal plane within the SETTLE3 program, with subsurface data input from available boring logs that has been normalized to the excavation grades (i.e., grades below the bottom liner system) designed for the landfill. Thus, the horizontal plane within the model represents the soil conditions beneath the excavation grade contours. Vertical fill loads are then calculated by subtracting the final landfill elevation from the excavation grades, and then multiplying the fill height by the unit weight assumed at each fill point. Unit weight values are adjusted based on the total waste thickness and assume that deeper waste fill heights result in higher waste densities and associated consolidation pressures.

For the analysis, a conservative approach of disregarding pre-consolidation stresses was used, resulting in the model calculating settlement values exceeding the actual anticipated settlement values. This is a conservative approach in that it results in greater settlement at each analysis point when compared to analyses performed using an assumed or calculated pre-consolidation stress value. The results of the analyses are presented in Appendix IIIE-B. As demonstrated in Appendix IIIE-B, even with this more conservative approach the settlement at the site is negligible and will not adversely affect the performance of the leachate collection systems and will not result in detrimental strain on the liner system components.

6.3 Final Cover Settlement and Strain

The Final Cover Settlement Analysis is presented in Appendix IIIE-B-2. Landfill final cover settlement occurs due to settlement of foundation soils and the settlement of waste materials. In general, foundation settlement is insignificant in comparison to the settlement of deposited waste. Waste settlement consists of primary and secondary settlement.

Settlement of solid waste generally begins rapidly as the waste load is placed and continues to occur for long periods of time after the initial placement. Initially, municipal solid waste will undergo primary settlement due to its own weight, final cover, equipment, etc. Primary settlement occurs quickly, generally within the first month after loading. Therefore, the weight of the final cover system is the only remaining factor that contributes to primary consolidation. By the time the construction of the final cover is complete, settlement of the waste due to the weight of the final cover will be complete.

Secondary settlement continues at substantial rates for periods of time well beyond primary settlement. It is a combination of mechanical secondary compression, physico-chemical reaction, and bio-chemical decay.

A strain analysis has been incorporated into the final cover settlement analysis presented in Appendix IIIE-B-2. The purpose of the settlement and strain analysis is to demonstrate that the final cover system will be stable as designed and maintain positive drainage. If it is considered that the waste settlement is uniform, then the sideslopes are expected to maintain positive drainage. Based on the estimates of settlement for the maximum waste thickness (where maximum waste settlement is expected to occur on the top deck of the landfill) and minimum waste thickness (where minimum settlement is expected to occur on the top deck of the landfill), the landfill final cover will be subject to a (compressive) strain of 0.36 percent. That is less than the allowable strain for the final cover components. A strain demonstration in Appendix IIIE-B-2 shows that the top deck areas of the final cover will be stable and maintain positive drainage after settlement.

6.4 Foundation Heave

The foundation heave analysis is presented in Appendix IIIE-B-3. As shown, the calculations were performed using the standard consolidation theory for soils and the recompression index assumed from available consolidation tests of clay soils at the site. The analysis is highly conservative in that the sands and clayey sands within the foundation are not likely to heave significantly during unloading.

Using a maximum excavation depth of approximately 64 feet (existing ground elevation minus bottom of excavation at a given location), a heave of approximately 18 inches was conservatively estimated. The depth of floor grade excavation for each individual sector (liner area draining to an LCS sump) is generally uniform (i.e., depth of soil to be removed from the floor grades does not change drastically within a given sector). Where the excavation depth is less, heave will also be less and therefore negligible. These calculations are included in Appendix IIIE-B-3. Heave will occur soon after excavation (before and during liner construction) and will not adversely affect the performance of the liner system.

7 CONCLUSIONS AND RECOMMENDATIONS

This geotechnical analysis has been developed using (1) various geotechnical data obtained from field and laboratory testing performed on the soil samples recovered at the site; (2) general soil stratigraphy of the project area; and (3) known geotechnical characteristics of the founding geological formation, of solid waste, of geosynthetic materials commonly used for landfill development, and of soils used for various components of landfills. It is concluded, based on this geotechnical analysis, that the proposed landfill and its components (e.g., leachate collection system, liner systems, cover systems, excavation and interim fill slopes) will be geotechnically stable and will function as designed. The following summarizes various findings of the geotechnical analysis.

- Geotechnical laboratory testing was performed in accordance with industry practice and recognized procedures (e.g., ASTM standards).
- Stability of the proposed landfill excavation slopes, constructed liner slopes, interim fill slopes and the final cover are acceptable as designed (see Appendix IIIE-A).
- Stability of the liner and final cover system components is acceptable as designed (see Appendix IIIE-A).
- Foundation settlement after filling is expected to be negligible and within the strain limits of the liner system (refer to Appendix IIIE-B). Settlement of the liner system will not adversely affect the liner system, and the liner system will perform as designed (i.e., maintain positive drainage to the LCS sumps).
- Settlement of the final cover system will not adversely affect the final cover system, and the final cover system will function as designed (refer to Appendix IIIE-B).

APPENDIX IIIE-A

SLOPE STABILITY ANALYSIS

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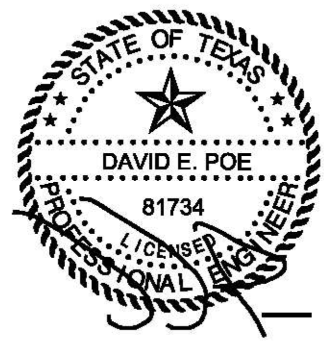
PROFESSIONAL ENGINEER

05/20/2024

CONTENTS



05/20/2024

INTRODUCTION

This appendix includes the slope stability analysis for the landfill during various phases of the site development and the final closure. General slope stability for the excavation and interim and closed conditions were evaluated by using the SLIDE2 computer program, as developed by RocScience, Inc. (2023). The Simplified Bishop method was used for circular failure surfaces, and the Simplified Janbu method using Rankine Block was used for the translational (block) slope stability analysis. Infinite slope stability has also been analyzed for the bottom liner, overliner, and final cover system. Soil profiles analyzed for each configuration for the slope stability analysis are provided in the sub-appendices, along with SLIDE2 computer output files as applicable. The stability analysis for the site is provided in the following five appendices.

- Appendix IIIE-A-1 includes the slope stability analysis for the excavated landfill condition.
- Appendix IIIE-A-2 includes the slope stability analysis for the interim slope condition.
- Appendix IIIE-A-3 includes the slope stability analysis of the final closure configuration.
- Appendix IIIE-A-4 includes the infinite slope stability evaluation.
- Appendix IIIE-A-5 includes the interface shear strength conformance testing requirements (for use during future cell bottom liner design and construction).

Required:

A. Evaluate the slope stability of the proposed landfill configuration including excavation grades, interim fill slopes, and final closure condition slopes.

B. Evaluate the veneer stability of the bottom liner and final cover systems. Analysis is performed by the Infinite Slope Analysis Method.

C. After completing the analysis of the selected sections above using the weakest liner interface for each condition, the worst case section (i.e., the section with the lowest resulting factors of safety) was then re-analyzed to determine the minimum required strength parameters to meet the minimum required factors of safety (for block failure along the liner system interfaces). These strength values will then be used in material specification and conformance testing during future bottom liner and overliner construction projects. For this project, Section F-F was selected as the worst case condition. The results of the conformance testing analysis and the Geosynthetic Conformance Testing Requirements are presented in Appendix IIIE-A-5.

For this slope stability analysis, the analysis description, input parameters, analysis section plans, and the sections analyzed (with analysis results) are presented in Appendix IIIE-A. SLIDE2 computer model output files are presented in Appedices IIIE-A-1 (Excavation Grades), IIIE-A-2 (Interim Conditions) and IIIE-A-3 (Final Closure Conditions). Infinite slope stability analyses are presented in Appendix IIIE-A-4.

Given:

1. Site plans showing the sections analyzed for this analysis are presented on Sheets IIIE-A-7 and IIIE-A-8.

2. Modeling parameters were derived from field and laboratory testing, and are summarized in Table IIIE-A-1, below. The results of field and laboratory testing are discussed in Section 5.5 of Appendix IIIE. Assumptions regarding waste density are discussed in Appendix IIIE, Table 5-1.

3. The proposed bottom liner system for the landfill will consist of (from the bottom up) 2-foot-thick compacted clay liner ($k < 1x10^{-7}$ cm/s), 60-mil HDPE geomembrane, geotextile-geonet composite drainage layer, and 2-foot-thick soil protective cover. A GCL may be substituted for the clay liner component. Infinite stability analysis results for both the GCL and the clay liner option of the bottom liner system are presented in Appendix IIIE-A-4.

4. The proposed final cover system for the landfill will consist of (from the bottom up) an infiltration layer, 40-mil LLDPE geomembrane, geotextile-geonet drainage layer, and 1-foot-thick erosion layer. The infiltration layer may be comprised of 18-inch thick clay layer or GCL. Infinite stability analysis results for the final cover system are presented in Appendix IIIE-A-4.

5. The bottom liner sysem was analyzed for stability as a single (thickened) layer with assigned strength parameters of the weakest component of the proposed composite liner system.

Method:

A. Evaluate the slope stability of the proposed landfill configuration including excavation grades, interim fill slopes, and final landfill slopes.

1. Determine critical excavation, interim and final landfill configuration slopes in the proposed design.

2. Select a soil profile for each critical section using available boring logs and geologic cross sections near each section. Information for this effort was derived from Appendix IIIG-Geology Report.

3. Select material properties using unit weights and strength parameters for the proposed sections (See Table IIIE-A-1, below).

4. Perform slope stability analyses:
    a. Analyze the <u>excavation and exterior liner slopes</u> using SLIDE2 computer model and the simplified Bishop method of circular failure surfaces. Analyses were performed for both effective (drained) stress conditions and total (undrained) stress conditions. The effective stress conditions represent long-term conditions, and the total stress conditions represent short-term conditions. Analysis section plans and analysis sections are presented as Sheets IIIE-A-7 through 13, and the SLIDE2 output files and results are presented in Appendix IIIE-A-1.

    b. Analyze the <u>interim and final closure condition slopes</u> using SLIDE2 computer model and the simplified Bishop method of circular failure surfaces and the Bishops method for block failure surfaces at the bottom liner interface. Circular failure plane analyses were performed for total (undrained) stress and effective (drained, or long term) stress conditions. The effective stress conditions represent long-term conditions, and the total stress conditions represent short-term conditions. Analysis section plans and analysis sections are presented as Sheets IIIE-A-7 through IIIE-A-13, and the SLIDE2 output files and results are presented in Appendices IIIE-A-2 (interim conditions) and IIIE-A-3 (final closure conditions).

5. Using the worst case section analyzed for the stability analysis above (Section F-F), develop the minimum strength parameters required to obtain the minimum required stability factors of safety (for peak and residual strength of block failures along the geosynthetic liner interfaces). This information will be used during future conformance testing during landfill cell design and construction to qualify selected geosynthetic materials. The Conformance Testing Requirements worksheets are provided in Appendix IIIE-A-5. Conformance Testing Requirements are provided for both cell bottom and sideslope (3H:1V) conditions.

6. Evaluate the stability of the proposed bottom liner and the final cover system using infinite slope stability analysis. The results of the infinite slope stability analyses are presented in Appendix IIIE-A-4.

    a. Verify that the tensile stress in the liner system will be less than the yield stress by using Koerner's method (reference 4) for determination of shear stress in liner systems considering cohesion/adhesion forces.
    b. Provide anchor trench design considering pullout of the geomembrane (incorporated into the bottom liner infinite slope stability analysis).
    c. Use Duncan and Buchignani's method for infinite stability analyses to evaluate the internal stability of the liner systems.

References:

1. Duncan, J.M. and Buchignani, A.L., *An Engineering Manual for Slope Stability Studies,* Department of Civil Engineering-University of California-Berkeley, 1975.
2. TRI, Interface Friction/Direct Shear Testing & Slope Stability Issues. Short Course, November 12-13, 1998. Austin, Texas.
3. US Army Corps of Engineers, *Slope Stability* , Engineering and Design Manual, EM 1110-2-1902, October 31, 2003.
4. Koerner, Robert M., *Designing with Geosynthetics* , 5th Ed., Prentice-Hall, Inc., 2005.
5. SLIDE 2 (computer program for slope stability analyses), Rocscience Inc.
6. Das, Braja M., *Principles of Geotechnical Engineering* , 5th Ed., Brooks/Cole, 2002.
7. Gilbert, Robert B, *Peak Versus Residual Strength for Waste Containment Systems,* Proceedings the 15th GRI Conference on Hot Topics in Geosynthetics-II (Peak/Residual; RECMs; Installation; Concerns)
8. Cetco Lining Technologies, Laboratory Data Reports, Bentomat Direct Shear Testing Summary, Summary of Bentomat Direct Shear Test Data Internal, Revised 08/02
9. Bouzza, A., Zornberg, J.G., and Adam, D. *Geosynthetics in Waste Containment Facilities: Recent Advances* , 2002.

Solution:

A. Slope stability analyses of the proposed slopes.

1. The locations of the critical sections selected for the stability analysis for the proposed slopes are shown on Sheets IIIE-A-7 and IIIE-A-8. Sections analyzed are also shown with the most critical failure surfaces for each of the analyses performed and the resulting factors of safety.

2. The soil profile used for each analysis was based on boring log data from previous site investigations from the undeveloped area of the site and the geologic cross sections (see Appendix IIIG-Geology Report). Generalized soil profiles for the site also are shown in Appendix IIIG-Geology Report of this application.

3. A summary table (IIIE-A-1) presents the assumed material weight and strength properties for the analyses performed for this appendix.

4. The material weight and strength parameter determination for each material type was based on laboratory testing results (Atterberg limits, natural moisture content, unit weight, percent finer than #200 sieve, and Standard Proctor), industry references and engineering judgment based on previous experience with similar materials. Laboratory testing results from the 2023 investigations are included in Appendix IIIE-C.

5. The output from the slope stability analyses is summarized in Section 5.5, Appendix IIIE.

B. Infinite slope stability of the proposed bottom liner and final cover systems.

1. The anchor trench design for bottom liner installations is provided on Sheets IIIE-A-4-7 and 8.

2. Infinite slope stability analysis of the bottom liner system is provided on Sheets IIIE-A-4-9 through 12.

Conclusion: Based on the slope stability analyses provided in this Appendix, the proposed critical slopes for the excavation, interim and final cover conditions have adquate factors of safety to be considered stable. In addition, the infinite stability analysis demonstrates that the proposed liner system has adequate factors of safety to be considered stable. Lastly, this appendix presents the minimum strength parameters to be used during future cell and closure designs in selecting the appropriate liner and cover system components and geosynthetics.

TABLE IIIE-A-1 SLOPE STABILITY MODEL PARAMETER SELECTION

GEOLOGY/COMPACTED FILL ASSUMPTIONS

			Effective		Total	
Layer	**Moist Unit Weight (pcf)**	**Saturated Unit Weight (pcf)**	**c (psf)**	**φ**	**c (psf)**	**φ**
Interbedded sandy clays and silts, silty clayey sands	115	130	800	19	1000	14
Sand (silty, clayey)	120	135	200	28	500	18
Compacted Fill	123	132	800	19	1000	14
Clay Internal	108	115	100	18	500	10

GEOSYNTHETIC INTERFACE ASSUMPTIONS

			Peak		Residual	
Layer	**Moist Unit Weight (pcf)**	**Saturated Unit Weight (pcf)**	**c (psf)**	**φ**	**c (psf)**	**φ**
PC-SSGC or DSGC	120	125	200	20	270	15
TGM-SSGC	108	115	0	13	0	10
SGM-SSGC (NOT USED)	108	115	0	11	0	9
TGM-DSGC	108	115	200	19	120	10
TGM-GCL	108	115	850	25	400	10
SGM-GCL	108	115	0	15	0	12
GCL-Subgrade	108	115	500	22	0	12
GCL Internal Reinforced	108	115	800	18	380	11
TGM-CCL	108	115	210	18	50	14
SGM-CCL	108	115	0	22	50	18
Soil-DDGC	108	115	200	20	150	10

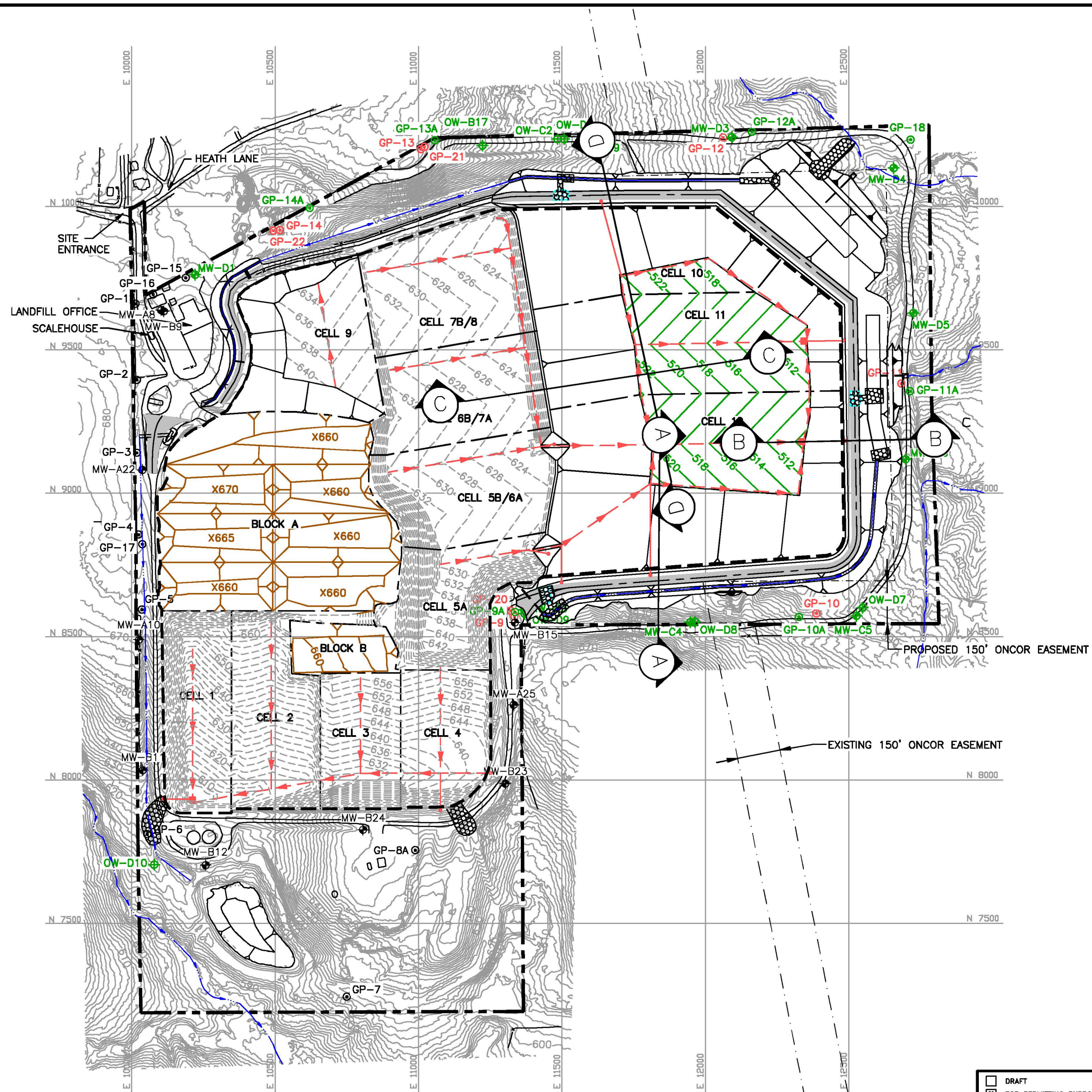
1. Unit weights of geosynthetics assumed equal to unit weight of compacted clay liner.
2. Weakest values in above table incorporated into interface block analyses.

WASTE

Layer	**Moist Unit Weight (pcf)**	**Saturated Unit Weight (pcf)**	**c (psf)**	**φ**
Waste (0-625 psf)	65	65	500	0
Waste (>625 pcf)	65	65	0	33

Legend

PC = Protective Cover
CCL = Compacted Clay Liner
TGM = Textured Geomembrane
SMG = Smooth Geomembrane (not used)
SSGC = Single Sided Geocomposite
DSGC = Double Sided Geocomposite
GCL = Geosynthetic Clay Liner
c = Cohesion (psf)
phi = Angle of Internal Friction (degrees)



SCALE IN FEET 0 200 400

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DAVID E. POE
81734
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PROFESSIONAL ENGINEER
05/20/2024

LEGEND

PERMIT BOUNDARY
LIMIT OF WASTE
CELL BOUNDARY
EXISTING EASEMENT
PROPOSED EASEMENT
E 10500 SITE GRID
610 EXISTING CONTOUR (SEE NOTE 1)
516 TOP OF LINER CONTOUR
630 AS-BUILT TOP OF LINER
X660 APPROXIMATE TOP OF PRE-SUBTITLE D LINER (SEE NOTE 8)
CHANNEL CENTERLINE
LEACHATE COLLECTION PIPE
LEACHATE COLLECTION SUMP
LEACHATE RISER PIPE
MW-10 EXISTING GROUNDWATER MONITORING WELL
MW-D1 PROPOSED GROUNDWATER MONITORING WELL
OW-D7 PROPOSED OBSERVATION WELL
GP-1 EXISTING GAS PROBE
GP-10A PROPOSED GAS PROBE
A — A SECTION LOCATION

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022. THE GRID SYSTEM IS BASED ON A SITE GRID SYSTEM. ELEVATIONS ARE BASED ON NAVD 88.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.
3. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY STANGER SURVEYING COMPANY, DATED APRIL 1995.
4. INTERIM SLOPE SECTION (D-D) DEVELOPED FROM DRAWING I/IIA.6.

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-76-11
CAD: SHEET IIIE-A-7-SLOPE STABILITY.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
SLOPE STABILITY ANALYSIS
SECTION LOCATIONS 1

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

WWW.WCGRP.COM
SHEET IIIE-A-7

O:\0120\76\EXPANSION 2023\PART III\IIIE\IIIE-A-7 SLOPE STABILITY SECTION LOCATIONS 1.dwg, byoung, 1:2

N

0 200 400
SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	LIMIT OF WASTE
	EXISTING EASEMENT
	PROPOSED EASEMENT
E 10500	SITE GRID
610	EXISTING CONTOUR (SEE NOTE 1)
624	PROPOSED FINAL COVER CONTOUR
	DRAINAGE SWALE
	DRAINAGE LETDOWN
	CHANNEL CENTERLINE
MW-10	EXISTING GROUNDWATER MONITORING WELL
MW-D1	PROPOSED GROUNDWATER MONITORING WELL
OW-D7	PROPOSED OBSERVATION WELL
GP-1	EXISTING GAS PROBE
GP-10A	PROPOSED GAS PROBE
F F	SECTION LOCATION

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PROFESSIONAL ENGINEER
05/20/2024

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. MAXIMUM FINAL COVER ELEVATION IS 759.0 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 757.0 FT-MSL.
3. TYPICAL SIDESLOPES ARE 4H:1V. TYPICAL TOPSLOPE IS 5%.
4. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY LANDES & ASSOCIATES, INC. DATED MARCH 2011.

- [ ] DRAFT
- [x] FOR PERMITTING PURPOSES ONLY
- [ ] ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-76-11
CAD: SHEET IIIE-A-8-SLOPE ANALYSIS.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
SLOPE STABILITY ANALYSIS
SECTION LOCATIONS 2

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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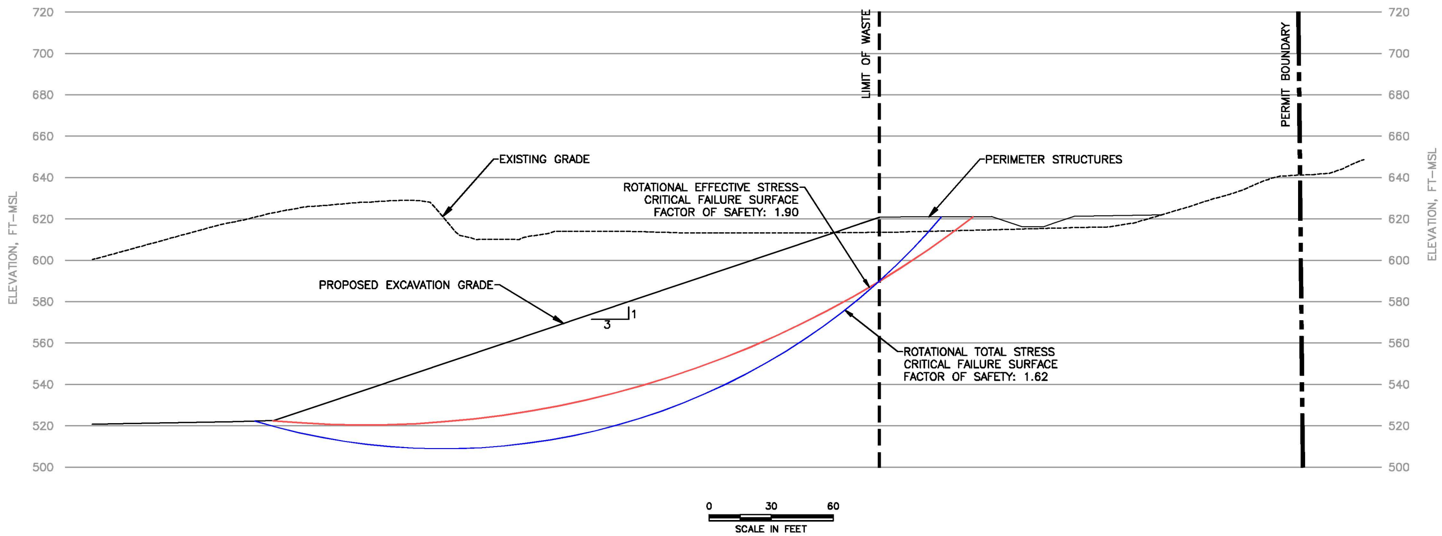
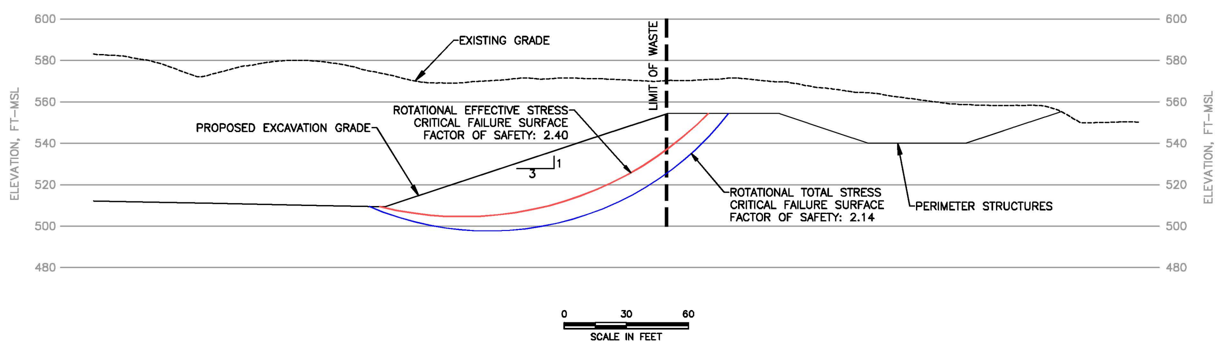
SHEET IIIE-A-8

O:\0120\76\EXPANSION 2023\PART III\IIIE\IIIE-A-8 SLOPE STABILITY SECTION LOCATIONS 2.dwg, byoung, 1:2

EXCAVATION SECTION A–A – INTERIOR



EXCAVATION SECTION B–B – INTERIOR



STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER
05/20/2024

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DATE: 05/2024
FILE: 0120–76–11
CAD: SHEET IIIE–A–9.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
EXCAVATION SLOPE STABILITY SECTIONS

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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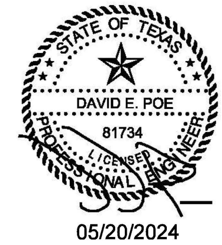
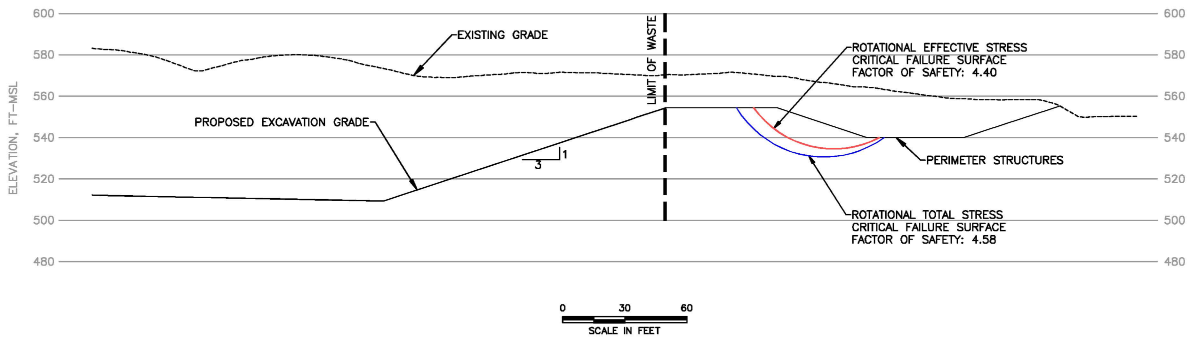
SHEET IIIE–A–9

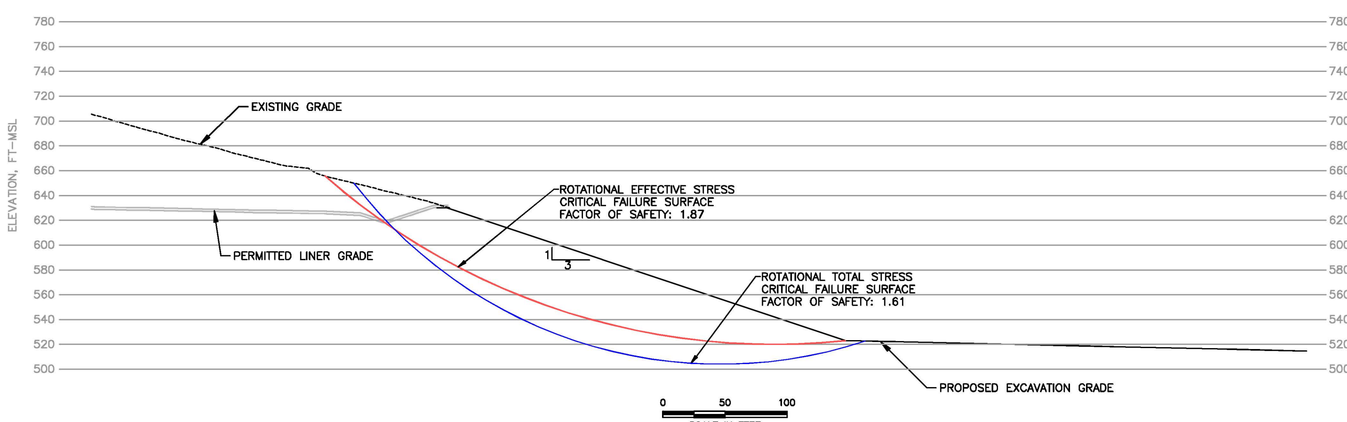
O:\0120\76\EXPANSION 2023\PART III\IIIE\SHEET IIIE–A–9.dwg, byoung, 1:2



EXCAVATION SECTION B–B – EXTERIOR



EXCAVATION SECTION C–C – INTERIOR



☐ DRAFT ☒ FOR PERMITTING PURPOSES ONLY ☐ ISSUED FOR CONSTRUCTION	PREPARED FOR PINE HILL FARMS LANDFILL TX, LP	MAJOR PERMIT AMENDMENT EXCAVATION SLOPE STABILITY SECTIONS ROYAL OAKS LANDFILL CHEROKEE COUNTY, TEXAS	
DATE: 05/2024 FILE: 0120-76-11 CAD: SHEET IIIE-A-10.DWG	DRAWN BY: RAA DESIGN BY: JBM REVIEWED BY: DEP	REVISIONS	
Weaver Consultants Group TBPE REGISTRATION NO. F-3727		WWW.WCGRP.COM	SHEET IIIE-A-10

NO.	DATE	DESCRIPTION

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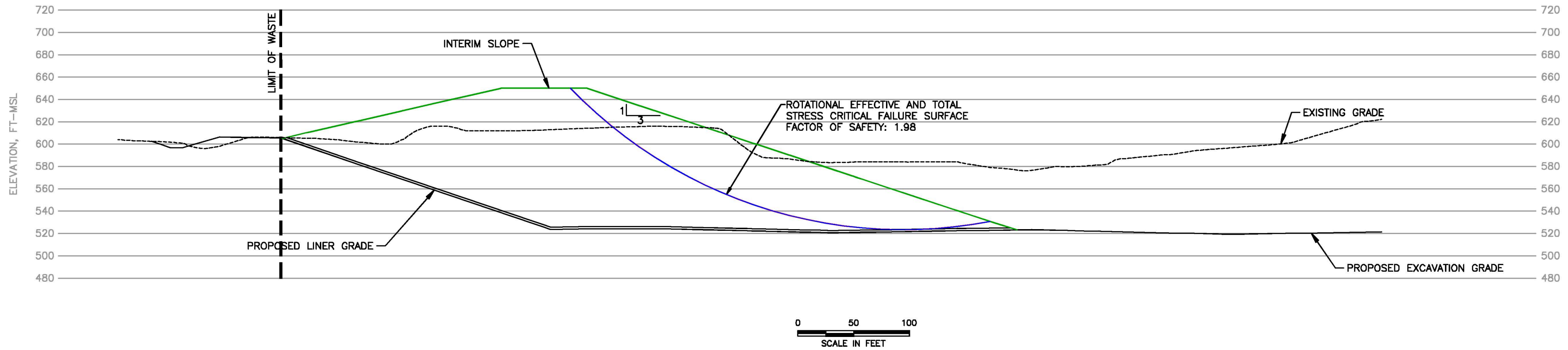
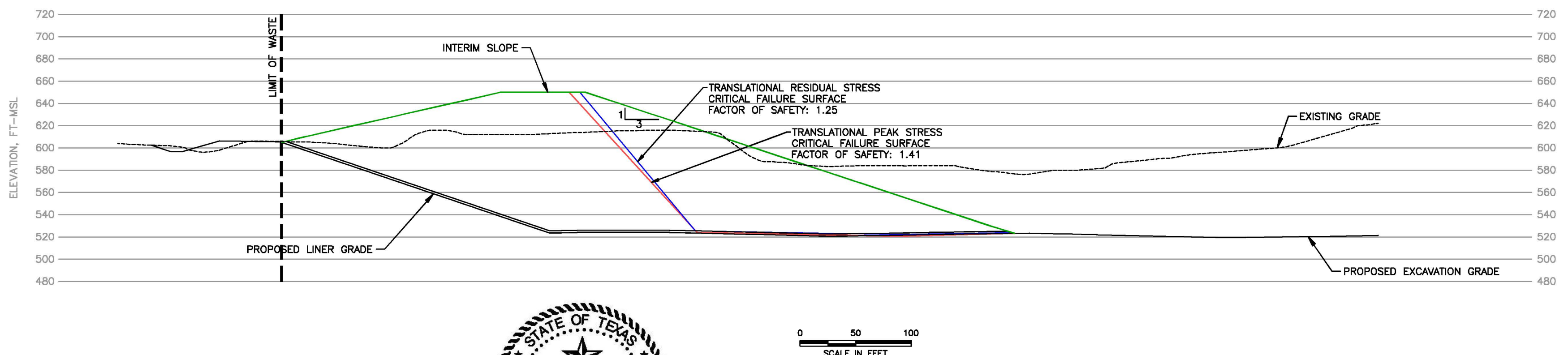
INTERIM SECTION D–D



INTERIM SECTION D–D



STATE OF TEXAS
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81734
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PROFESSIONAL ENGINEER
05/20/2024

- [ ] DRAFT
- [x] FOR PERMITTING PURPOSES ONLY
- [ ] ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-76-11
CAD: SHEET IIIE-A-11.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
INTERIM SLOPE STABILITY SECTIONS

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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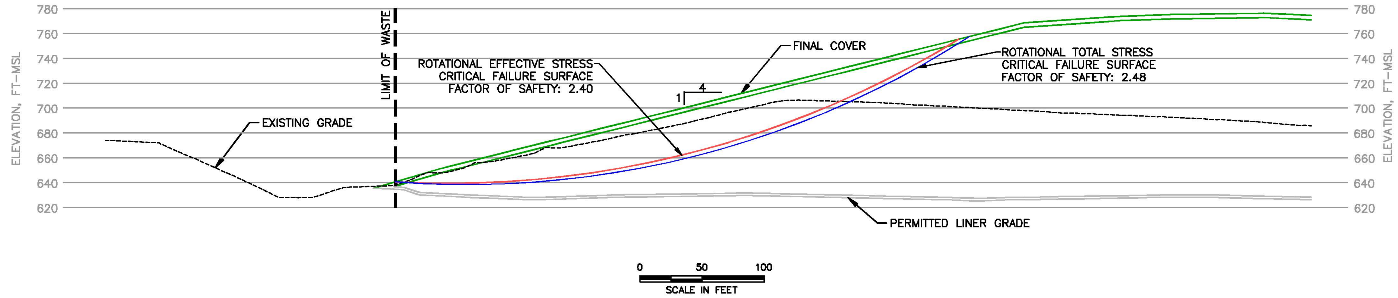
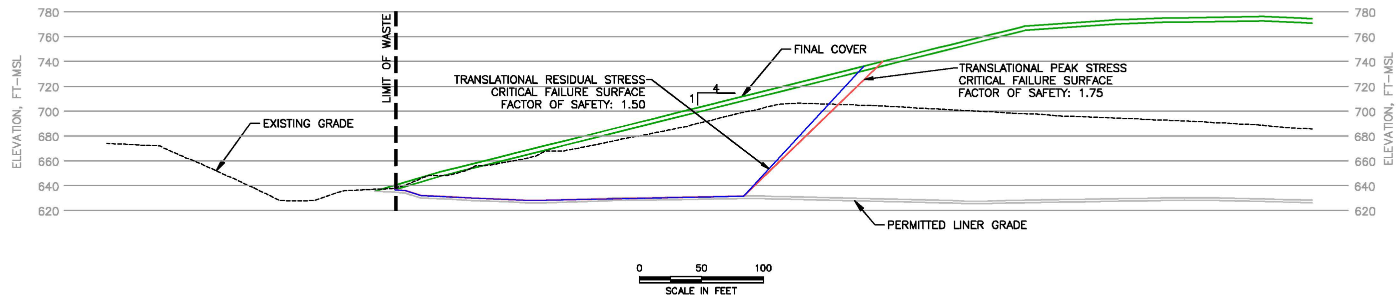
SHEET IIIE-A-11

FINAL COVER SECTION E–E



FINAL COVER SECTION E–E



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☐ ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-76-11
CAD: SHEET IIIE-A-12.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
FINAL CONDITIONS SLOPE STABILITY SECTIONS
ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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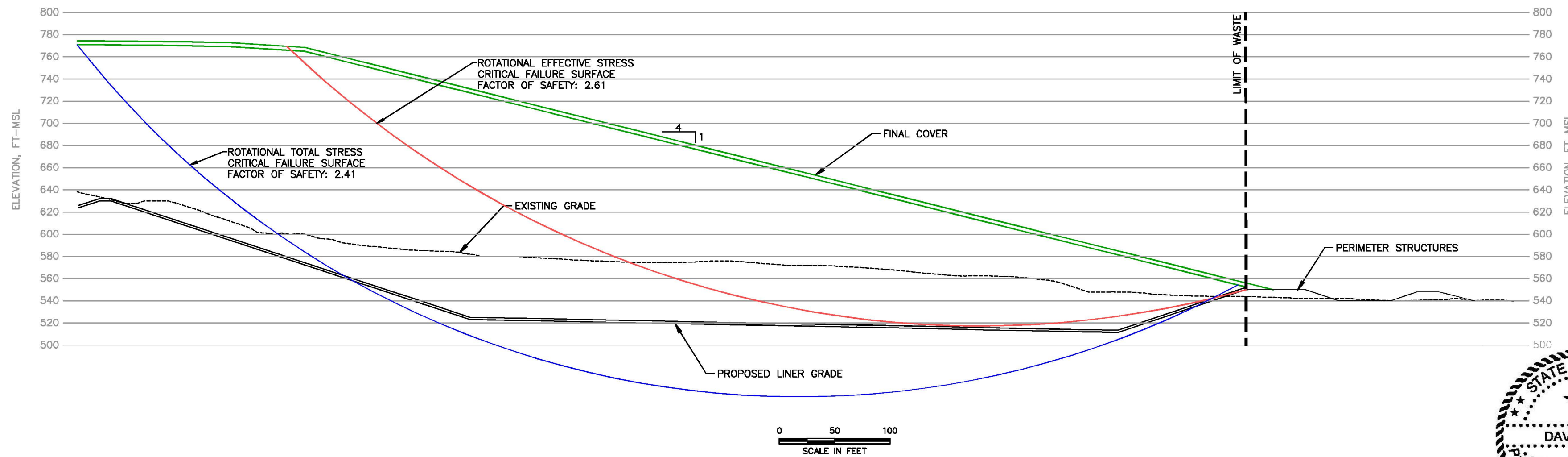
SHEET IIIE-A-12

FINAL COVER SECTION F–F



STATE OF TEXAS
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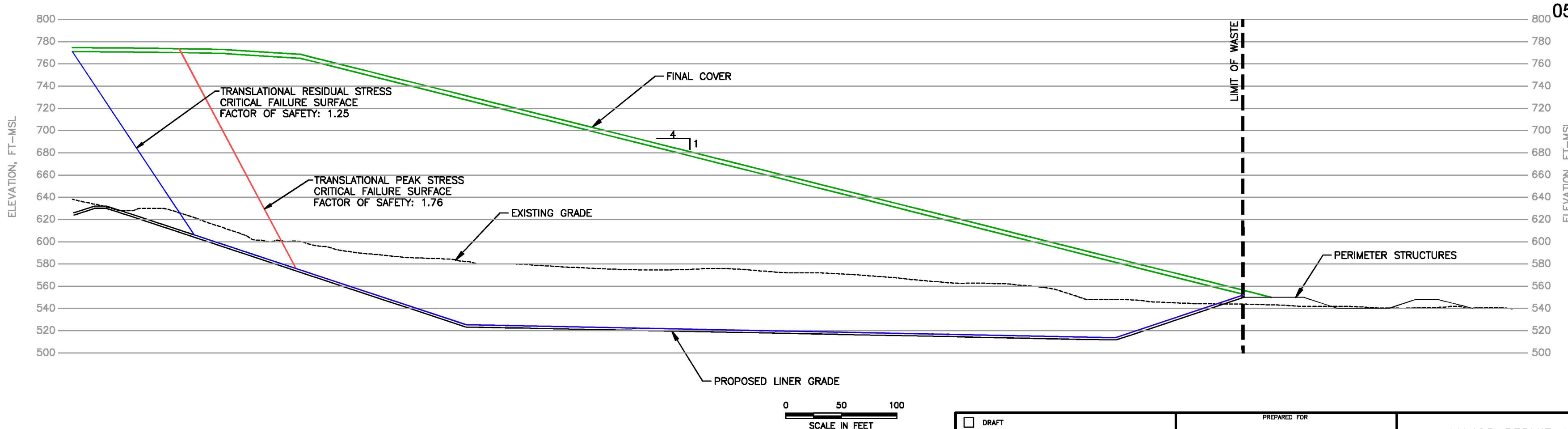
LICENSED
PROFESSIONAL ENGINEER
05/20/2024

FINAL COVER SECTION F–F



☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0023-404-11
CAD: SHEET IIIE-A-13.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
FINAL CONDITIONS SLOPE STABILITY SECTIONS

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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SHEET IIIE–A–13

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APPENDIX IIIE-A-1

LANDFILL EXCAVATION CONFIGURATION
STABILITY ANALYSIS
SLIDE2 OUTPUT FILES

SECTIONS A-A, B-B AND C-C

Includes pages IIIE-A-1-1 through IIIE-A-1-45

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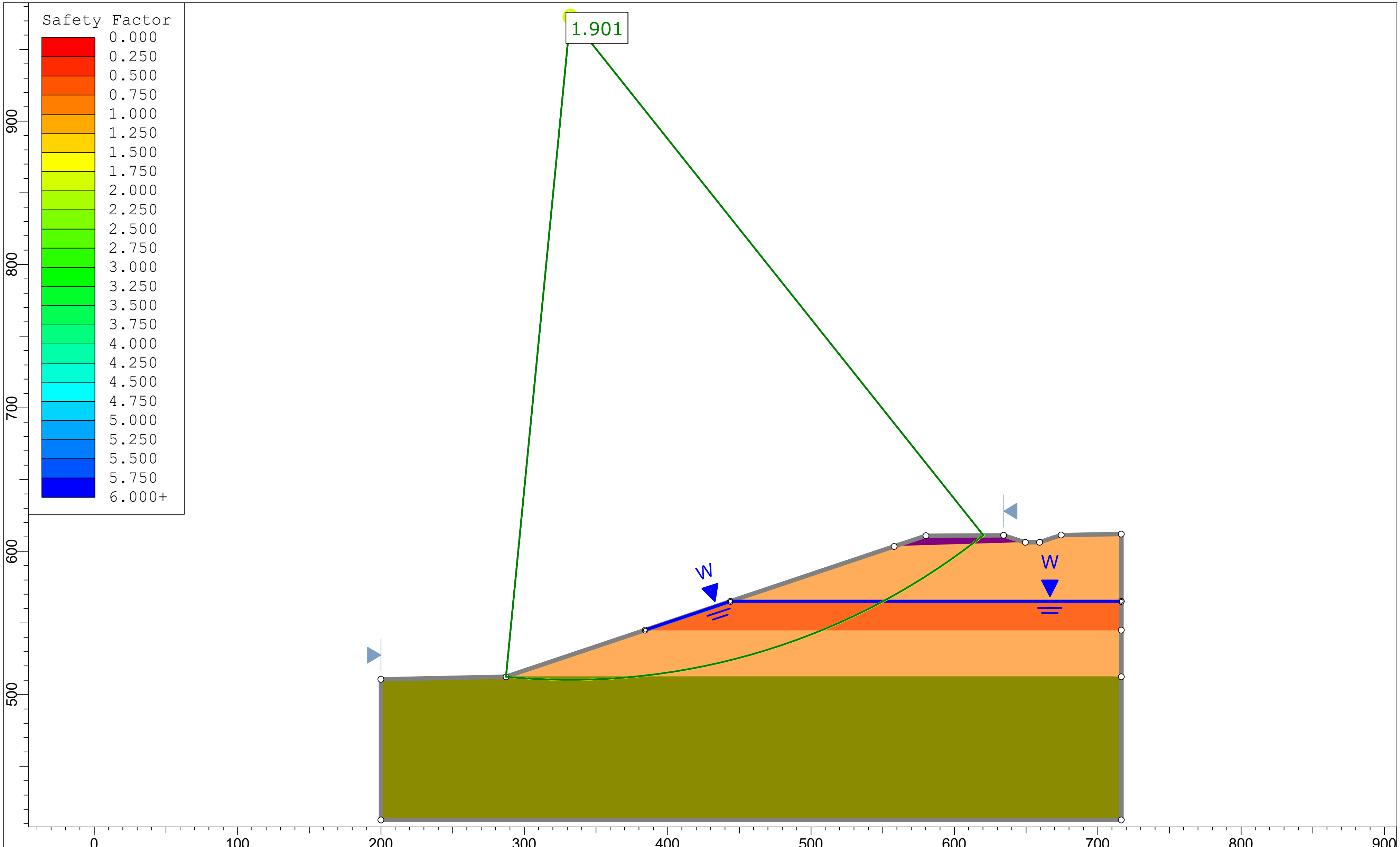
05/20/2024

SLOPE STABILITY SECTION A-A
SLIDE2 OUTPUT RESULTS



Weaver Consultants Group	Project	ROYAL OAKS LANDFILL	
	Group: ENGINEERING	Scenario: EFFECTIVE STRESS - CIRCULAR SECTION A-A	
	Drawn By: PREP BY: JBM CHKD BY:DEP	Company: WEAVER CONSULTANTS GROUP	
SLIDEINTERPRET 9.008	Date: 12/7/2023	File Name: EFFECTIVE CIRCULAR A-A.slmd	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

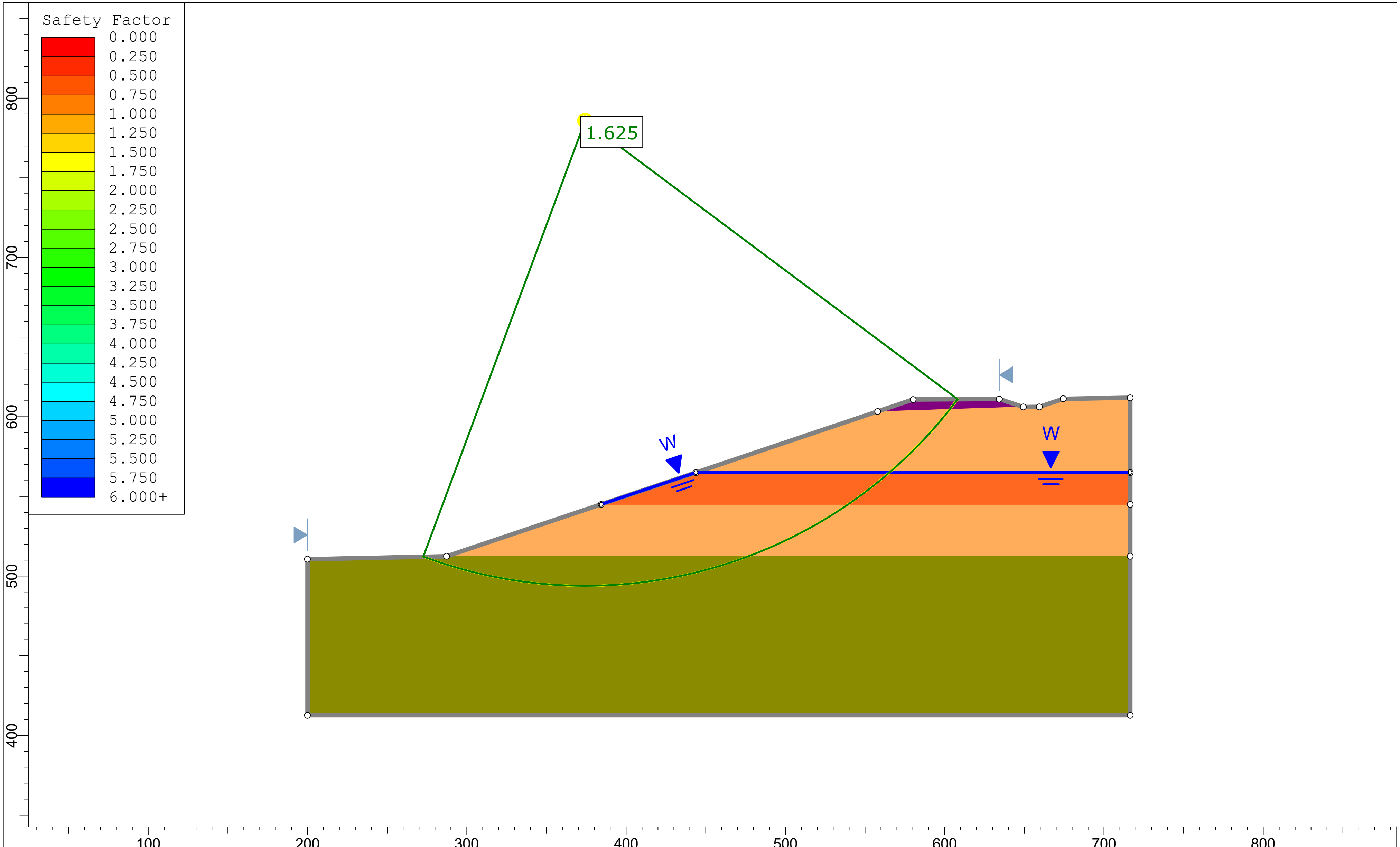
Materials

COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	1.901090
Center:	332.055, 972.783
Radius:	462.561
Left Slip Surface Endpoint:	287.225, 512.400
Right Slip Surface Endpoint:	620.282, 610.999
Resisting Moment:	3.03089e+08 lb-ft
Driving Moment:	1.59429e+08 lb-ft
Total Slice Area:	9833.64 ft2
Surface Horizontal Width:	333.057 ft
Surface Average Height:	29.5254 ft



Weaver Consultants Group	*Project* ROYAL OAKS LANDFILL		
	Group ENGINEERING	*Scenario*	TOTAL STRESS - CIRCULAR SECTION A-A
	Drawn By PREP BY: JBM CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
SLIDEINTERPRET 9.008	*Date* 12/7/2023	*File Name*	TOTAL CIRCULAR A-A.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

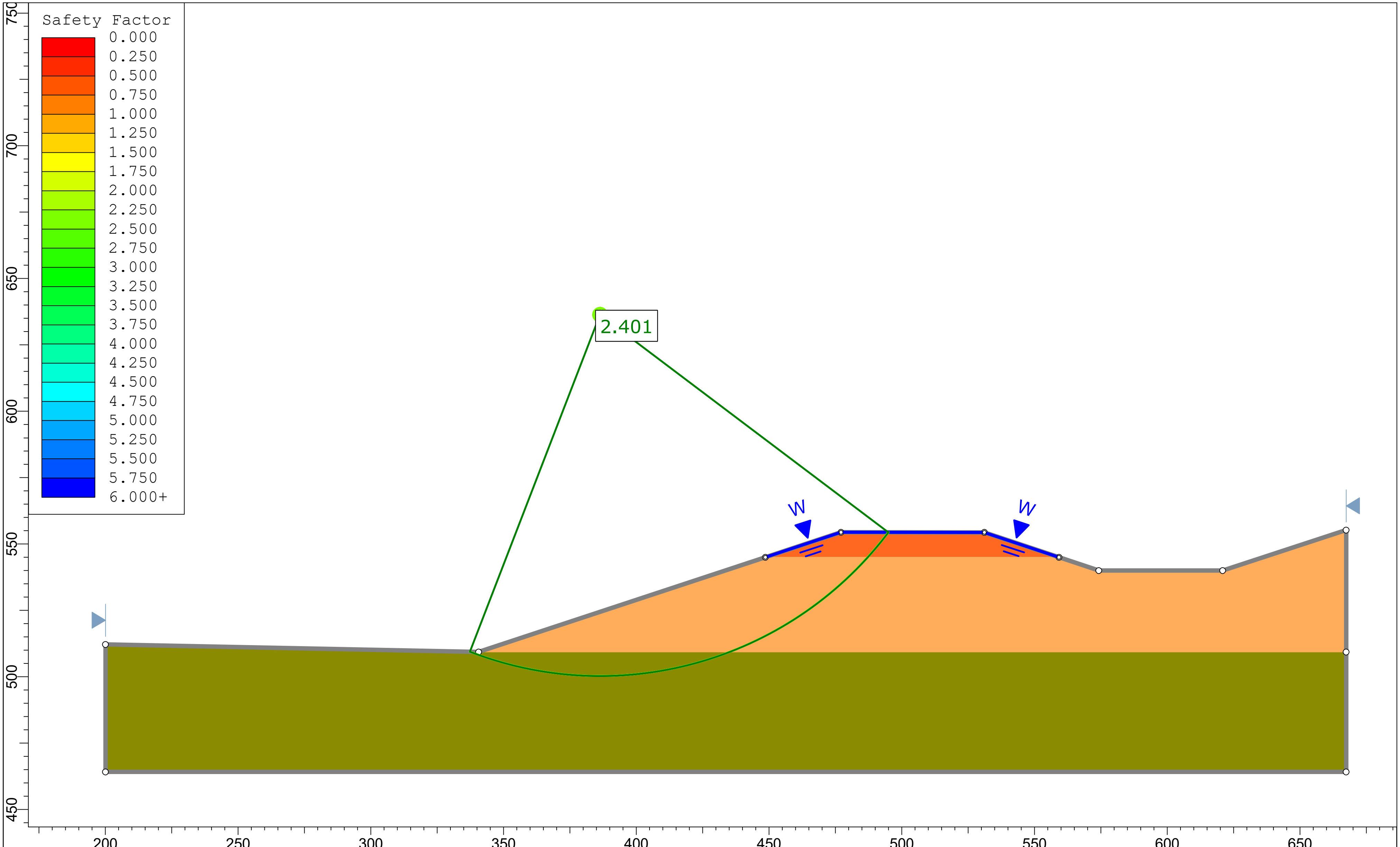
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	1.624580
Center:	374.312, 785.811
Radius:	291.986
Left Slip Surface Endpoint:	272.637, 512.100
Right Slip Surface Endpoint:	608.137, 610.934
Resisting Moment:	2.3266e+08 lb-ft
Driving Moment:	1.43212e+08 lb-ft
Total Slice Area:	14533.7 ft2
Surface Horizontal Width:	335.5 ft
Surface Average Height:	43.3195 ft

SLOPE STABILITY SECTION B-B
SLIDE2 OUTPUT RESULTS



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL
Group	ENGINEERING
Scenario	EFFECTIVE STRESS - CIRCULAR SECTION B-B
Drawn By	PREP BY: JBM CHKD BY:DEP
Company	WEAVER CONSULTANTS GROUP
Date	12/7/2023
File Name	EFFECTIVE CIRCULAR B-B.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

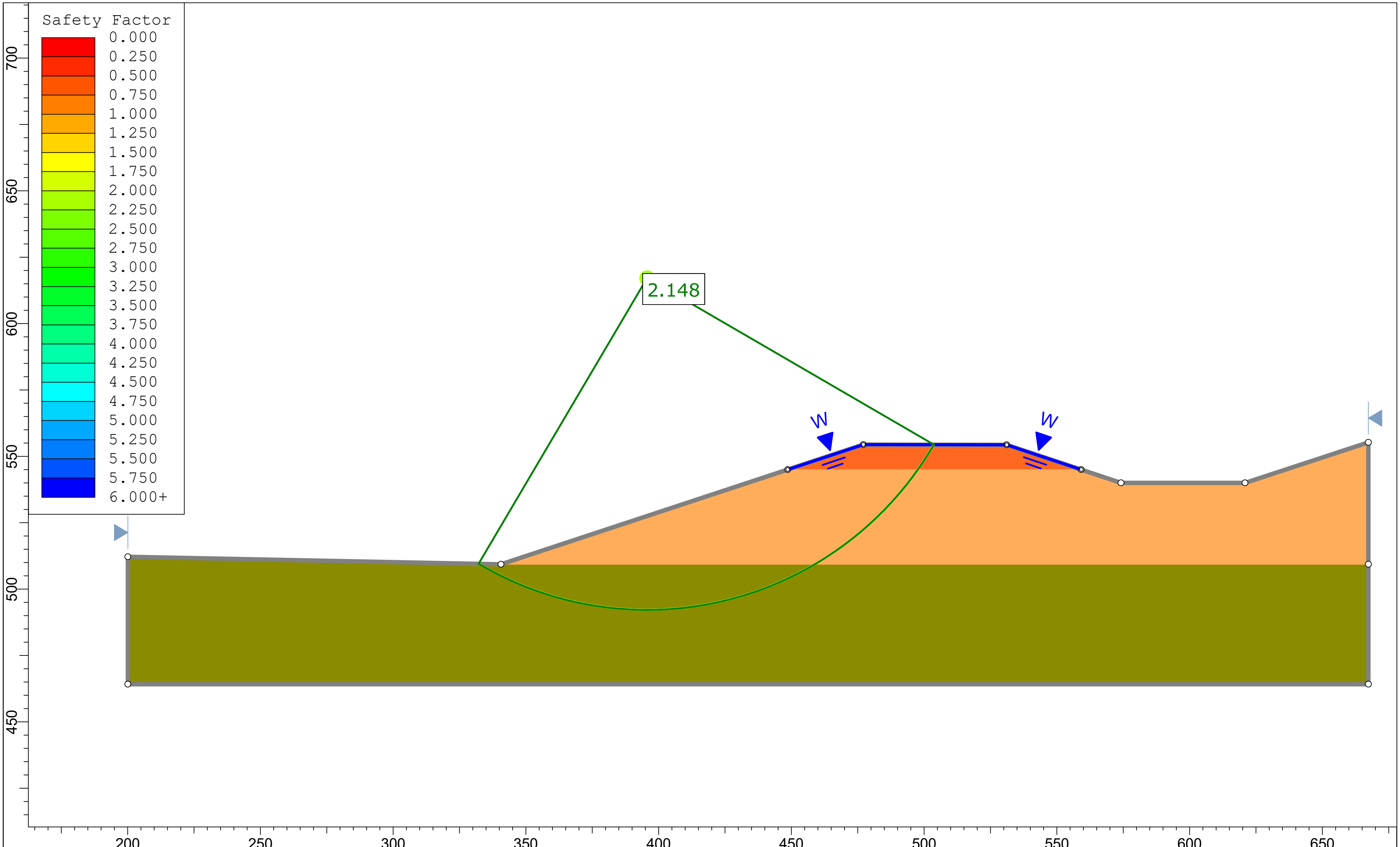
Materials

INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	2.400890
Center:	386.325, 636.374
Radius:	136.107
Left Slip Surface Endpoint:	337.297, 509.404
Right Slip Surface Endpoint:	494.971, 554.391
Resisting Moment:	3.60386e+07 lb-ft
Driving Moment:	1.50105e+07 lb-ft
Total Slice Area:	3391.77 ft2
Surface Horizontal Width:	157.674 ft
Surface Average Height:	21.5113 ft



Weaver Consultants Group

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	TOTAL STRESS - CIRCULAR SECTION B-B
Drawn By	PREP BY: JBM CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	12/7/2023	*File Name*	TOTAL CIRCULAR B-B.slmd

SLIDEINTERPRET 9.008

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

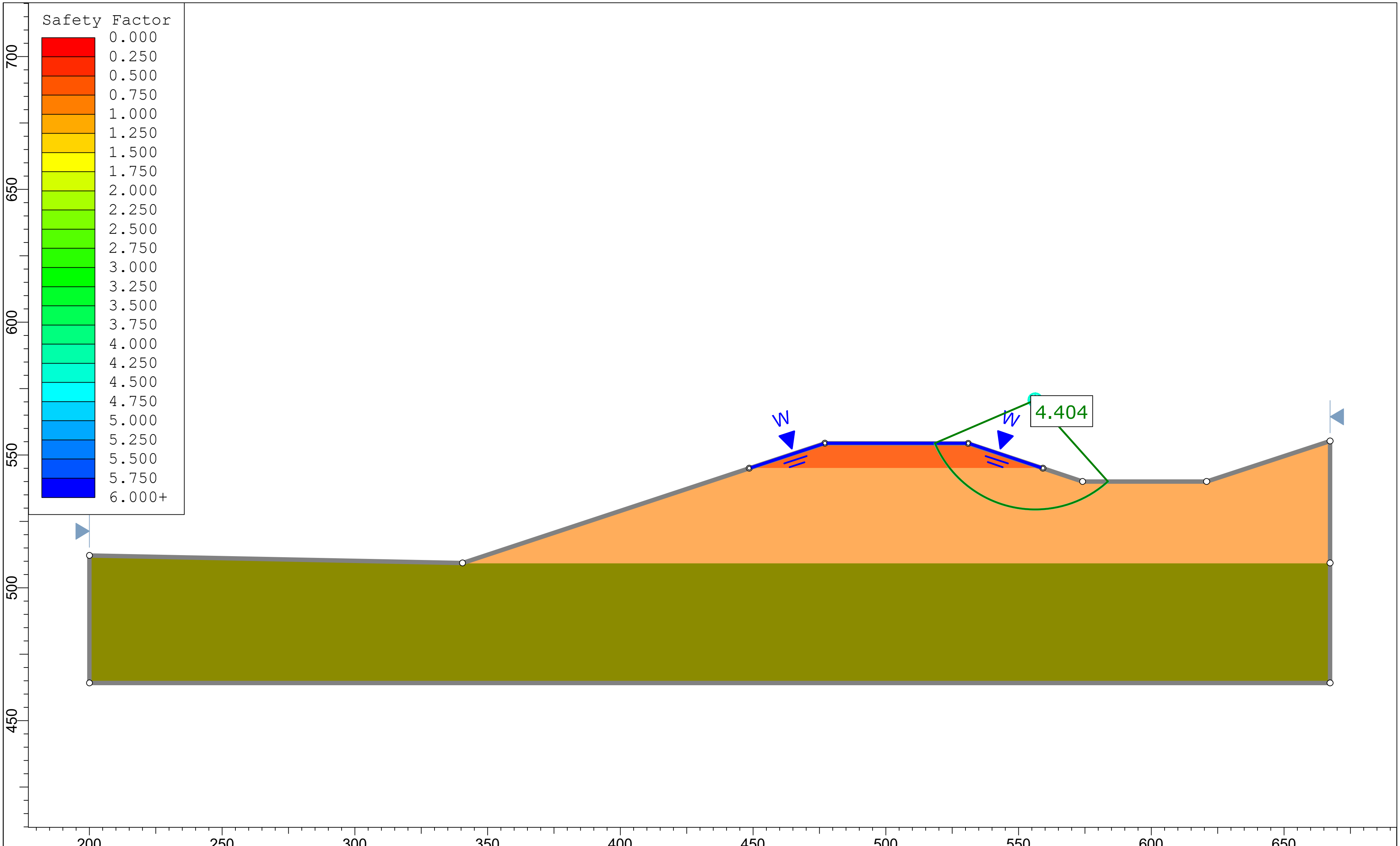
Materials

INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	2.147510
Center:	395.628, 617.113
Radius:	124.941
Left Slip Surface Endpoint:	332.136, 509.507
Right Slip Surface Endpoint:	503.681, 554.387
Resisting Moment:	3.95146e+07 lb-ft
Driving Moment:	1.84002e+07 lb-ft
Total Slice Area:	4905.11 ft2
Surface Horizontal Width:	171.545 ft
Surface Average Height:	28.5937 ft



Weaver Consultants Group	Project: ROYAL OAKS LANDFILL	
	Group: ENGINEERING	Scenario: EFFECTIVE STRESS - CIRCULAR SECTION B-B
	Drawn By: PREP BY: JBM CHKD BY:DEP	Company: WEAVER CONSULTANTS GROUP
SLIDEINTERPRET 9.008	Date: 12/7/2023	File Name: EFFECTIVE CIRCULAR B-B EXT.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

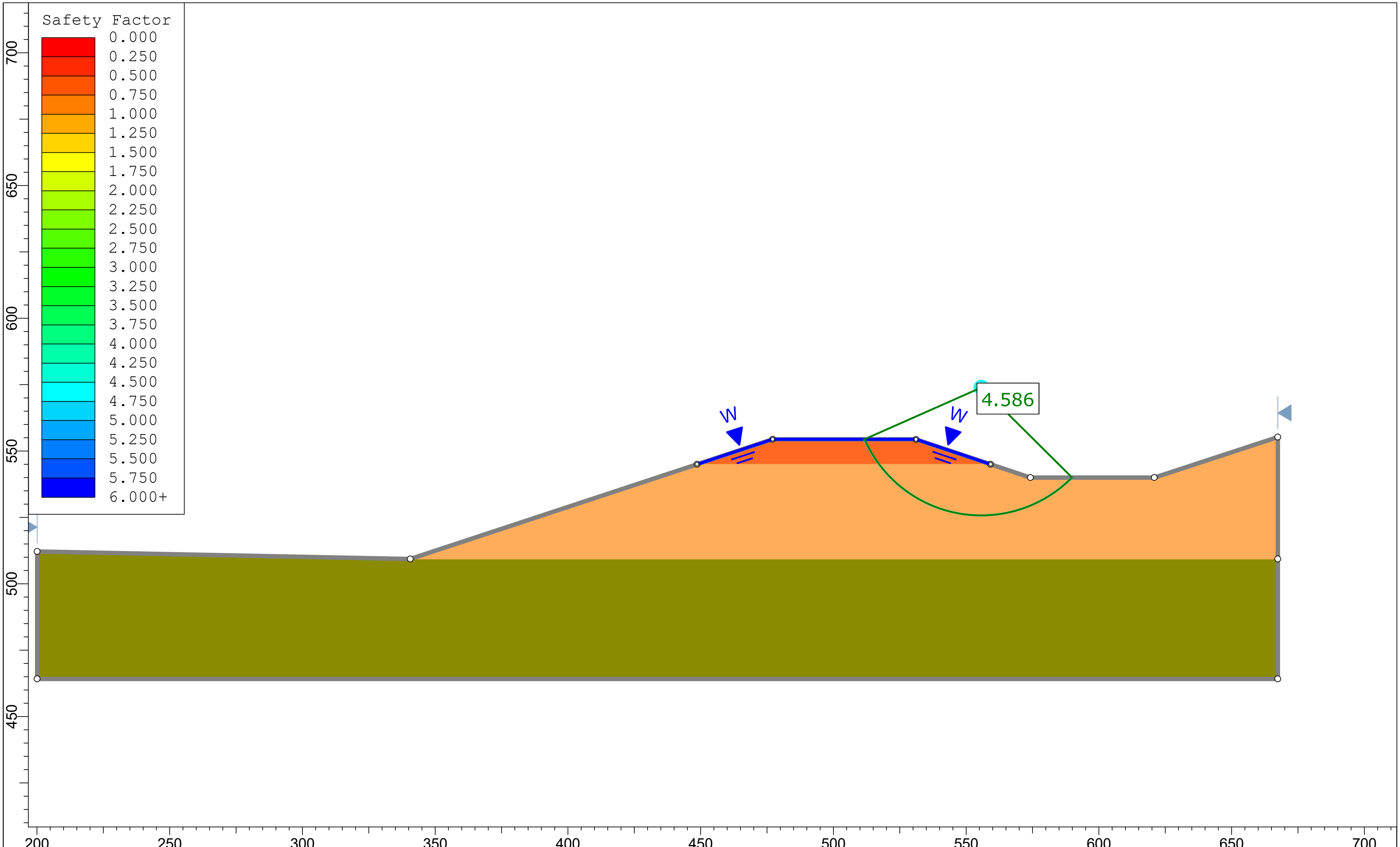
Materials

INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	4.404220
Center:	556.268, 570.629
Radius:	41.129
Left Slip Surface Endpoint:	518.484, 554.379
Right Slip Surface Endpoint:	583.718, 540.000
Resisting Moment:	4.08772e+06 lb-ft
Driving Moment:	928138 lb-ft
Total Slice Area:	822.844 ft2
Surface Horizontal Width:	65.2335 ft
Surface Average Height:	12.6138 ft



Weaver Consultants Group	*Project* ROYAL OAKS LANDFILL
	Group ENGINEERING
	Scenario TOTAL STRESS - CIRCULAR SECTION B-B
	Drawn By PREP BY: JBM CHKD BY:DEP
	Company WEAVER CONSULTANTS GROUP
	Date 12/11/2023
SLIDEINTERPRET 9.008	*File Name* TOTAL CIRCULAR B-B EXT.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

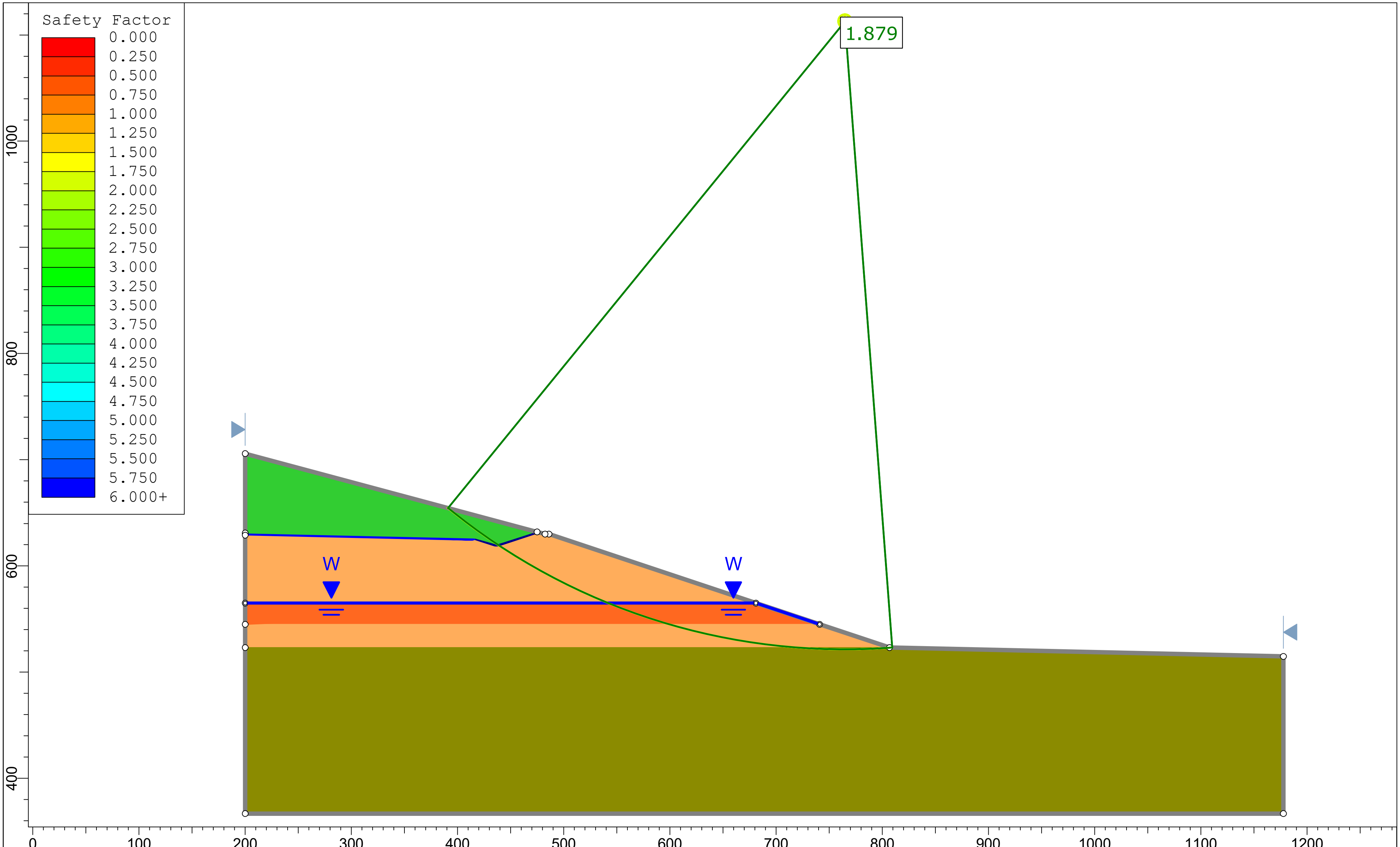
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	4.585770
Center:	555.696, 573.870
Radius:	48.148
Left Slip Surface Endpoint:	511.668, 554.383
Right Slip Surface Endpoint:	589.915, 540.000
Resisting Moment:	6.42091e+06 lb-ft
Driving Moment:	1.40018e+06 lb-ft
Total Slice Area:	1200.48 ft2
Surface Horizontal Width:	78.247 ft
Surface Average Height:	15.3422 ft

SLOPE STABILITY SECTION C-C
SLIDE2 OUTPUT RESULTS



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	EFFECTIVE STRESS - CIRCULAR SECTION C-C
Drawn By	PREP BY: JBM CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	12/7/2023	*File Name*	EFFECTIVE CIRCULAR C-C.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0

Shear Normal Functions

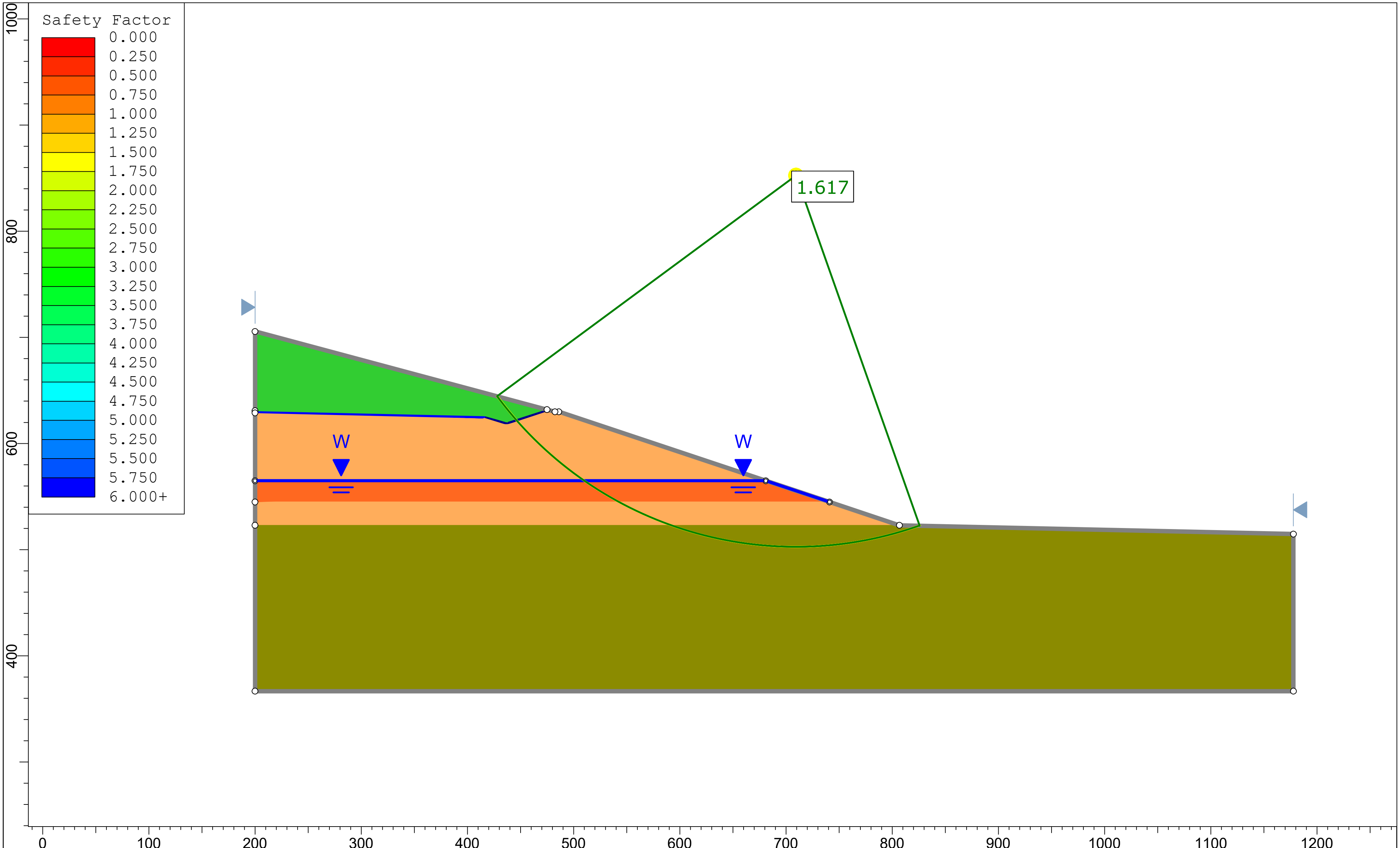
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.878500
Center:	764.870, 1112.522
Radius:	591.208
Left Slip Surface Endpoint:	391.096, 654.460
Right Slip Surface Endpoint:	809.369, 522.991
Resisting Moment:	4.83879e+08 lb-ft
Driving Moment:	2.57587e+08 lb-ft
Total Slice Area:	13358.2 ft2
Surface Horizontal Width:	418.273 ft
Surface Average Height:	31.9365 ft



Weaver Consultants Group	Project	ROYAL OAKS LANDFILL	
	Group: ENGINEERING	Scenario: TOTAL STRESS - CIRCULAR SECTION C-C	
	Drawn By: PREP BY: JBM CHKD BY:DEP	Company: WEAVER CONSULTANTS GROUP	
SLIDEINTERPRET 9.008	Date: 12/7/2023	File Name: TOTAL CIRCULAR C-C.slmd	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.616570
Center:	709.387, 852.637
Radius:	349.960
Left Slip Surface Endpoint:	427.992, 644.576
Right Slip Surface Endpoint:	825.842, 522.622
Resisting Moment:	3.46658e+08 lb-ft
Driving Moment:	2.14441e+08 lb-ft
Total Slice Area:	19082.8 ft2
Surface Horizontal Width:	397.85 ft
Surface Average Height:	47.9647 ft

APPENDIX IIIE-A-2

INTERIM SLOPE CONFIGURATION
STABILITY ANALYSIS
SLIDE2 OUTPUT FILES

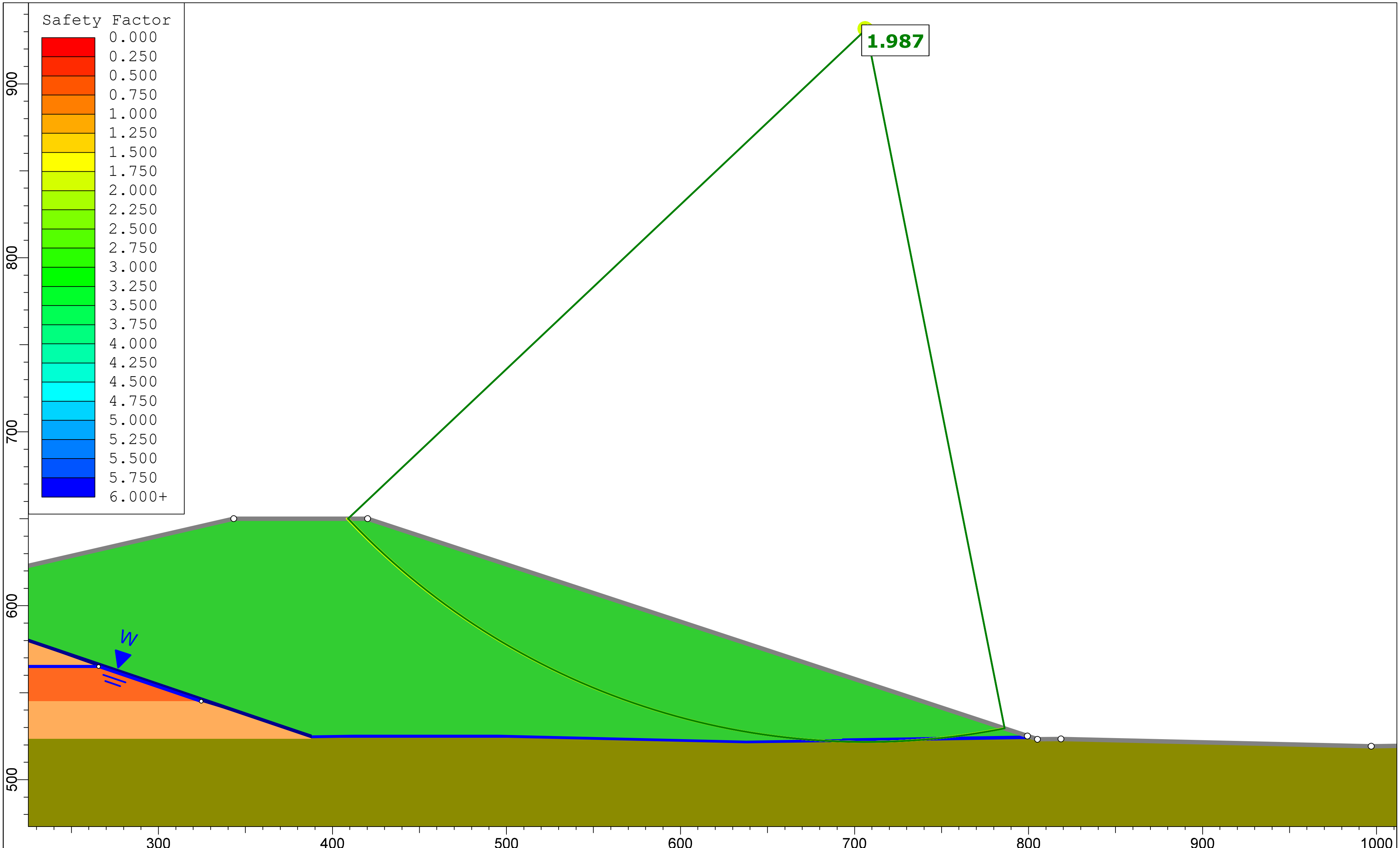
SECTION D-D

Includes pages IIIE-A-2-1 through IIIE-A-2-25

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

SLOPE STABILITY SECTION D-D – INTERIM SLOPE
SLIDE2 OUTPUT RESULTS



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	EFFECTIVE STRESS - CIRCULAR SECTION D-D
Drawn By	PREP BY: MB CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	4/17/2024	*File Name*	EFFECTIVE CIRCULAR D-D.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

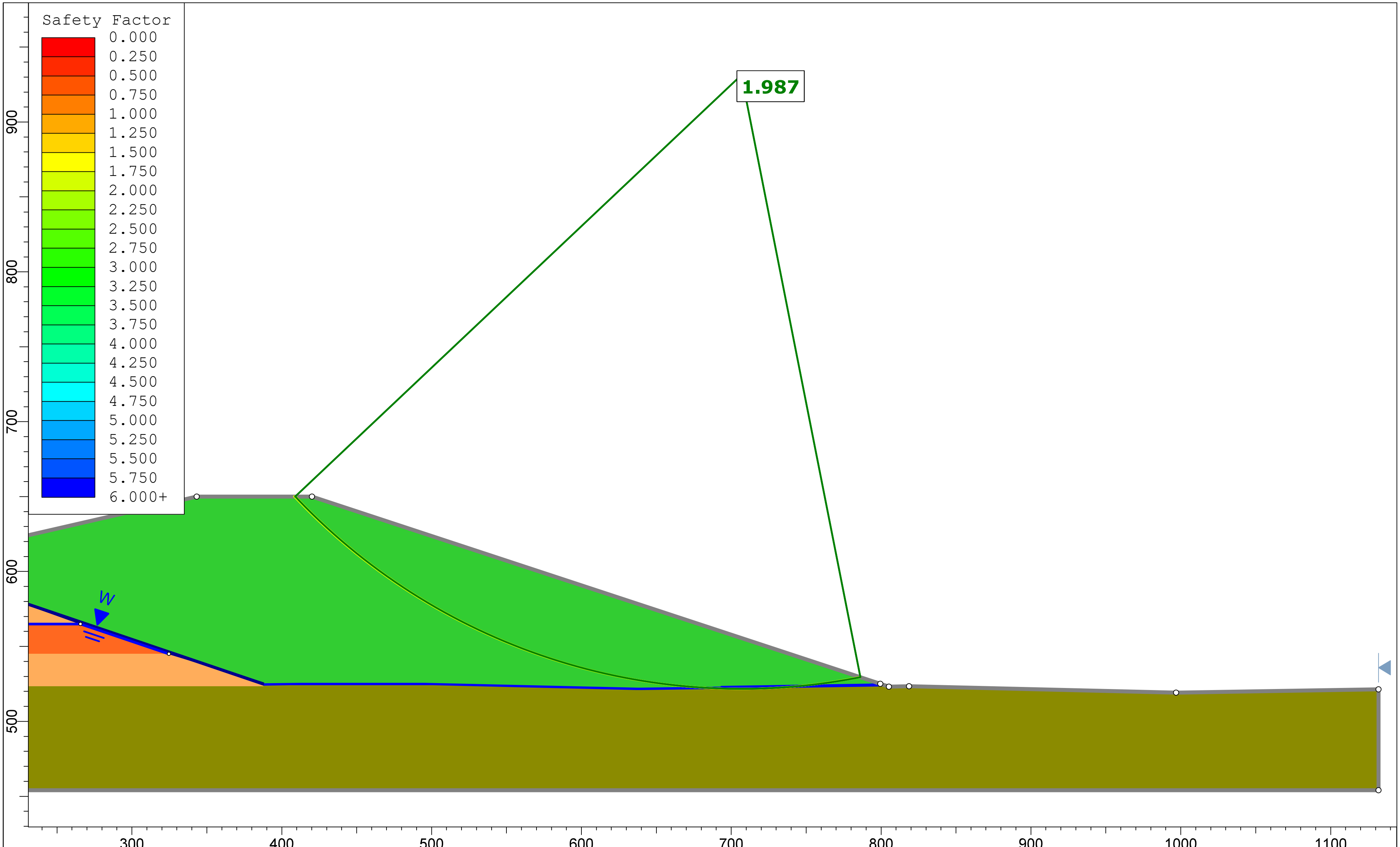
Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.987150
Center:	706.656, 931.302
Radius:	409.640
Left Slip Surface Endpoint:	408.874, 650.000
Right Slip Surface Endpoint:	786.196, 529.458
Resisting Moment:	2.18245e+08 lb-ft
Driving Moment:	1.09828e+08 lb-ft
Total Slice Area:	14338 ft2
Surface Horizontal Width:	377.322 ft
Surface Average Height:	37.9994 ft



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	Scenario	TOTAL STRESS - CIRCULAR SECTION D-D
Drawn By	PREP BY: MB CHKD BY:DEP	Company	WEAVER CONSULTANTS GROUP
Date	4/17/2024	File Name	TOTAL CIRCULAR D-D.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

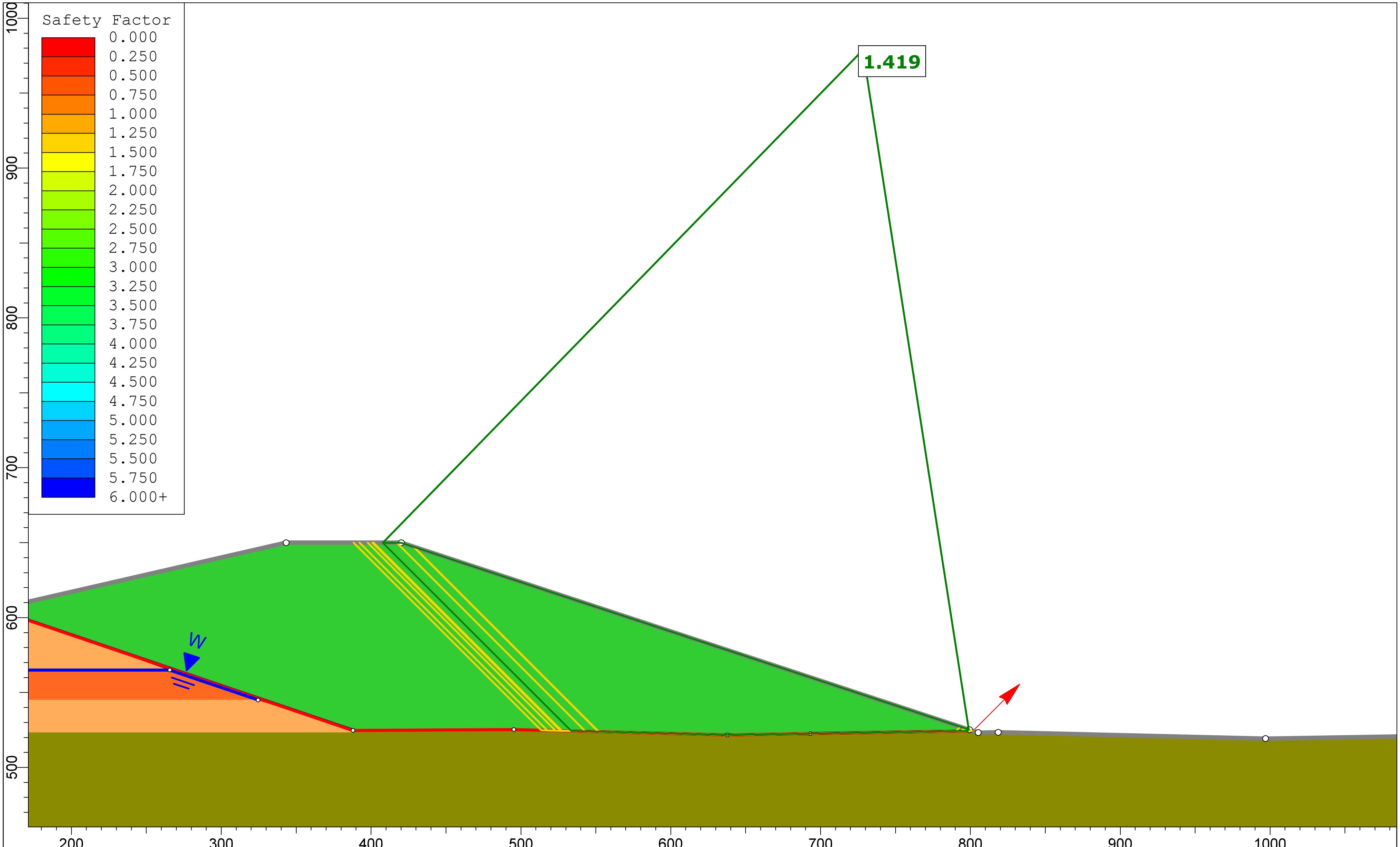
Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.987150
Center:	706.656, 931.302
Radius:	409.640
Left Slip Surface Endpoint:	408.874, 650.000
Right Slip Surface Endpoint:	786.196, 529.458
Resisting Moment:	2.18245e+08 lb-ft
Driving Moment:	1.09828e+08 lb-ft
Total Slice Area:	14338 ft2
Surface Horizontal Width:	377.322 ft
Surface Average Height:	37.9994 ft



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	PEAK STRESS - CIRCULAR SECTION D-D
Drawn By	PREP BY: MB CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	4/17/2024	*File Name*	PEAK BLOCK D-D.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

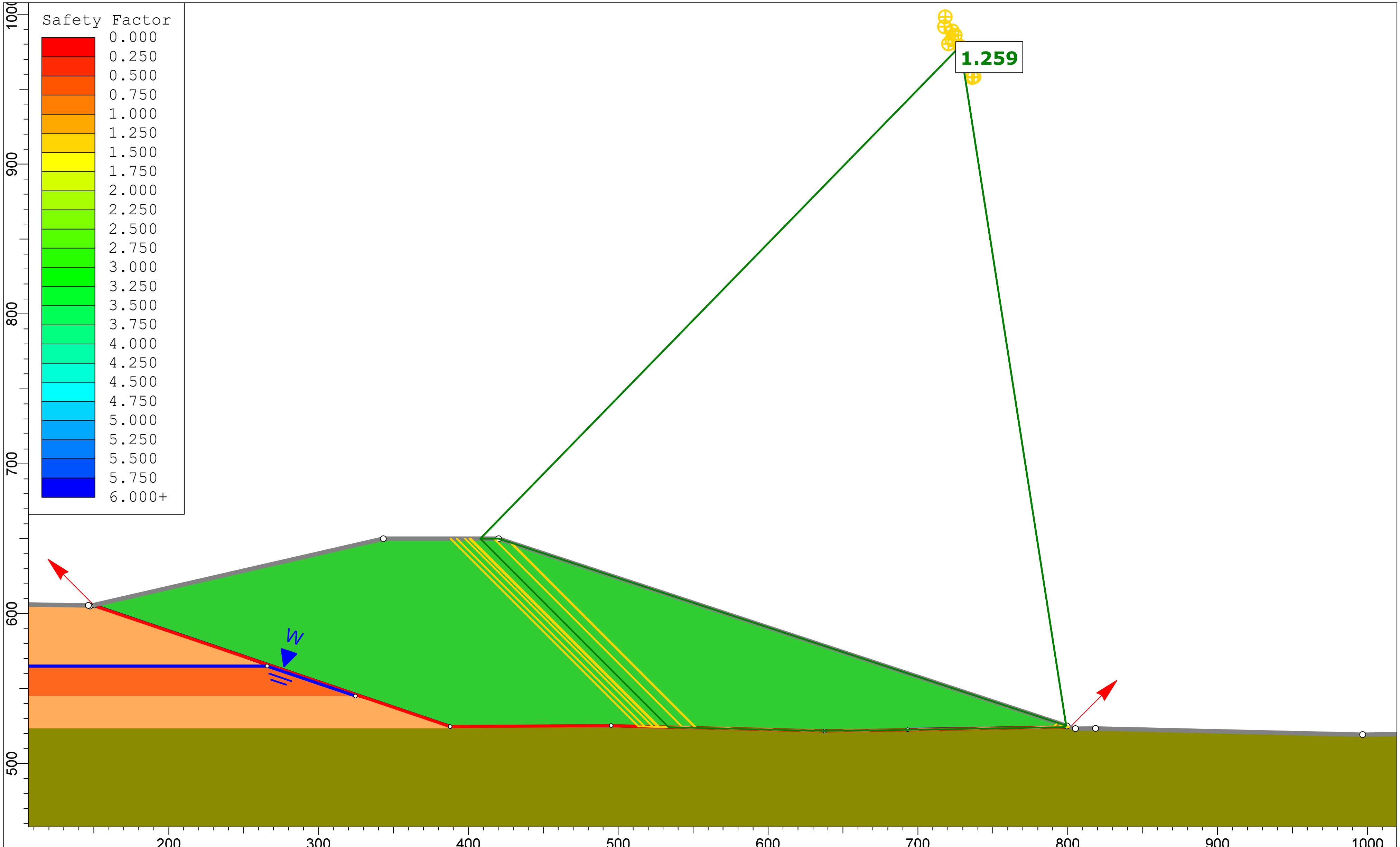
Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.419200
Axis Location:	728.176, 978.671
Left Slip Surface Endpoint:	407.911, 650.000
Right Slip Surface Endpoint:	798.954, 525.256
Resisting Moment:	1.95631e+08 lb-ft
Driving Moment:	1.37846e+08 lb-ft
Total Slice Area:	17984.9 ft2
Surface Horizontal Width:	391.043 ft
Surface Average Height:	45.9923 ft



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	RESIDUAL STRESS - CIRCULAR SECTION D-D
Drawn By	PREP BY: MB CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	4/17/2024	*File Name*	RESIDUAL BLOCK D-D.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	120
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

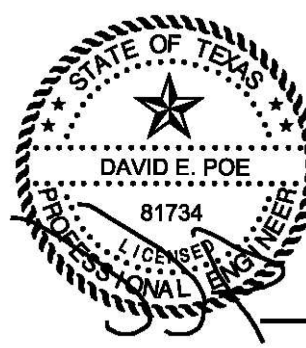
FS	1.259420
Axis Location:	728.176, 978.671
Left Slip Surface Endpoint:	407.911, 650.000
Right Slip Surface Endpoint:	798.954, 525.256
Resisting Moment:	1.7146e+08 lb-ft
Driving Moment:	1.36142e+08 lb-ft
Total Slice Area:	17984.9 ft2
Surface Horizontal Width:	391.043 ft
Surface Average Height:	45.9923 ft

APPENDIX IIIE-A-3

FINAL CLOSURE CONFIGURATION
STABILITY ANALYSIS
SLIDE2 OUTPUT FILES

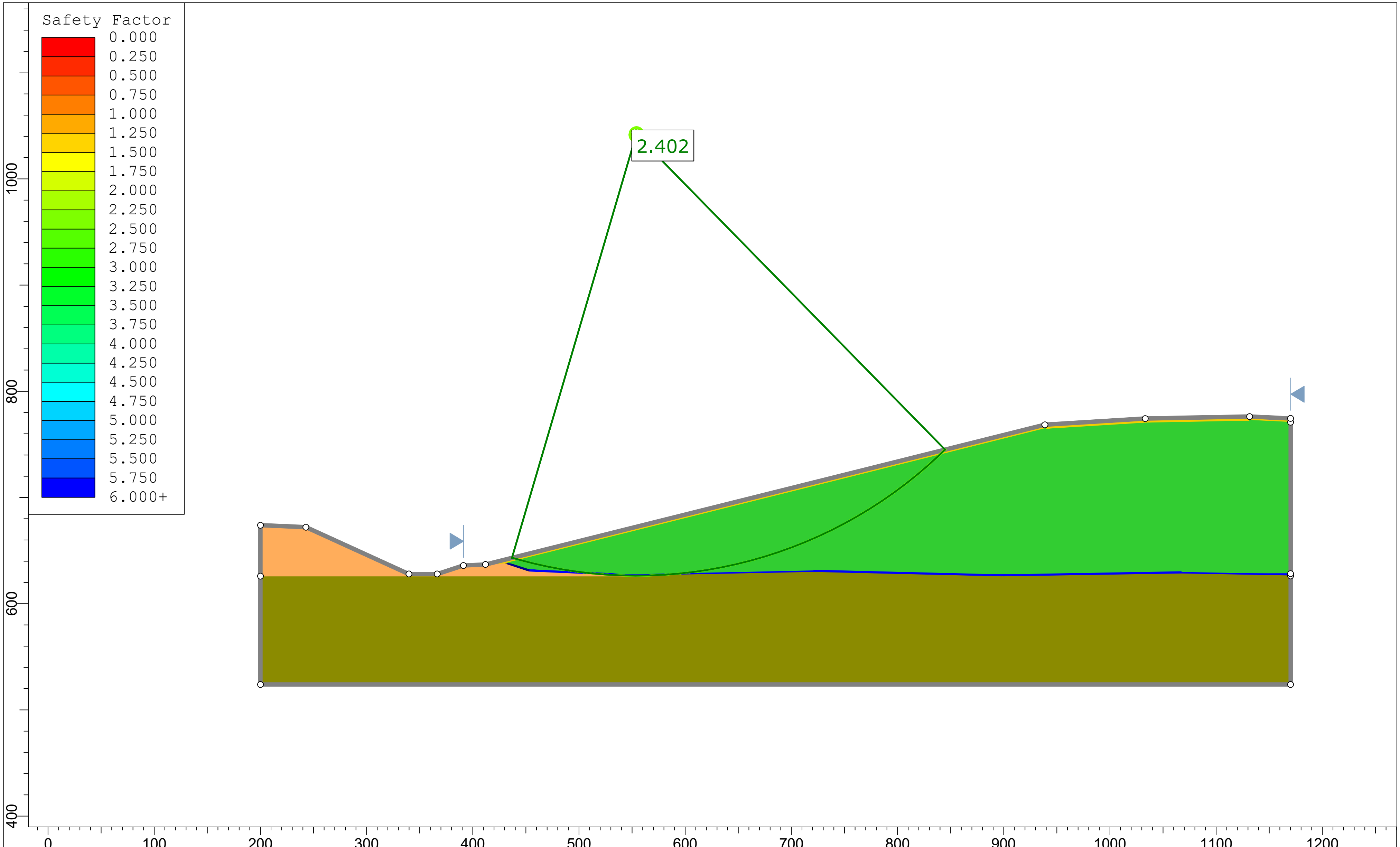
SECTIONS E-E AND F-F

Includes pages IIIE-A-3-1 through IIIE-A-3-54



05/20/2024

SLOPE STABILITY SECTION E-E – FINAL CLOSURE CONDITIONS
SLIDE2 OUTPUT RESULTS



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	EFFECTIVE STRESS - CIRCULAR SECTION E-E
Drawn By	PREP BY: JBM CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	12/7/2023	*File Name*	EFFECTIVE CIRCULAR E-E.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	115
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0

Shear Normal Functions

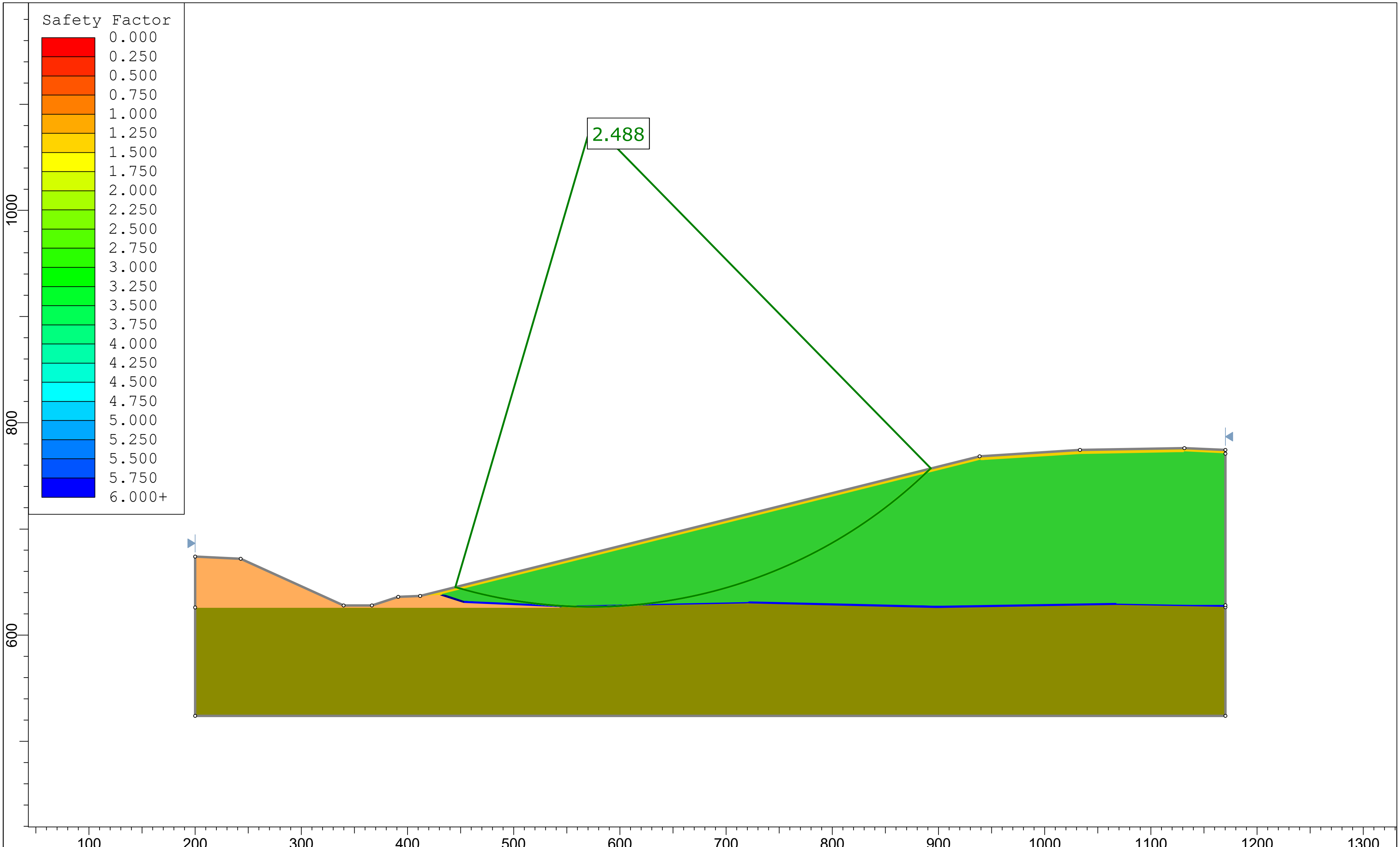
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	2.402200
Center:	553.979, 1041.939
Radius:	415.677
Left Slip Surface Endpoint:	436.862, 643.102
Right Slip Surface Endpoint:	844.892, 745.026
Resisting Moment:	2.47513e+08 lb-ft
Driving Moment:	1.03036e+08 lb-ft
Total Slice Area:	16239.9 ft2
Surface Horizontal Width:	408.03 ft
Surface Average Height:	39.8007 ft



Weaver Consultants Group	*Project* ROYAL OAKS LANDFILL		
	Group ENGINEERING	*Scenario* TOTAL STRESS - CIRCULAR SECTION E-E	
	Drawn By PREP BY: JBM CHKD BY:DEP	*Company* WEAVER CONSULTANTS GROUP	
SLIDEINTERPRET 9.008	*Date* 12/7/2023	*File Name* TOTAL CIRCULAR E-E.slmd	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	115
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

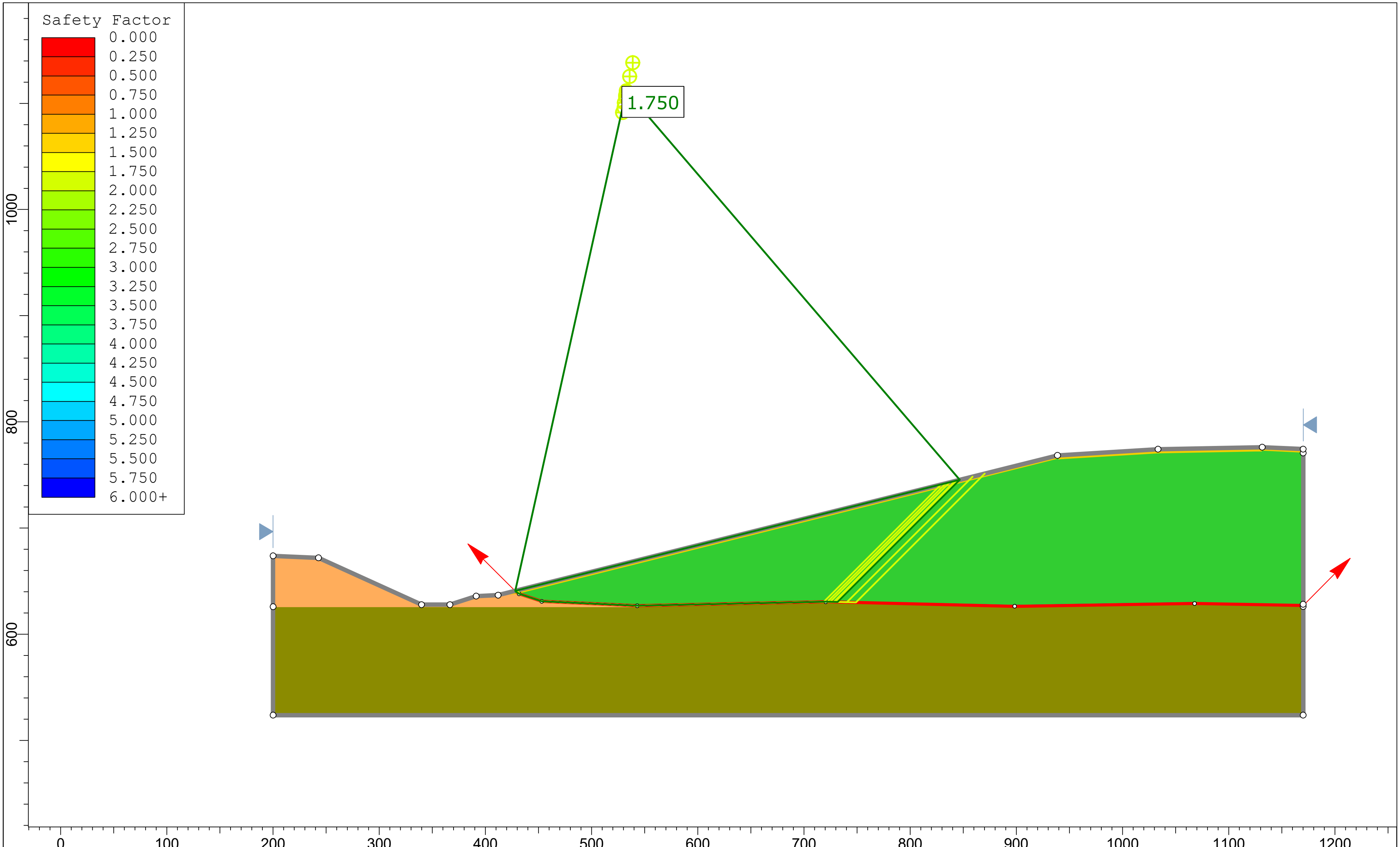
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	2.487810
Center:	573.570, 1082.834
Radius:	456.169
Left Slip Surface Endpoint:	445.044, 645.146
Right Slip Surface Endpoint:	892.821, 756.998
Resisting Moment:	3.37342e+08 lb-ft
Driving Moment:	1.35598e+08 lb-ft
Total Slice Area:	19557.9 ft2
Surface Horizontal Width:	447.777 ft
Surface Average Height:	43.6777 ft



Weaver Consultants Group	Project: ROYAL OAKS LANDFILL		
	Group: ENGINEERING	Scenario: PEAK STRESS - BLOCK SECTION E-E	
	Drawn By: PREP BY: JBM CHKD BY:DEP	Company: WEAVER CONSULTANTS GROUP	
SLIDEINTERPRET 9.008	Date: 12/7/2023	File Name: PEAK BLOCK E-E.slmd	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	115
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

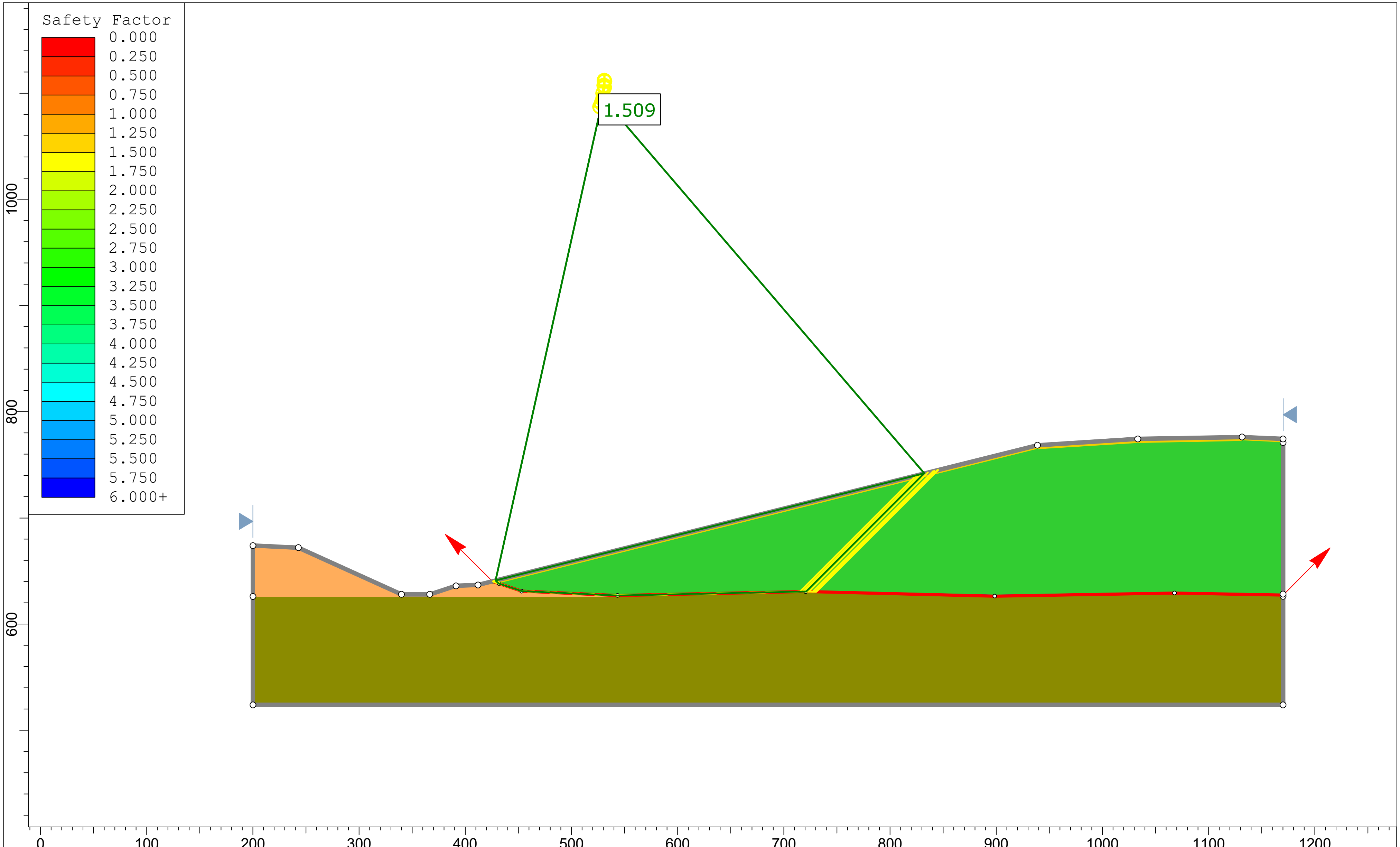
Shear Normal Functions

Name: User Defined 1	
Normal (psf)	**Shear (psf)**
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.750310
Axis Location:	532.682, 1111.737
Left Slip Surface Endpoint:	427.951, 640.876
Right Slip Surface Endpoint:	846.533, 745.436
Resisting Moment:	2.2982e+08 lb-ft
Driving Moment:	1.31303e+08 lb-ft
Total Slice Area:	19930.9 ft2
Surface Horizontal Width:	418.582 ft
Surface Average Height:	47.6154 ft



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	Scenario	RESIDUAL STRESS - BLOCK SECTION E-E
Drawn By	PREP BY: JBM CHKD BY:DEP	Company	WEAVER CONSULTANTS GROUP
Date	12/7/2023	File Name	RESIDUAL BLOCK E-E.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	120
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	120
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	108
Cohesion [psf]	0
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	115
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

Name: User Defined 1

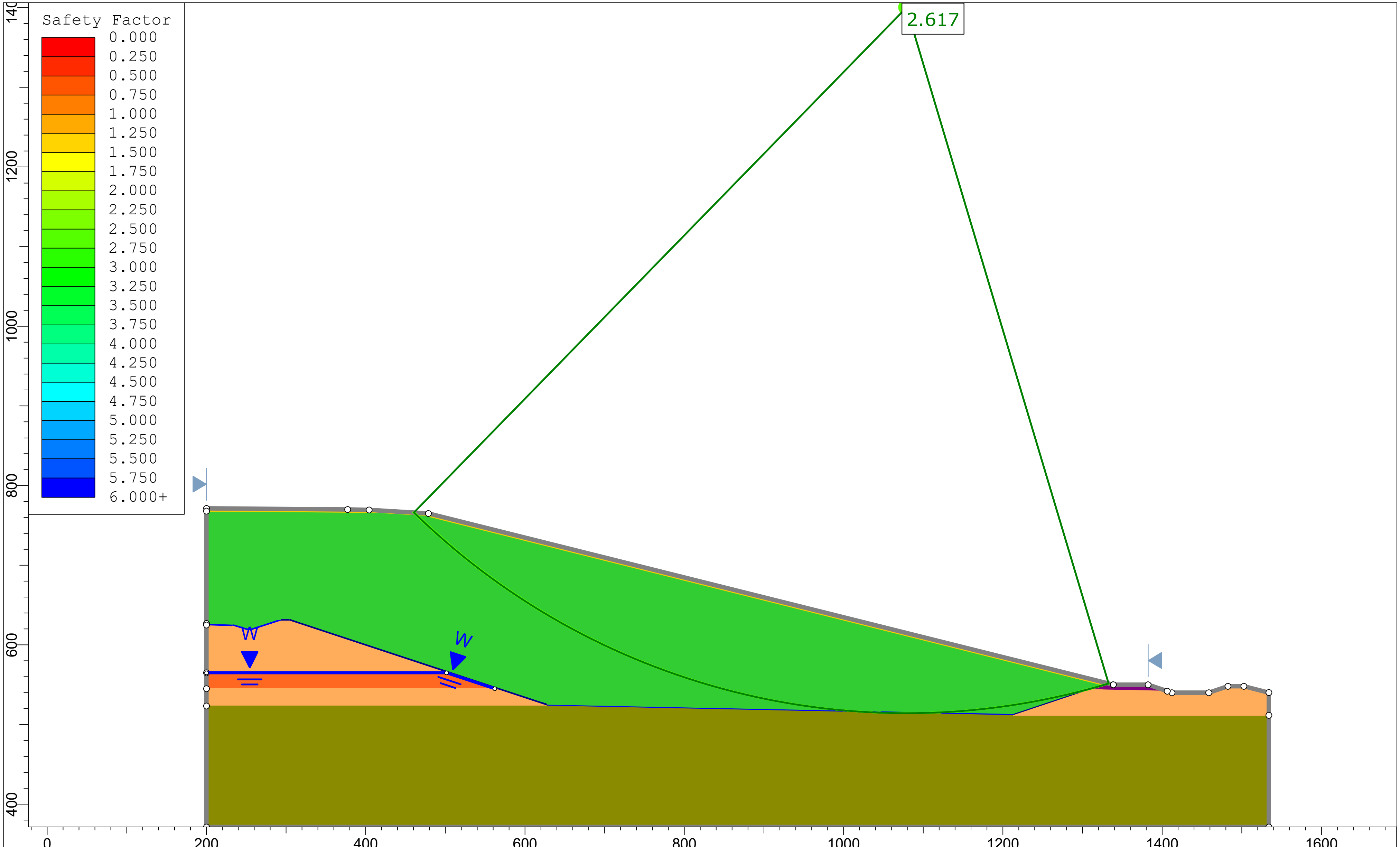
Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.508990
Axis Location:	529.512, 1095.088
Left Slip Surface Endpoint:	428.515, 641.017
Right Slip Surface Endpoint:	832.171, 741.848
Resisting Moment:	1.77127e+08 lb-ft
Driving Moment:	1.17381e+08 lb-ft
Total Slice Area:	18711 ft2
Surface Horizontal Width:	403.656 ft
Surface Average Height:	46.3539 ft

SLOPE STABILITY SECTION F-F – FINAL CLOSURE CONDITIONS
SLIDE 2 OUTPUT RESULTS



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	Scenario	EFFECTIVE STRESS - CIRCULAR SECTION F-F
Drawn By	PREP BY: JBM CHKD BY:DEP	Company	WEAVER CONSULTANTS GROUP
Date	12/11/2023	File Name	EFFECTIVE CIRCULAR F-F.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	200
Friction Angle [deg]	28
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	800
Friction Angle [deg]	19
Water Surface	None
Ru Value	0

Shear Normal Functions

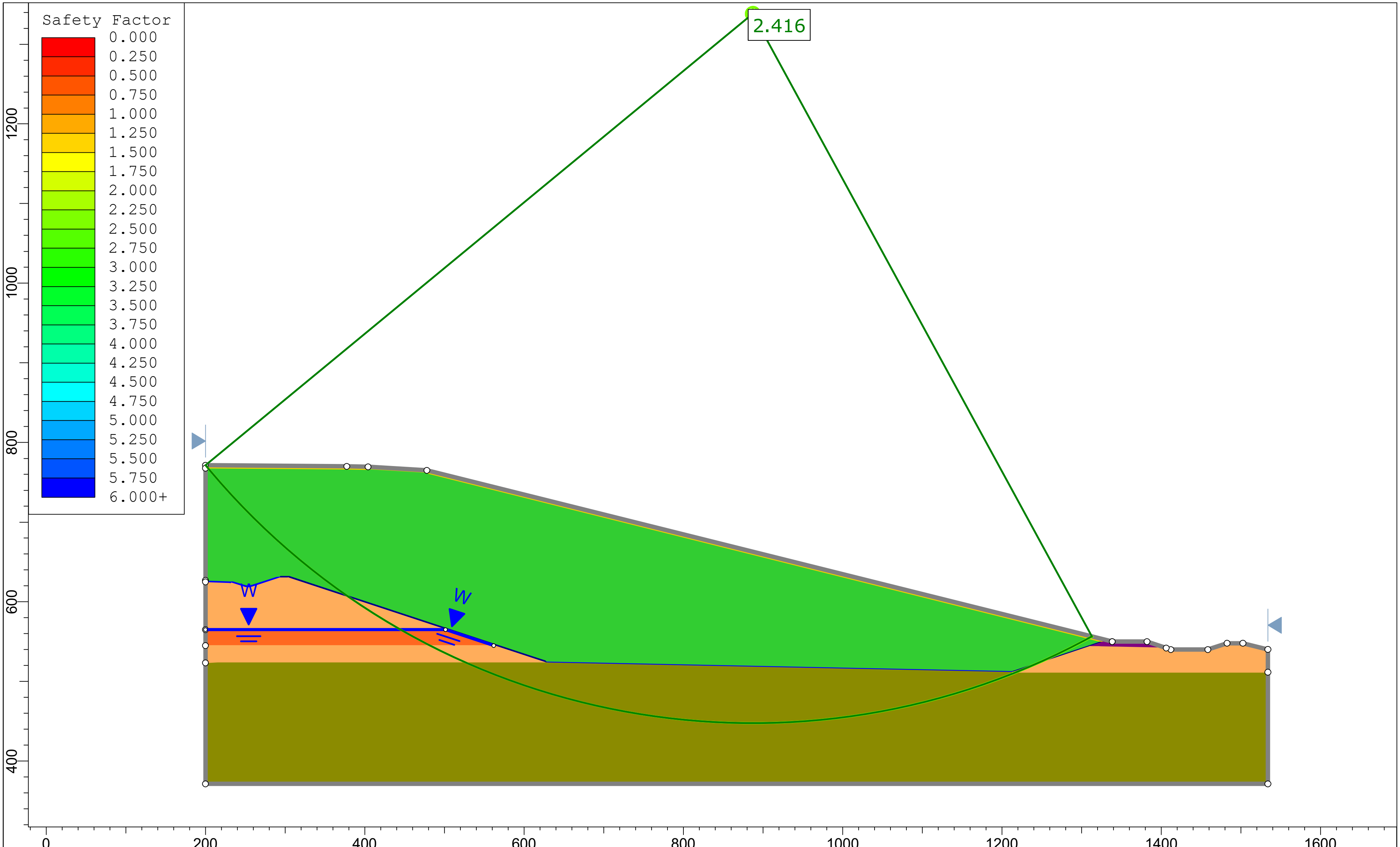
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	2.616900
Center:	1078.884, 1400.119
Radius:	885.792
Left Slip Surface Endpoint:	460.314, 766.085
Right Slip Surface Endpoint:	1332.691, 551.467
Resisting Moment:	2.59911e+09 lb-ft
Driving Moment:	9.93201e+08 lb-ft
Total Slice Area:	75801.6 ft2
Surface Horizontal Width:	872.377 ft
Surface Average Height:	86.8909 ft



Weaver Consultants Group	Project: ROYAL OAKS LANDFILL		
	Group: ENGINEERING	Scenario: TOTAL STRESS - CIRCULAR SECTION F-F	
	Drawn By: PREP BY: JBM CHKD BY:DEP	Company: WEAVER CONSULTANTS GROUP	
SLIDEINTERPRET 9.008	Date: 12/11/2023	File Name: TOTAL CIRCULAR F-F.slmd	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

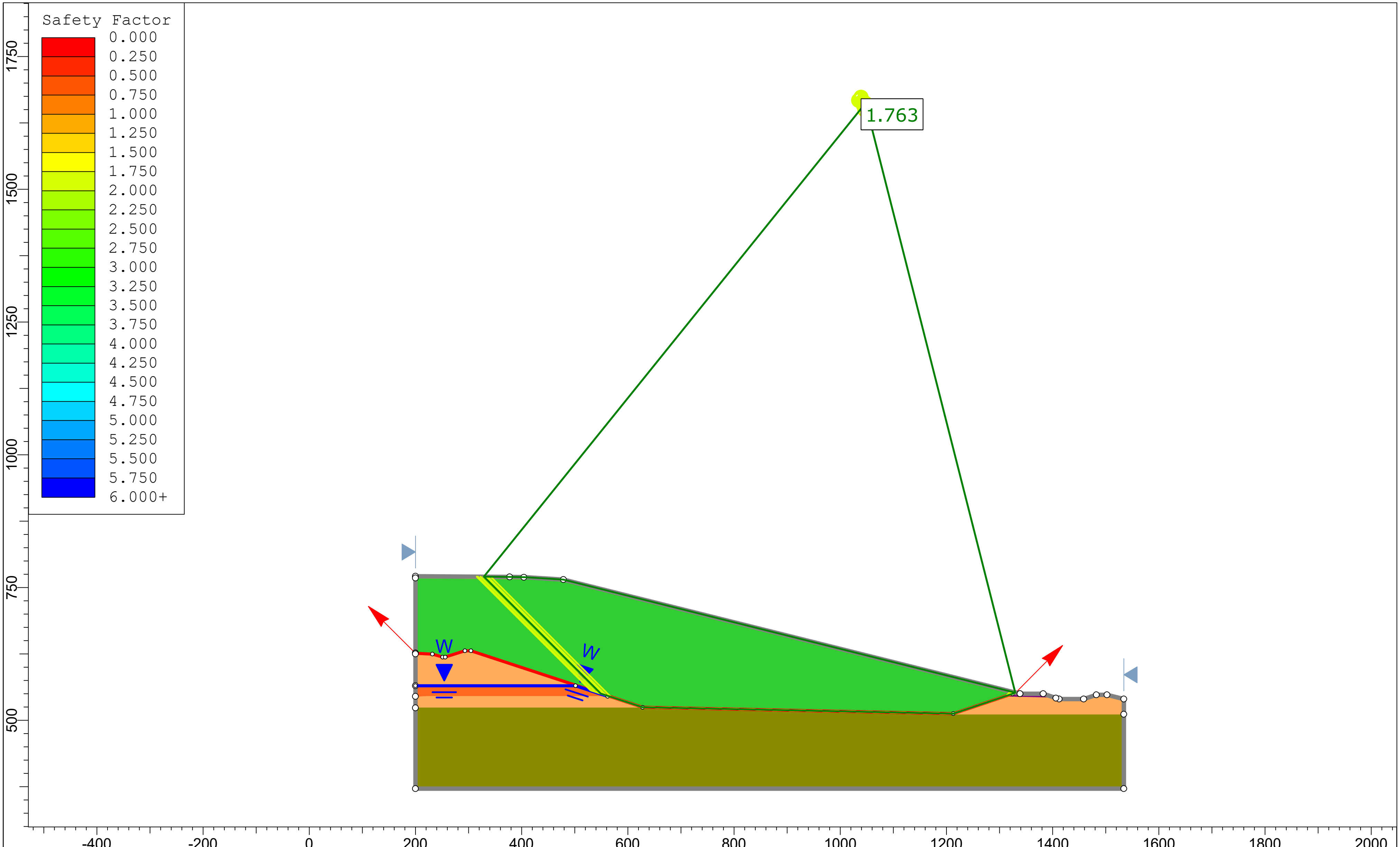
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	2.415540
Center:	886.958, 1338.267
Radius:	890.626
Left Slip Surface Endpoint:	200.059, 771.359
Right Slip Surface Endpoint:	1313.294, 556.313
Resisting Moment:	4.94986e+09 lb-ft
Driving Moment:	2.04918e+09 lb-ft
Total Slice Area:	184587 ft2
Surface Horizontal Width:	1113.23 ft
Surface Average Height:	165.812 ft



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project ROYAL OAKS LANDFILL	
Group ENGINEERING	*Scenario* PEAK STRESS - BLOCK SECTION F-F
Drawn By PREP BY: JBM CHKD BY:DEP	*Company* WEAVER CONSULTANTS GROUP
Date 12/11/2023	*File Name* PEAK BLOCK F-F.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	13
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

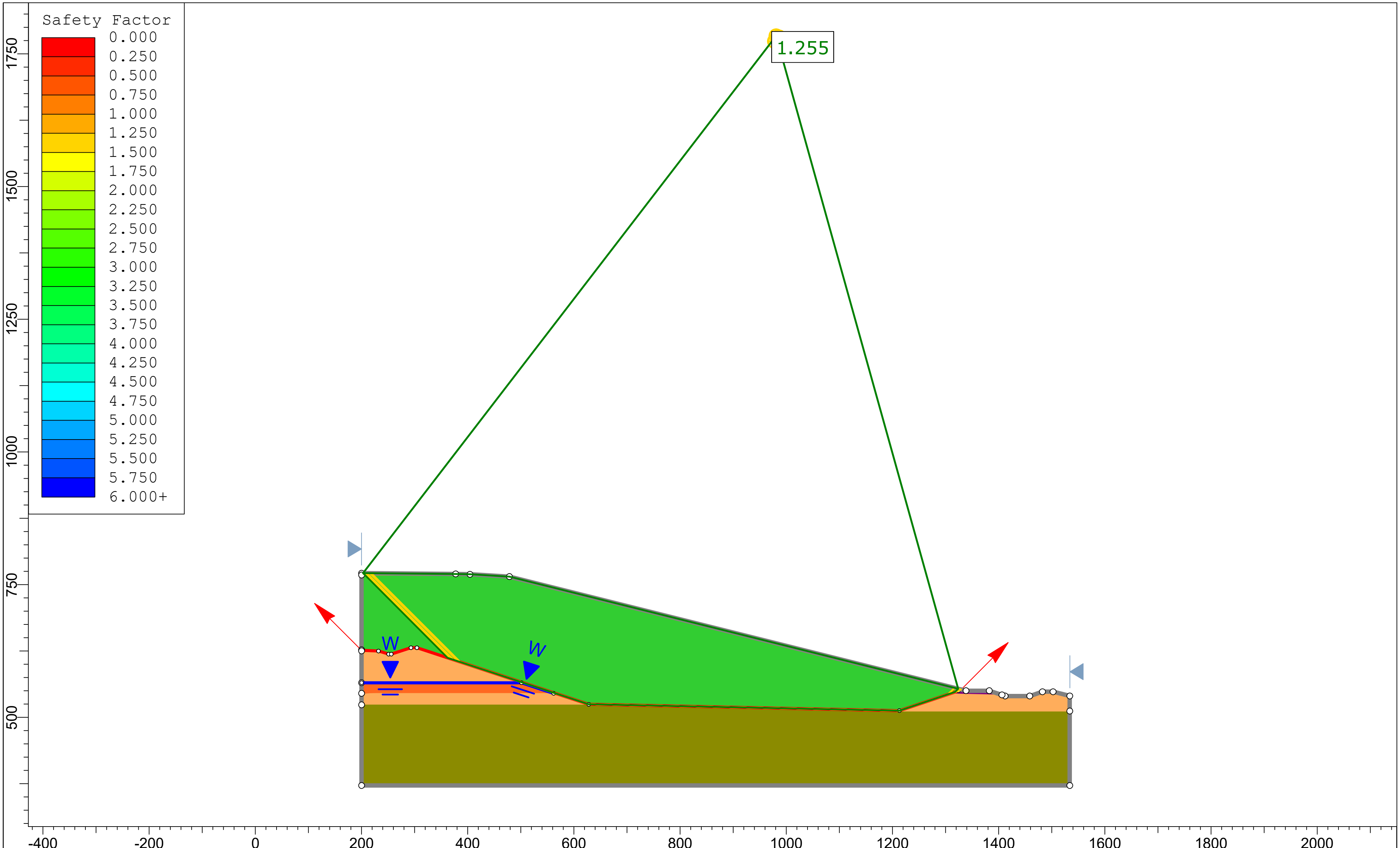
Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.763110
Axis Location:	1047.480, 1662.222
Left Slip Surface Endpoint:	328.836, 770.372
Right Slip Surface Endpoint:	1329.774, 552.197
Resisting Moment:	3.16434e+09 lb-ft
Driving Moment:	1.79475e+09 lb-ft
Total Slice Area:	122441 ft2
Surface Horizontal Width:	1000.94 ft
Surface Average Height:	122.326 ft



Weaver Consultants Group

SLIDEINTERPRET 9.008

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	RESIDUAL STRESS - BLOCK SECTION F-F
Drawn By	PREP BY: JBM CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	12/11/2023	*File Name*	RESIDUAL BLOCK F-F.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	120
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	120
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1

Shear Normal Functions

Name: User Defined 1

Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

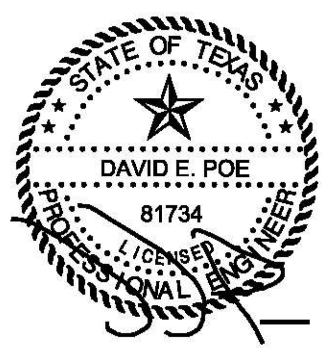
Method: bishop simplified

FS	1.255250
Axis Location:	980.932, 1783.556
Left Slip Surface Endpoint:	202.761, 771.339
Right Slip Surface Endpoint:	1323.804, 553.689
Resisting Moment:	2.71439e+09 lb-ft
Driving Moment:	2.16243e+09 lb-ft
Total Slice Area:	146175 ft2
Surface Horizontal Width:	1121.04 ft
Surface Average Height:	130.392 ft

APPENDIX IIIE-A-4

INFINITE SLOPE STABILITY ANALYSIS

Includes pages IIIE-A-4-1 through IIIE-A-4-12



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STABILITY ANALYSIS OF THE BOTTOM LINER AND FINAL COVER SYSTEMS

Required: Evaluate the stability of the bottom liner system components.

Procedure:

A. Bottom Liner System Stability - Sideslope and Anchor Trench Design
   1. Verify that the tensile stress in the liner system will be less than the yield stress by using Koerner's method for determination of shear stress in liner systems considering cohesion/adhesion forces.
   2. Provide liner anchor trench design considering pullout of the geomembrane.

B. Infinite Slope Stability Analysis
   1. Use Duncan and Buchignani's method for infinite stability analyses to evaluate the internal stability of the bottom liner and final cover systems using peak and residual shear strength values.

Contents:

- Interface and internal strength parameters are provided on Sheet IIIE-A-4-2.
- Verification that the tensile stress in the bottom liner system will be less than yield stress is provided on Sheets IIIE-A-4-3 through IIIE-A-4-6.
- Anchor trench design is provided on Sheets IIIE-A-4-7 through IIIE-A-4-8.
- Infinite stability analysis to evaluate the internal stability of the bottom liner and final cover systems is provided on Sheets IIIE-A-4-9 through IIIE-A-4-11.
- Figure E-7, Slope Stability Charts for Infinite Slopes is provided on Sheet IIIE-A-3-12.

References:

1. Koerner, Robert M., *Designing with Geosynthetics* , 3rd Edition, Prentice-Hall Inc., 1994.
2. Duncan, J.M. and Buchignani, A. L., *An Engineering Manual for Slope Stability Studies,* Department of Civil Engineering - University of California-Berkeley, 1975.
3. USACE, *Slope Stability* , Engineering and Design Manual, EM 1110-2-1902, October 31, 2003.
4. Koerner, Robert M., *Analysis and Design of Veneer Cover Soils* , 1998 Sixth International Conference of Geosynthetics.
5. Koerner, George R. and Narejo, Dhani, *Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces,* GRI Report #30, June 14, 2005.
6. Gilbert, Robert B., *Peak Versus Residual Strength for Waste Containment Systems,*
7. Proceedings of the 15th GRI Conference, December 13, 2001.
8. NAVFAC Design Manual 7.01, September 1986.

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APPENDIX IIIE-A-4
MODELING STRENGTH PARAMETER SUMMARY

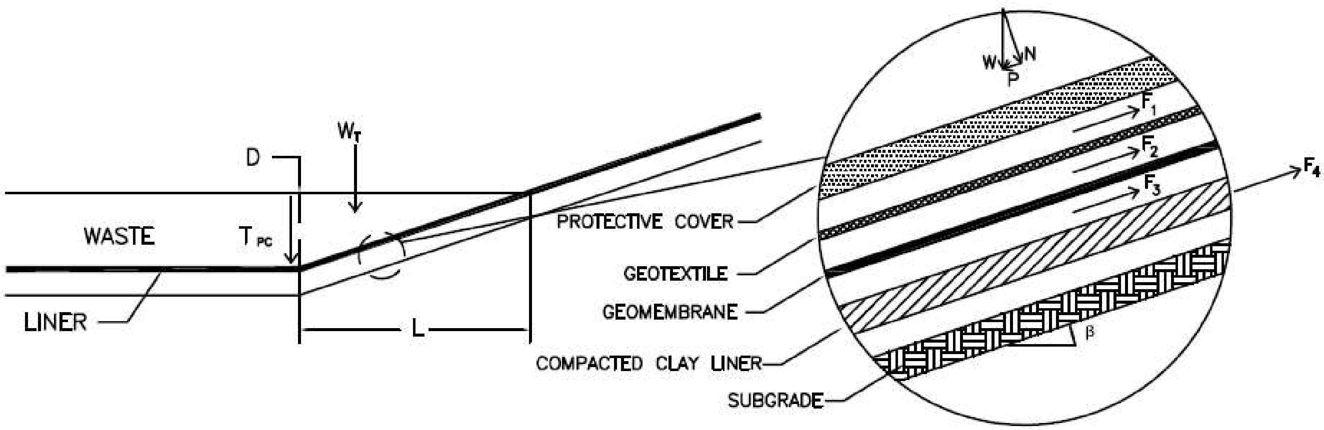
Foundation and Liner System Component/Interface		Cohesion/Adhesion, psf	Friction Angle, degree
Interbedded Sandy Clays and Silts, Silty Clayey Sands	Effective	800	19
	Total	1000	14
Sand (Silty, Clayey)	Effective	200	28
	Total	500	18
Compacted Fill	Effective	800	19
	Total	1000	14
Protective Cover/Single and Double-sided Geocomposite (also applicable to Underdrain)	Peak	200	20
	Residual	270	15
Double-sided Geocomposite/Textured Geomembrane	Peak	200	19
	Residual	120	10
Single-sided Geocomposite/Textured Geomembrane	Peak	0	13
	Residual	0	10
Textured Geomembrane/CCL	Peak	210	18
	Residual	50	14
Textured Geomembrane/GCL	Peak	850	25
	Residual	400	10
GCL Internal (Reinforced Only)	Peak	800	18
	Residual	380	11
GCL/Intermediate Cover Soils	Peak	100	18
	Residual	-	-

A. Liner System Stability - Anchor Trench Design

Note: The liner system includes a 2-foot-thick protective cover, double-sided geocomposite, textured geomembrane, and Geosynthetic Clay Liner (GCL) as an alternative to the Compacted Clay Liner (CCL) layer.

1. Verify that tensile stress in liner system is less than yield stress for the liner system.

GCL (Sideslope areas)



Definition of terms/variables:

W_E = Weight of equipment, lb/ft

Assume a Caterpillar D8T WH Track-Type Tractor

Operational Weight =	85,150	lb
Number of Tracks =	2	
Track Width =	1.84	ft

W_W = Weight of solid waste, lb/ft

W_{PC} = Weight of protective cover, lb/ft

W_T = Combined weight of equipment, solid waste, and protective cover, lb/ft

T_{PC} = Friction force on edge of protective cover, lb/ft

W = Net force of equipment, waste, and protective cover on liner system, lb/ft

N = Normal force on liner system, lb/ft

P = Shearing force on liner system, lb/ft

β = Slope angle, deg

F_n = Resisting force, lb/ft, calculated using the equation:
$(N * \tan(\Delta_n)) + (C_{an} * L / \cos(\beta))$

F_1 = Resistance of protective cover/double-sided geocomposite interface, lb/ft

F_2 = Resistance of double-sided geocomposite/textured geomembrane interface, lb/ft

F_3 = Resistance of textured geomembrane/compacted clay liner (CCL) interface, lb/ft

F_4 = Resistance of textured geomembrane/geosynthetic clay liner (GCL) interface, lb/ft

F_5 = Resistance of internal GCL, lb/ft

F_6 = Resistance of GCL/intermediate cover interface, lb/ft

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BOTTOM LINER SYSTEM STABILITY - SIDESLOPE AND ANCHOR TRENCH DESIGN

Δ_n = Interface friction angle of interface "n", deg
C_{an} = Adhesion of interface "n", psf
ϕ_n = Internal friction angle of material "n", deg
C_n = Cohesion of material "n", psf

γ_{was} = Unit weight of solid waste (including daily cover), pcf
D_{was} = Individual lift height, ft
ϕ_{was} = Internal friction angle of waste, deg
γ_{pc} = Unit weight of protective cover, pcf
D_{pc} = Thickness of protective cover, ft
ϕ_{pc} = Internal friction angle of protective cover, deg
L = Horizontal length of lift, ft

Parameters:

$\beta_{sideslope}$ =	18.43	deg	γ_{was} =	65	pcf
Δ_1 =	20	deg	D_{was} =	10	ft
C_{a1} =	200	psf	ϕ_{was} =	33	deg
Δ_2 =	19	deg	γ_{pc} =	108	pcf
C_{a2} =	200	psf	D_{pc} =	2	ft
Δ_3 =	18	deg	ϕ_{pc} =	18	deg
C_{a3} =	210	psf	L =	30	ft
Δ_4 =	25	deg			
C_4 =	850	psf			
ϕ_5 =	18	deg			
C_5 =	800	psf			
Δ_6 =	18	deg			
C_6 =	100	psf			

Note:
Interface friction strength values are selected conservatively from laboratory testing of similar material/interfaces.

Weight of Equipment

$$W_E = \frac{\text{Operational Weight}}{\text{Number of Tracks x Width of Track}}$$

W_E = 23,139 lb/ft

Weight of Solid Waste

$$W_W = \frac{D_{was} \text{ x } L \text{ x } \gamma_{was}}{2}$$

W_W = 9,750 lb/ft

Weight of Protective Cover

$$W_{PC} = D_{pc} \text{ x } \gamma_{pc} \text{ x } \frac{L}{\cos(\beta_{sideslope})}$$

W_{PC} = 6,830 lb/ft

Combined Weight of Equipment, Solid Waste, and Protective Cover,

$W_T = W_E + W_W + W_{PC}$ $\qquad W_T =$ 39,719 lb/ft

Friction Force on Edge of Protective Cover

$T_{PC} = k_o \times \sigma_v \times \tan \phi_{pc} \times D_{pc}$

where: $k_o = 1 - \sin \phi_{pc}$

$$\sigma_v = \frac{D_{pc} \times \gamma_{pc}}{2}$$

$T_{PC} =$ 48 lb/ft

Net Force of Equipment, Waste, and Protective Cover on Liner System

$W = W_T - T_{PC}$ $\qquad W =$ 39,670 lb/ft

$N = W \cos(\beta)$ $\qquad N =$ 37,636 lb/ft

$P_{sideslope} = W \sin(\beta)$ $\qquad P_{sideslope} =$ 12,542 lb/ft

LINER SYSTEM COMPONENTS

Protective Cover/Double-sided Geocomposite

Resistance of Protective Cover/Double-sided Geocomposite Interface = $F_1 =$ 20,023 lb/ft

$P_{sideslope} < F_1$ Therefore, protective cover soil/double-sided geocomposite is stable and a driving force equal to P is transferred to the next interface.

Double-sided Geocomposite/Textured Geomembrane Interface

Resistance of Double-sided Geocomposite/Textured Geomembrane Interface = $F_2 =$ 19,283 lb/ft

$P_{sideslope} < F_2$ Therefore, double-sided geocomposite/textured geomembrane interface is stable and a driving force equal to P is transferred to the next interface.

Textured Geomembrane/Compacted Clay Liner Interface

Resistance of Textured Geomembrane/Compacted Clay Liner Interface = $F_3 =$ 18,869 lb/ft

$P_{sideslope} < F_3$ Therefore, the textured geomembrane/compacted clay liner interface is stable and a driving force equal to P is transferred to the next interface.

Textured Geomembrane/GCL (Alternative)

Resistance of Textured Geomembrane/GCL Interface = $F_4 =$ 44,428 lb/ft

$P_{sideslope} < F_4$ Therefore, the textured geomembrane/GCL interface is stable and a driving force equal to P is transferred to the next interface.

Internal GCL

Resistance of Internal GCL Layer = F_5 = 37,526 lb/ft

$P_{sideslope} < F_5$ Therefore, the GCL internal strength is stable and a driving force equal to P is transferred to the next interface.

GCL/Intermediate Cover Soils Interface

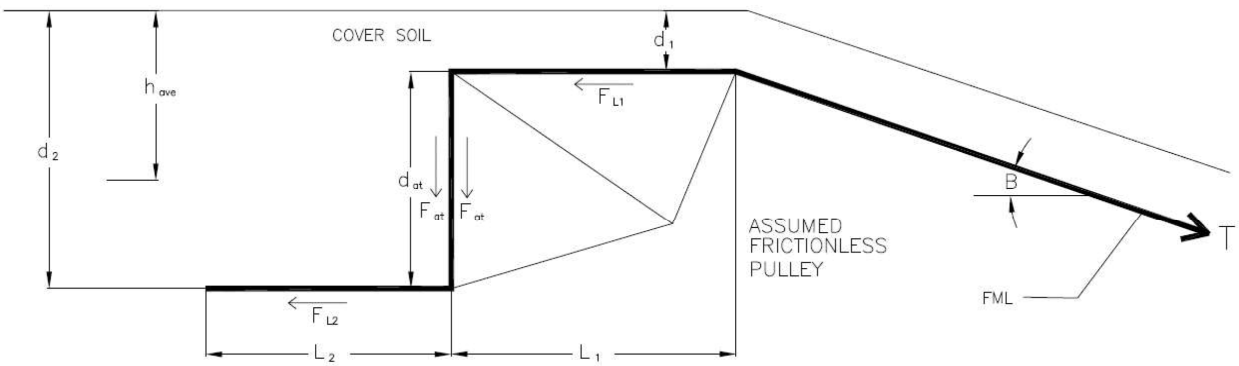
Resistance of GCL/Intermediate Cover Soils interface = F_6 = 15,391 lb/ft

$P_{sideslope} < F_6$ Therefore, GCL/Intermediate cover interface is stable and driving force equal to P is transferred to the next interface.

The Actual Tensile Force on liner system (T_{act}) = 0 lb/ft

2. Provide liner anchor trench design considering pullout of the geomembrane.

Force Diagram for Liner System (analyzed for worst case liner system interface)



$T = F_{L1} + F_{L2} + F_{at}$

Where T is the tensile force necessary for pullout

$F_{L1} = (q_1 \tan\Delta)(L_1)$

q_1 = Surcharge pressure = $d_1 \times \gamma_{soil}$
d_1 = Depth of soil, ft
γ_{soil} = Unit weight of soil, pcf
Δ = Interface friction angle, degrees
L_1 = Length of runout, ft

$F_{L2} = (q_2 \tan\Delta)(L_2)$

q_2 = Surcharge pressure = $d_2 \times \gamma_{soil}$
d_2 = Depth of soil, ft
γ_{soil} = Unit weight of soil, pcf
Δ = Interface friction angle, degrees
L_2 = Length of runout, ft

$F_{at} = (V \tan\Delta)(d_{at})$

V = Average horizontal stress = K_o x y
K_o = 1 - sin(r)
r = Internal friction angle of soil, degrees
y = γ_{soil} x h_{ave}
γ_{soil} = Unit weight of soil, pcf
h_{ave} = Average depth of trench, ft
Δ = Interface friction angle, degrees
d_{at} = Depth of trench, ft

Parameters:

γ_{soil} =	108	pcf
Δ =	16	deg
r =	18	deg

d_1 =	2.0	ft
L_1 =	6.0	ft
d_2 =	4.0	ft
L_2 =	2.0	ft
d_{at} =	2.0	ft
h_{ave} =	2.0	ft

Calculations:

F_{L1} = 371.6 lb / ft

F_{L2} = 247.7 lb / ft

F_{at} = 85.6 lb / ft

T = 705.0 lb / ft

Compare force required for pullout (T) with the actual tensile force in the geomembrane from Part 1:

T = 705 lb / ft

T_{act} = 0 lb / ft

$T > T_{act}$

Therefore, the runout lengths are sufficient to prevent pullout.

B. Infinite Slope Stability Analysis

Interface friction strength values are selected conservatively from laboratory testing of similar material/interfaces. Prior to construction, laboratory tests will be performed to verify the assumed values for interface adhesion (or cohesion) and friction angle using project-specific soil and synthetic materials. The interface friction testing will be performed for the specific conditions analyzed. If test results differ from the assumed values, this analysis will be updated for acceptable factor of safety values using the procedure presented in the following sections.

The liner and final cover systems are described below.

LINER SYSTEM

The liner system includes a 2-foot-thick protective cover, double-sided geocomposite, textured geomembrane, and a 2-foot-thick compacted clay liner (CCL) or GCL. The calculations performed herein incorporate the GCL alternative.

FINAL COVER SYSTEM

The final cover system includes a 1-foot-thick erosion/vegetation support layer, a double-sided geocomposite (sideslopes), textured geomembrane, and a 18-inch-thick compacted clay infiltration layer or GCL (as an alternative).

1. Use Duncan and Buchignani's method for infinite stability analyses to evaluate the internal stability of the liner, overliner, and final cover systems using peak and residual shear strength values.

The factor of safety is calculated using the following equation:

$$F.S. = A\frac{\tan\Delta}{\tan\beta} + B\frac{C_a}{\gamma H}$$

where:

Δ = Interface friction angle, deg
C_a = Adhesion, psf
β = Slope angle, deg
A = Parameter A from chart on page IIIE-A-4-12
B = Parameter B from chart on page IIIE-A-4-12
γ = Unit weight of soil, pcf
H = Thickness of material above interface, ft

An example using the protective cover/double-sided geocomposite interface of the liner system is provided below.

A. Define the shear strength parameters (peak shear strength parameters will be used for this example).

$\Delta =$ 20 deg
$C_a =$ 200 psf

B. Calculate the pore pressure, r_u, using the following equation:

$$r_u = (T \times \gamma_w \times \cos^2\beta) / (H \times \gamma)$$

where:
H = Thickness of material above interface, ft
γ_w = Unit weight of water, pcf
β = Slope angle, deg
T = Maximum head above interface, ft
γ = Unit weight of soil, pcf

H = 2 ft
γ_w = 62.4 pcf
β = 18.43 deg (3H:1V)
T = 0 ft
γ = 120 pcf

r_u = 0.00

Since T=0, there is no pore pressure build-up in the protective cover. If the soil material is assumed to be saturated, use a unit weight of 125 pcf for soil.

C. Calculate the slope ratio, b.

b = cot β = 3.0

D. Using r_u and b, determine Parameters A and B from the charts on page IIIE-4-A-12.

A = 1.0
B = 3.3

E. Calculate the factor of safety and compare against the minimum recommended factor of safety.

F.S. =	3.84	>	$F.S._{min}$ =	1.5

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INFINITE SLOPE STABILITY ANALYSIS SUMMARY

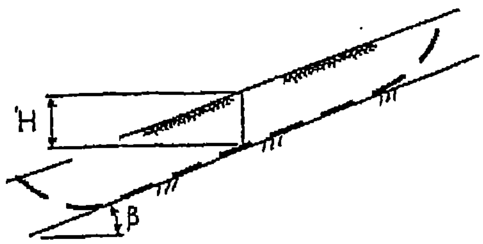
Component/Interface	Strength Parameters: Cohesion/Adhesion (psf)		Friction Angle (deg)		H (ft)	γ (pcf)	β (deg)	T (ft)	r_u	b	A	B	Factor of Safety Generated		Recommended Minimum Factor of Safety		Acceptable Factor of Safety	
	Peak	Residual	Peak	Residual									Peak	Residual	Peak	Residual	Peak	Residual
Liner System (3H:1V Maximum Slope)																		
Composite Liner																		
Protective Cover/Double-sided Geocomposite	200	270	20	15	2	120	18.43	0	0.00	3.0	1.0	3.3	3.84	4.52	1.5	1.0	YES	YES
Double-sided Geocomposite/Textured Geomembrane	200	120	19	10	2	108	18.43	0	0.00	3.0	1.0	3.3	4.09	2.36	1.5	1.0	YES	YES
Textured Geomembrane / CCL	210	50	18	14	2	108	18.43	0	0.00	3.0	1.0	3.3	4.18	1.51	1.5	1.0	YES	YES
Textured Geomembrane / GCL	850	400	25	10	2	108	18.43	0	0.00	3.0	1.0	3.3	14.39	6.64	1.5	1.0	YES	YES
GCL Internal (reinforced only)	800	380	18	11	2	108	18.43	0	0.00	3.0	1.0	3.3	13.20	6.39	1.5	1.0	YES	YES
GCL/Foundation Soils	100	-	18	-	2	108	18.43	0	0.00	3.0	1.0	3.3	2.50	NA	1.5	1.0	YES	YES

CCL = Compacted Clay Liner

GCL = Geosynthetic Clay Liner

Component/Interface	Strength Parameters[1]: Cohesion/Adhesion (psf)		Friction Angle (deg)		H (ft)	γ (pcf)	β (deg)	T (ft)	r_u	b	A	B	Factor of Safety Generated		Recommended Minimum Factor of Safety		Acceptable Factor of Safety	
	Peak	Residual	Peak	Residual									Peak	Residual	Peak	Residual	Peak	Residual
Final Cover System (4H:1V Maximum Sideslope)																		
(Saturated Erosion Layer)																		
Erosion/Vegetation Layer/Double-sided Geocomposite	200	270	20	15	1	130	14.04	1	0.45	4.0	0.45	3.75	6.42	8.27	1.5	1.0	YES	YES
Double-sided Geocomposite/Textured Geomembrane	200	120	19	10	1	130	14.04	1	0.45	4.0	0.45	3.75	6.39	3.78	1.5	1.0	YES	YES
Textured Geomembrane / CCL	210	50	18	14	1	130	14.04	1	0.45	4.0	0.45	3.75	6.64	1.89	1.5	1.0	YES	YES
Textured Geomembrane / GCL	850	400	25	10	1	130	14.04	1	0.45	4.0	0.45	3.75	25.36	11.86	1.5	1.0	YES	YES
GCL Internal (reinforced only)	800	380	18	11	1	130	14.04	1	0.45	4.0	0.45	3.75	23.66	11.31	1.5	1.0	YES	YES
GCL/Intermediate Cover Soils	100	-	18	-	1	130	14.04	1	0.45	4.0	0.45	3.75	3.47	NA	1.5	1.0	YES	NA
Minimum Required Interface Friction Strength Values for Conformance Testing (All Interfaces or Stack Testing)	**65**	**35**	**6**	**5**	**1**	**130**	**14.04**	**1**	**0.45**	**4.0**	**0.45**	**3.75**	**2.06**	**1.17**	**1.5**	**1.0**	**YES**	**YES**

1. Shear strength values shown in above table are provided as example only. The "Minimum Required Interface Friction Strength Values for Conformance Testing" in above table shall be used in assessing the adequacy of final cover system components (by stack testing or shear testing of the individual interaces) prior to implementation into the project. In the event conformance test results do not meet the standard above, additional infinite stability analyses will be required.



γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept

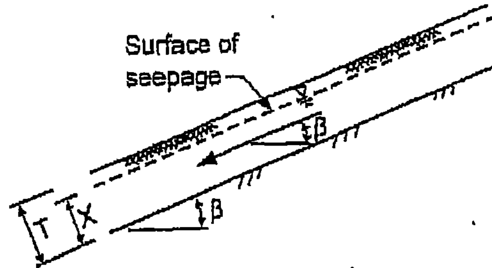
ϕ' = friction angle

r_u = pore pressure ratio = $u/\gamma H$

u = pore pressure at depth H

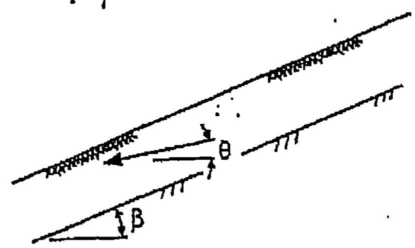
Steps:

1. Determine r_u from measured pore pressure or formulas at right.
2. Determine A and B from charts below.
3. Calculate $F = A\dfrac{\tan\phi'}{\tan\beta} + B\dfrac{c'}{\gamma H}$



Seepage parallel to slope

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta$$



Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta}$$

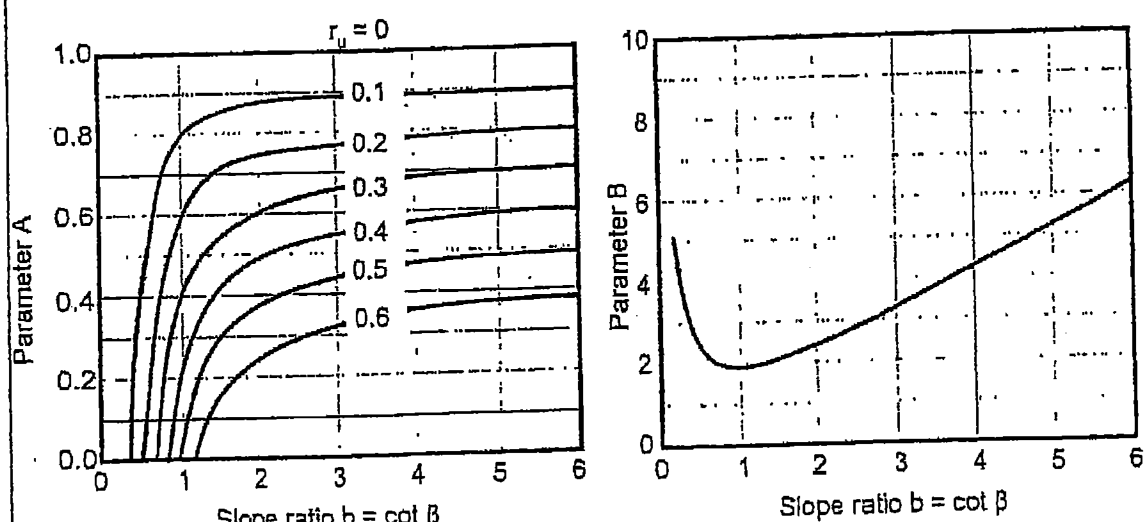
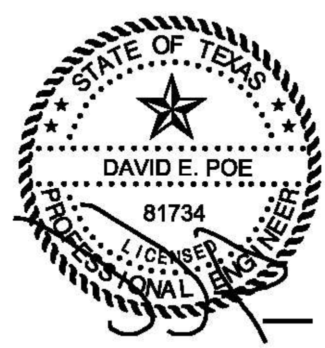


Figure E-7. Slope stability charts for infinite slopes (after Duncan, Buchianani, and DeWet 1987)

APPENDIX IIIE-A-5

INTERFACE SHEAR STRENGTH CONFORMANCE TESTING REQUIREMENTS

Includes Pages IIIE-A-5-1 through IIIE-A-5-18



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INTERFACE SHEAR STRENGTH CONFORMANCE TESTING REQUIREMENTS

Prior to each construction event, interface shear strength conformance testing will be required for the specific soils and geosynthetics to be incorporated into the project. The required conformance testing requirements have been established for the project based on stability analyses performed for the expansion, as presented in Appendix IIIE-A. Based on geometry and stability modeling results, Section F-F (see Appendix IIIE-A-3) was selected as the condition to demonstrate the adequacy of the assumed conformance testing limits, and the stability analyses was iterated to find the minimum factors of safety were met (i.e., FS=1.5 for total stress and FS=1.1 for residual stress conditions). The results of this analysis are presented on Sheets IIIE-A-5-5 through IIIE-A-5-16. Note that confirmation testing was also performed of the interim conditions presented as Section D-D in Appendix IIIE-A-2 to demonstrate that the conformance values presented in Tables IIIE-A-5-1 and IIIE-A-5-2 provide adequate factors of safety.

The global stability analysis results represent the minimum interface shear strength required during future conformance testing. Note that separate values are provided for cell floor (Table IIIE-A-5-1) and cell sideslope (3H:1V) (Table IIIE-A-5-2) liners. The values in the following table were developed to represent the minimum shear strength at the geosynthetic interfaces required during conformance testing.

Table IIIE-A-5-1
Minimum Shear Strength Values for Future Interface Shear Strength Conformance Testing – Cell Bottom Liners

Peak Shear Strength Parameters		Residual Shear Strength Parameters[1]		Average Waste Unit Weight (lb/cf)
Cohesion/ Adhesion (psf)	Friction Angle (degrees)	Cohesion/ Adhesion (psf)	Friction Angle (degrees)	
0	12	0	9.5	65

Table IIIE-A-5-2
Minimum Shear Strength Values for Future Interface Shear Strength Conformance Testing – Cell Sideslope (3H:1V) Liners

Peak Shear Strength Parameters		Residual Shear Strength Parameters[1]		Average Waste Unit Weight (lb/cf)
Cohesion/ Adhesion (psf)	Friction Angle (degrees)	Cohesion/ Adhesion (psf)	Friction Angle (degrees)	
0	18	0	10	65

[1] Residual shear strength values (i.e., large displacement) will be determined based on 3 inches displacement during laboratory shear testing.

Graphs of the shear strength envelopes represented by the values in the above tables (for both Peak and Residual Stress Conditions) are presented on Sheets IIIE-A-5-3 through IIIE-A-5-6. Future laboratory conformance test results will be required to plot within the shaded zone on the graph, with test-specific shear strength values calculated assuming a waste density of 65 lb/cf (consistent with the values used for the graph) and strength parameters developed within the laboratory.

The above values may be used for stack testing of multiple geosynthetic and clay liner layers or testing of individual interfaces. A stack test (i.e., multiple geosynthetic or soil layers tested concurrently) meeting the above strength requirements demonstrates conformance of the individual materials used in the stack. Internal shear strength testing of GCL, clay liner, and protective cover will be performed as stand-alone tests, although interfaces with other materials may be performed as a stack test.

In the event that the confirmation testing minimum values presented in Tables IIIE-A-5-1 and IIIE-A-5-2 are not achieved in the laboratory, additional stability modeling may be performed in order to demonstrate that the proposed liner materials meet the minimum factors of safety (for both peak and residual stress conditions) set forth in Appendix IIIE.

ROYAL OAKS LANDFILL
0120-067-11-106
APPENDIX IIIE-A-5
GEOSYNTHETIC INTERFACE SHEAR STRENGTH TESTING REQUIREMENTS
PEAK STRESS PARAMETERS - CELL FLOOR LINER (ONLY)

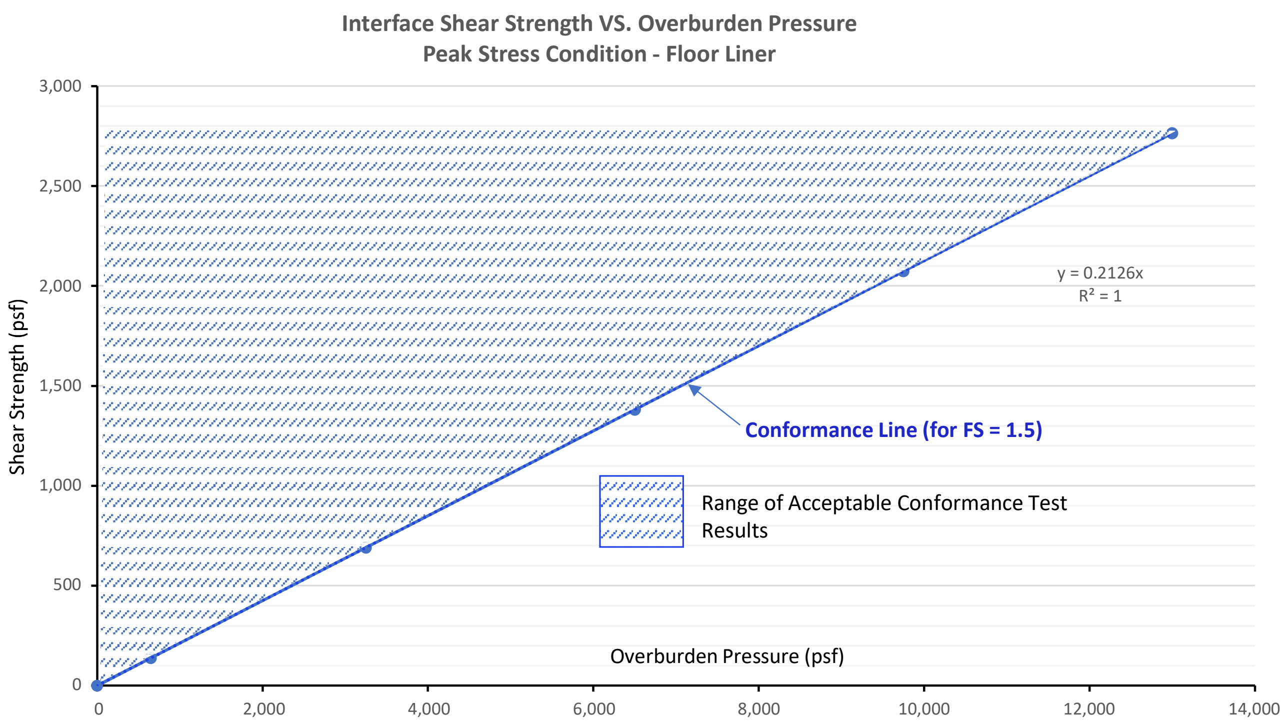
Minimum Allowable Peak Shear Strength Parameters[1]

Friction Angle (φ, degrees)	12
Cohesion (c, psf)	0
Unit Weight of Overburden Waste (γ_{waste}, pcf)	65

Peak Shear Strength Calculations[2]

Fill height (H, ft)	Overburden Pressure (psf)	Peak Shear Strength[3] (psf)
0	0	0
10	650	138
50	3,250	691
100	6,500	1,382
150	9,750	2,072
200	13,000	2,763

200 feet represents the maximum waste height at the landfill.



Notes

1. Values shown are minimums developed from global stability analysis, and were used to develop the conformance graph shown above.
2. Shear strength values calculated based on an overburden stress of 65 pounds per cubic foot.
3. Shear Strength = Cohesion (c) + (H) x (γ_{waste})(tanφ)
4. Graph applicable to cell floor liner only. Sideslope (3H:1V) liner system not addressed by this graph.

ROYAL OAKS LANDFILL
0120-076-11-106
APPENDIX IIIE-A-5
GEOSYNTHETIC INTERFACE SHEAR STRENGTH TESTING REQUIREMENTS
RESIDUAL STRESS PARAMETERS - CELL FLOOR LINER (ONLY)

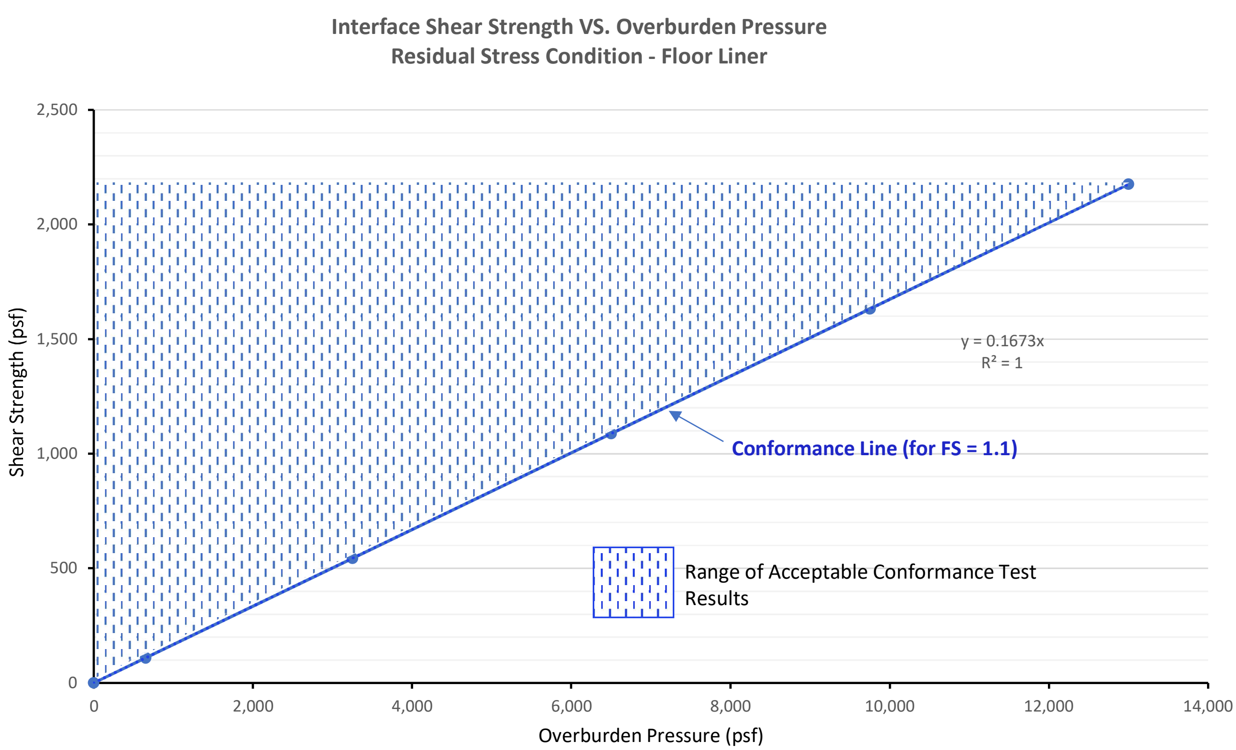
Minimum Allowable Residual Shear Strength Parameters[1]

Friction Angle (φ, degrees)	9.5
Cohesion (c, psf)	0
Unit Weight of Overburden Waste (γ_{waste}, pcf)	65

Residual Shear Strength Calculations[2]

Fill height (H, ft)	Overburden Pressure (psf)	Residual Shear Strength[3] (psf)
0	0	0
10	650	109
50	3,250	544
100	6,500	1,088
150	9,750	1,632
200	13,000	2,175

200 feet represents the maximum waste height at the landfill.



Notes

1. Values shown are minimums developed from global stability analysis, and were used to develop the conformance graph shown above.
2. Shear strength values calculated based on an overburden stress of 65 pounds per cubic foot.
3. Shear Strength = Cohesion (c) + (H) x (γ_{waste})(tanφ)
4. Graph applicable to cell floor liner only. Sideslope (3H:1V) liner system not addressed by this graph.

ROYAL OAKS LANDFILL
0120-067-11-106
APPENDIX IIIE-A-5
GEOSYNTHETIC INTERFACE SHEAR STRENGTH TESTING REQUIREMENTS
PEAK STRESS PARAMETERS - CELL 3H:1V SIDESLOPES (ONLY)

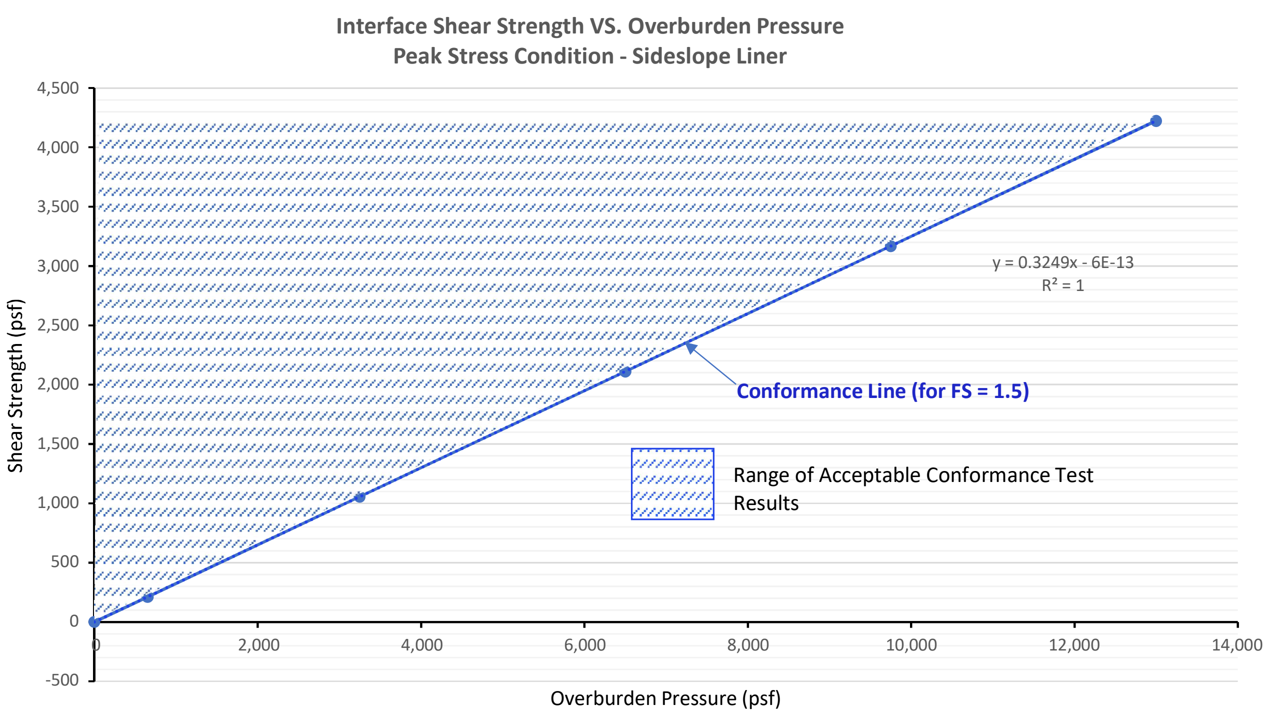
Minimum Allowable Peak Shear Strength Parameters[1]

Friction Angle (φ, degrees)	18
Cohesion (c, psf)	0
Unit Weight of Overburden Waste (γ_{waste}, pcf)	65

Peak Shear Strength Calculations[2]

Fill height (H, ft)	Overburden Pressure (psf)	Peak Shear Strength[3] (psf)
0	0	0
10	650	211
50	3,250	1,056
100	6,500	2,112
150	9,750	3,168
200	13,000	4,224

200 feet represents the maximum waste height at the landfill.



Notes

1. Values shown are minimums developed from global stability analysis, and were used to develop the conformance graph shown above.
2. Shear strength values calculated based on an overburden stress of 65 pounds per cubic foot.
3. Shear Strength = Cohesion (c) + (H) x (γ_{waste})(tanφ)
4. Graph applicable to cell sideslope (3H:1V) liner only. Cell floor liners not addressed by this graph.

ROYAL OAKS LANDFILL
0120-076-11-106
APPENDIX IIIE-A-5
GEOSYNTHETIC INTERFACE SHEAR STRENGTH TESTING REQUIREMENTS
RESIDUAL STRESS PARAMETERS - CELL 3H:1V SIDESLOPES (ONLY)

Minimum Allowable Residual Shear Strength Parameters[1]

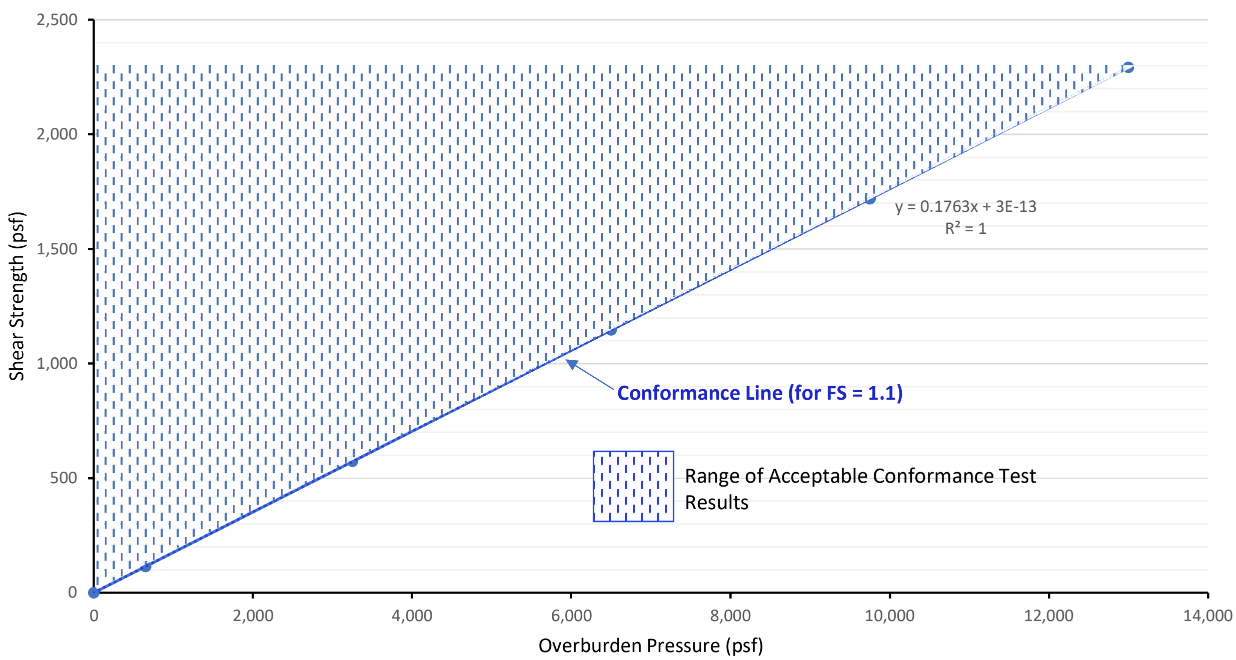
Friction Angle (φ, degrees)	10
Cohesion (c, psf)	0
Unit Weight of Overburden Waste (γ_{waste}, pcf)	65

Residual Shear Strength Calculations[2]

Fill height (H, ft)	Overburden Pressure (psf)	Residual Shear Strength[3] (psf)
0	0	0
10	650	115
50	3,250	573
100	6,500	1,146
150	9,750	1,719
200	13,000	2,292

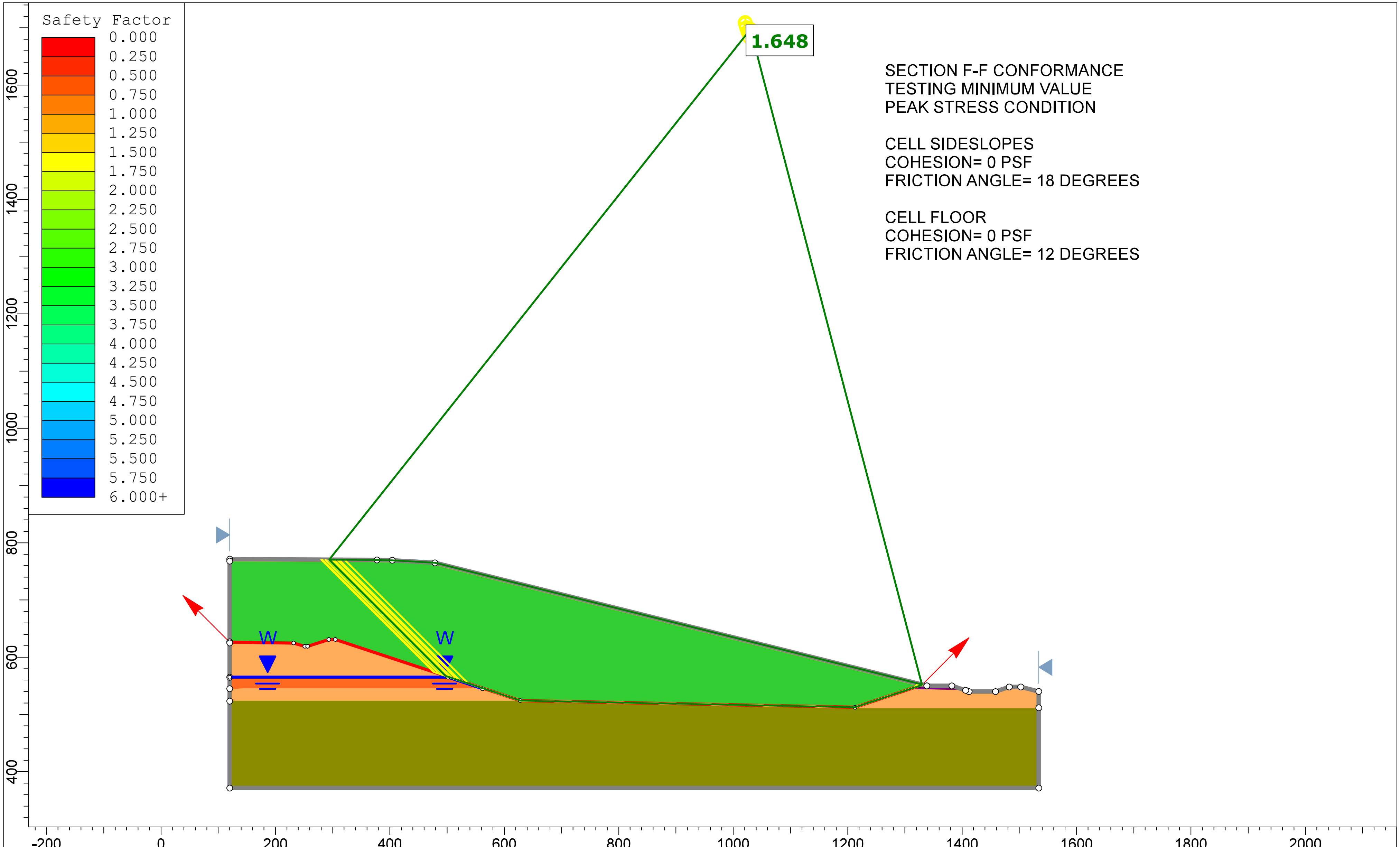
200 feet represents the maximum waste height at the landfill.

Interface Shear Strength VS. Overburden Pressure
Residual Stress Condition - Sideslope Liner



Notes

1. Values shown are minimums developed from global stability analysis, and were used to develop the conformance graph shown above.
2. Shear strength values calculated based on an overburden stress of 65 pounds per cubic foot.
3. Shear Strength = Cohesion (c) + (H) x (γ_{waste})(tanφ)
4. Graph applicable to cell sideslope (3H:1V) liner only. Cell floor liners not addressed by this graph.



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	Scenario	PEAK STRESS - BLOCK SECTION F-F
Drawn By	PREP BY: MB CHKD BY:DEP	Company	WEAVER CONSULTANTS GROUP
Date	4/18/2024	File Name	PEAK BLOCK F-F.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	18
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	12
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

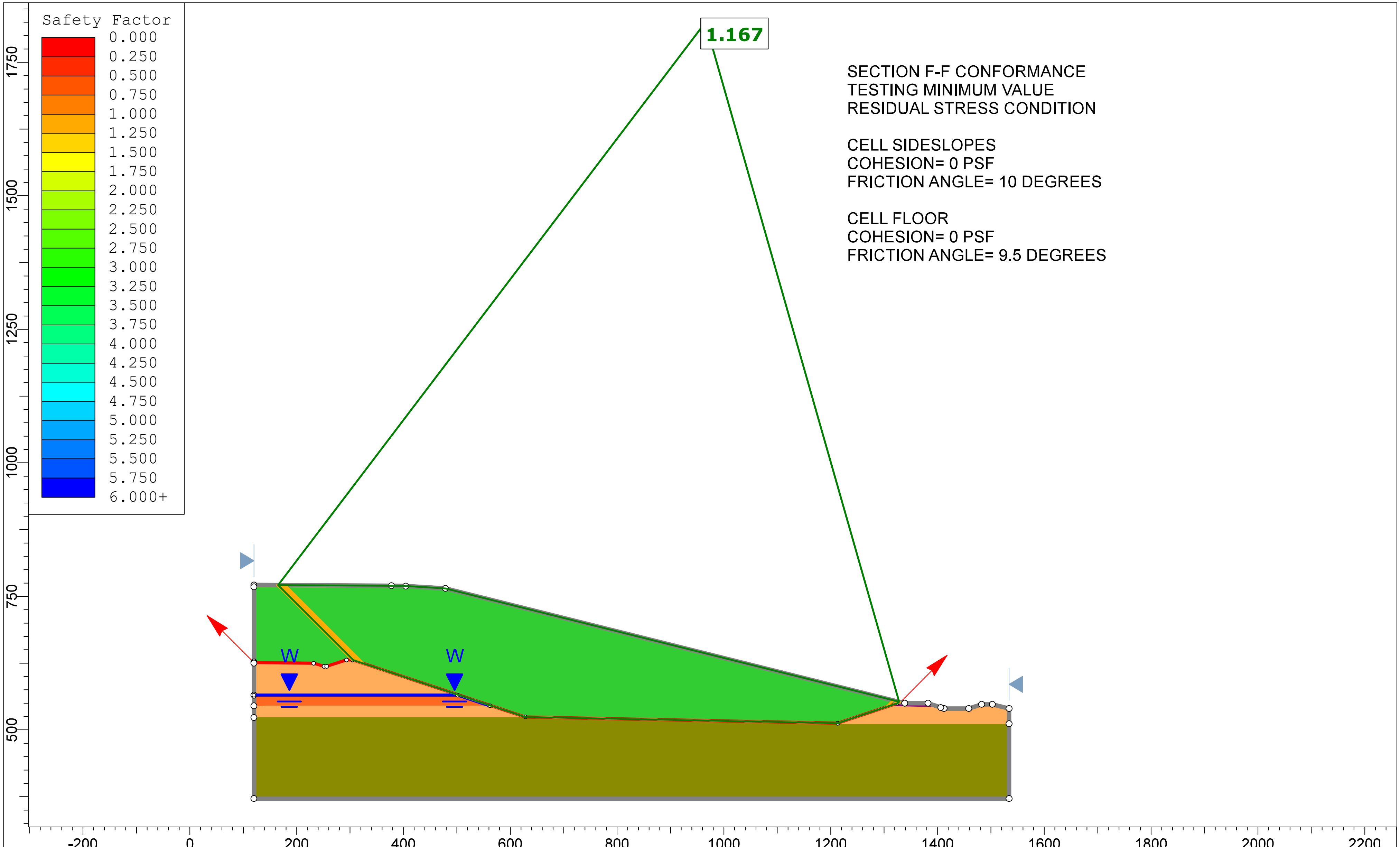
Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

Global Minimums

Method: bishop simplified

FS	1.648000
Axis Location:	1030.042, 1697.165
Left Slip Surface Endpoint:	293.888, 770.441
Right Slip Surface Endpoint:	1329.729, 552.208
Resisting Moment:	3.16623e+09 lb-ft
Driving Moment:	1.92126e+09 lb-ft
Total Slice Area:	129809 ft2
Surface Horizontal Width:	1035.84 ft
Surface Average Height:	125.318 ft



Weaver Consultants Group

SLIDEINTERPRET 9.018

Project	ROYAL OAKS LANDFILL		
Group	ENGINEERING	*Scenario*	RESIDUAL STRESS - BLOCK SECTION F-F
Drawn By	PREP BY: MB CHKD BY:DEP	*Company*	WEAVER CONSULTANTS GROUP
Date	4/18/2024	*File Name*	RESIDUAL BLOCK F-F.slmd

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Materials

FC Composite	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	200
Friction Angle [deg]	19
Water Surface	None
Ru Value	0
WASTE	
Color	
Strength Type	Shear Normal function
Unsaturated Unit Weight [lbs/ft3]	65
Saturated Unit Weight [lbs/ft3]	65
Water Surface	None
Ru Value	0
LINER (TGM-DSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	10
Water Surface	None
Ru Value	0
LINER (TGM-SSGC)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	108
Saturated Unit Weight [lbs/ft3]	115
Cohesion [psf]	0
Friction Angle [deg]	9.5
Water Surface	None
Ru Value	0
COMPACT FILL	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	123
Saturated Unit Weight [lbs/ft3]	132
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT	
Color	

Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	None
Ru Value	0
INTERBEDDED SANDY CLAY AND SILT (PERCHED WT)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	115
Saturated Unit Weight [lbs/ft3]	130
Cohesion [psf]	1000
Friction Angle [deg]	14
Water Surface	Water Table
Hu Value	1
SAND (SILTY/CLAYEY)	
Color	
Strength Type	Mohr-Coulomb
Unsaturated Unit Weight [lbs/ft3]	120
Saturated Unit Weight [lbs/ft3]	135
Cohesion [psf]	500
Friction Angle [deg]	18
Water Surface	None
Ru Value	0

Shear Normal Functions

Name: User Defined 1

Effective Normal (psf)	Shear (psf)
0	500
208	500
417	500
625	500
626	406.53
834	541.61
1040	675.38
1250	811.76
2500	1623.52
25000	16235.2

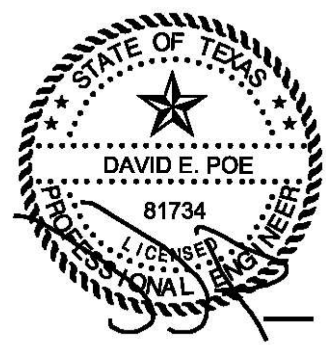
Global Minimums

Method: bishop simplified

FS	1.166800
Axis Location:	965.038, 1824.548
Left Slip Surface Endpoint:	165.252, 771.121
Right Slip Surface Endpoint:	1327.908, 552.663
Resisting Moment:	2.6471e+09 lb-ft
Driving Moment:	2.26868e+09 lb-ft
Total Slice Area:	151779 ft2
Surface Horizontal Width:	1162.66 ft
Surface Average Height:	130.545 ft

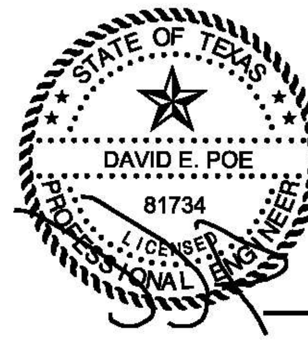
APPENDIX IIIE-B

SETTLEMENT AND HEAVE ANALYSIS



05/20/2024

CONTENTS



05/20/2024

INTRODUCTION

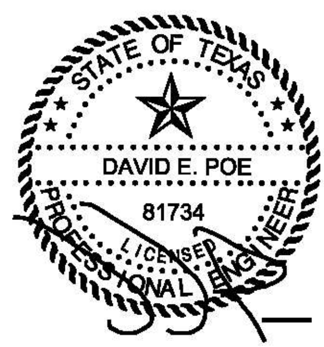
This appendix includes the settlement, strain, and heave analyses for the foundation soils and the settlement and strain analyses for the final cover system. The following three appendices are developed for the foundation soils and final cover, respectively.

- Appendix IIIE-B-1 includes the settlement and strain analyses for the foundation soils.
- Appendix IIIE-B-2 includes the settlement and strain analyses for the final cover system.
- Appendix IIIE-B-3 includes the heave analysis for the foundation.

APPENDIX IIIE-B-1

FOUNDATION/BOTTOM LINER SETTLEMENT ANALYSIS

Includes pages IIIE-B-1-1 through IIIE-B-1-38



05/20/2024

ROYAL OAKS LANDFILL
0120-76-11-106
APPENDIX IIIE-B-1
BOTTOM LINER SYSTEM
SETTLEMENT AND STRAIN

<u>Required:</u> Determine the post-settlement slope of the bottom liner system and verify that the strain induced on the bottom liner system due to settlement is within acceptable limits.

<u>Method:</u>

A. Estimate settlement of subsurface below the bottom liner system. Settlement calculated by consolidation theory using SETTLE3. The program uses the Boussinesq method to approximate 2 dimensional consolidation of the foundation strata.

1. Waste filling and liner and final cover installation will result in loading of the foundation soils causing consolidation and potential differential settlement. The magnitude of consolidation and settlement will be a function of the net stress increase and properties of the foundation soils. Net stress increase is assumed to result from loading of the foundation soils during landfilling.

2. Modeling was performed using SETTLE3, RocScience, Inc (2021). Procedures are described below. Primary settlement (only) was analyzed. Secondary settlement within the shale formation is assumed negligible.

2a. The subgrade conditions were developed from the available boring logs, normalized to the excavation grades proposed for the landfill. Normalization refers to inputting boring information from the proposed excavation grade downward, based on recorded elevations shown on the logs. The borehole locations used to establish the subgrade conditions are shown on Sheet IIIE-B-1-8. For the analysis vertical loads were applied for the closed condition at the locations shown on Sheet IIIE-B-1-9.

2b. Load polygons were developed for input into SETTLE3, for the loading conditions proposed for the landfill. Vertical loads were estimated for each polygon vertex (at the locations shown on Sheet IIIM-B-1-9), and this information inputted into SETTLE3. The load polygons are shown on Sheet IIIM-B-1-10. Loads at the polygon vertices were estimated based on waste fill height and an assumed unit weight of waste (varies based on total waste depth).

2c. The SETTLE3 program calculated total settlement based on Boussinesq equation. The model output files are included in Appendix IIIE-B-1-A. The settlement isopach created by SETTLE3 is presented on Sheet IIIE-B-1-11.

3. Utilizing the settlement values calculated by SETTLE3, post-settlement slopes and strains are calculated, as presented on Sheets IIIE-B-1-5 through IIIE-B-1-7. An example of the calculation method is presented on Sheets IIIE-B-1-3 and IIIE-B-1-4.

Description of Contents:

Sheet IIIE-B-1-1 presents the method used for the settlement analyses.
Sheets IIIE-B-1-3 and IIIE-B-1-4 present the method of analysis for post-settlement slopes and strain between designated Evaluation Points.

Sheet IIIE-B-1-8 presents the borehole locations used to develop the subsurface profile for the SETTLE3 model.

Sheet IIIE-B-1-9 presents the final configuration load locations incorporated into the SETTLE3 model.

Sheet IIIE-B-1-10 presents the SETTLE3 load polygons incorporated into model.
Sheet IIIE-B-1-11 presents the SETTLE3 settlement isopach.
Sheet IIIE-B-1-12 presents the Evaluation Points and Evaluation Lines used in analysis of the strain and post-settlement slopes for the bottom liner.

Tables 1A and 1B present the settlement results at the Evaluation Points and distances between the Evaluation Points.

Table 2 presents slope and strain summary results from the analysis.

References:

1. Sowers, George F., Settlement of Solid Waste, *Proceedings of the Eighth International Conference on Soil Mechanics and Foundations Engineering, 1973* .
2. Quian, Xuede, R.M. Koerner, D. H. Gray, Geotechnical Aspects of Landfill Design and Construction, Prentice-Hall, Inc., New Jersey, 2002.
3. Koerner, Robert M., Designing with Geosynthetics, Third Edition. Prentice-Hall, New Jersey, 1994.
4. Acar, Yalcin B. & Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.
5. Zornberg, Jorge G., et al., *Retention of Free Liquids in Landfills Undergoing* Vertical Expansion, Journal of Geotechnical and Geoenvironmental Engineering, July 1999.
6. Fassett, Jeffrey B., et al., Geotechnical Properties of Municipal Solid Wastes and Their Use in Landfill Design, Waste Tech, 1994.
7. SETTLE3, Version 5.009 Copyright © 2008-2021 Rocscience Inc.
8. Beggs, Ian D. et al, Assessment of Maximum Allowable Strains in Polyethylene and Polypropylene Geomembranes, Geo-Frontiers Congress, Austin, TX, 2005.

Solution: **A) Estimate settlement of bottom liner system.**

The SETTLE3 model was used to determine waste loading-induced settlement in the bottom liner system. The vertices and polygons developed for the modeling are shown on Sheet IIIE-B-1-10. The analysis was performed for the final contours (at build-out) of the landfill.

Post-settlement slopes were calculated between the points shown on Sheet IIIE-B-1-12. The pre- and post-settlement elevations were determined from AutoCAD surfaces for the design condition and the post-settlement conditions from the SETTLE3 model. The post-settlement condition was generated as output from SETTLE3, which was used to develop the post-settlement surface (isopach) shown on Sheet IIIE-B-1-11. The pre and post-settlement point elevations are presented in Table 1A and 1B, and the strain and slope calculations are presented in Table 2.

B) Verify that strain induced on the bottom liner system components due to settlement is within acceptable limits.

Determine the post-settlement slope of the bottom liner and verify the strain induced on the geocomposite due to settlement is within acceptable limits.

Note that negative values indicate the components are in compression.

$$\text{Strain} = \frac{L_f - L_o}{L_o} x100$$

(Reference 2, Page 472)

L_f = Final distance between evaluation points after total settlement (ft)
L_o = Initial distance between evaluation points before total settlement (ft)

An example calculation of the estimated strain is shown below for Evaluation Points BL8 and BL10. The estimated strain for all evaluation points is shown in Table 2.

Evaluation Point BL13 to Evaluation Point BL17:

Initial Distance:

Evaluation Point BL13 Elev. =	516.0	ft-msl
Evaluation Point BL17 Elev. =	510.0	ft-msl
Plan View Distance=	299.8	ft
L_o=	299.8	ft

Total Settlement:		
Total Settlement Point BL13=	3.08	ft
Total Settlement Point BL17=	2.08	ft
Final Distance (after settlement):		
Evaluation Point BL13 Elev. =	512.9	ft-msl
Evaluation Point BL17 Elev. =	507.9	ft-msl
Plan View Distance=	299.8	ft
L_f=	299.8	ft

Strain= **-0.006%**

Conclusions:

- Compacted clay liner component of bottom liner (if used) has the smallest allowable tensile strain value which is 0.5 percent (Reference 2, page 469).
- The allowable tensile strain for geosynthetic clay liner (GCL) is 10 percent (ranges from 10 to 22 percent, Koerner et.al., 1996).
- The allowable tensile strain for an HDPE geomembrane is 6 to 8 percent (Reference 8).
- The allowable tensile strain for a drainage geocomposite (if used) is more than 20 percent for the geotextile (reference 3, page 112) and 200 percent for the geonet (reference 3, page 400).
- The maximum calculated tensile strain (0.303%) is acceptable, therefore the system will be stable. The maximium compressive strain is -0.049%.

TABLE 1A. BOTTOM LINER SYSTEM - SETTLEMENT SUMMARY

Evaluation Point[1]	Initial Top of Bottom Liner Elevation (ft-msl)	Post-Settlement Top of Bottom Liner Elevation (ft-msl)	Total Top of Bottom Liner Settlement (ft)
BL1	521.1	519.5	1.61
BL2	516.0	514.6	1.44
BL3	510.0	508.9	1.12
BL4	520.0	518.7	1.23
BL5	520.0	517.5	2.46
BL6	630.0	627.5	2.47
BL7	524.2	522.3	1.95
BL8	518.0	515.5	2.45
BL9	512.4	510.9	1.58
BL10	548.0	547.3	0.63
BL11	514.0	511.5	2.47
BL12	520.0	517.1	2.92
BL13	516.0	512.9	3.08
BL14	607.2	606.9	0.38
BL15	518.6	515.6	2.98
BL16	520.0	517.6	2.44
BL17	510.0	507.9	2.08
BL18	632.0	631.2	0.84
BL19	620.0	618.5	1.52
BL20	630.9	628.9	1.92
BL21	630.0	627.8	2.15
BL22	627.0	624.7	2.32
BL23	621.3	618.4	2.92
BL24	632.0	630.3	1.74
BL25	622.7	620.6	2.15
BL26	628.0	626.3	1.75
BL27	632.0	629.8	2.21
BL28	632.2	631.4	0.89
BL29	637.1	635.6	1.53
BL30	639.6	638.0	1.63
BL31	640.1	638.3	1.71

[1] Refer to Sheet IIIE-B-1-12 for Evaluation Point locations BL1 thru BL31. Initial Top of Bottom Liner Elevations shown on

TABLE 1B. DISTANCES BETWEEN SETTLEMENT EVALUATION POINTS

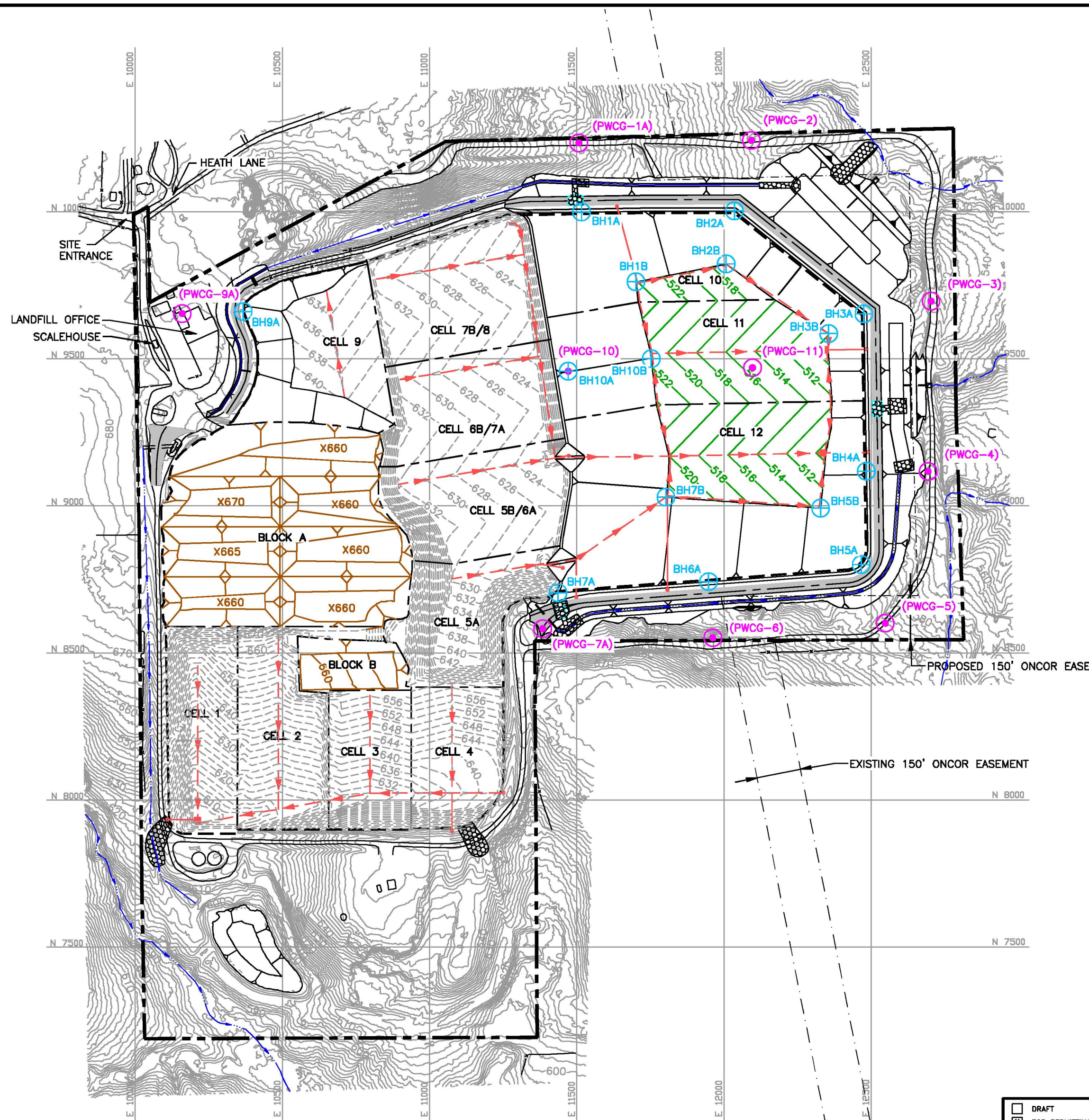
Evaluation Points[1]		Distance (ft)
From	To	
BL1	BL2	255.6
BL2	BL3	299.8
BL4	BL2	141.4
BL5	BL2	140.9
BL6	BL7	316.9
BL8	BL11	141.4
BL10	BL9	106.5
BL12	BL13	140.9
BL14	BL15	263.3
BL16	BL13	200.3
BL13	BL17	299.8
BL18	BL19	510.7
BL20	BL22	193.9
BL21	BL22	109.4
BL22	BL23	287.2
BL27	BL22	176.1
BL24	BL25	456.2
BL26	BL25	180.9
BL29	BL28	235.9
BL30	BL29	133.4
BL31	BL29	106.1

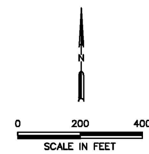
[1] Refer to Sheet IIIE-B-1-12 for Evaluation Points BL1 through BL31.

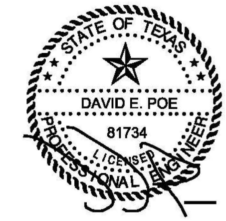
TABLE 2. BOTTOM LINER SYSTEM - SLOPE AND STRAIN SUMMARY

Evaluation Point[1]		Initial Top of Bottom Liner Elevation (ft-msl)		Post-Settlement Top of Bottom Liner Elevation (ft-msl)		Plan View Distance (ft)	L_o (ft)	L_f (ft)	Initial Slope (ft/ft)	Post-Settlement Slope (ft/ft)	Tensile Strain (%)
A	B	A	B	A	B						
BL1	BL2	521.1	516.0	519.5	514.6	255.6	255.6	255.6	0.020	0.019	-0.001
BL2	BL3	516.0	510.0	514.6	508.9	299.8	299.8	299.8	0.020	0.019	-0.002
BL4	BL2	520.0	516.0	518.7	514.6	141.4	141.5	141.5	0.028	0.029	0.004
BL5	BL2	520.0	516.0	517.5	514.6	140.9	140.9	140.9	0.028	0.021	-0.018
BL6	BL7	630.0	524.2	627.5	522.3	316.9	334.1	333.9	0.334	0.332	-0.049
BL8	BL11	518.0	514.0	515.5	511.5	141.4	141.5	141.5	0.028	0.028	0.000
BL10	BL9	548.0	512.4	547.3	510.9	106.5	112.3	112.6	0.334	0.343	0.271
BL12	BL13	520.0	516.0	517.1	512.9	140.9	140.9	140.9	0.028	0.030	0.003
BL14	BL15	607.2	518.6	606.9	515.6	263.3	277.8	278.7	0.337	0.347	0.303
BL16	BL13	520.0	516.0	517.6	512.9	200.3	200.3	200.3	0.020	0.023	0.007
BL13	BL17	516.0	510.0	512.9	507.9	299.8	299.8	299.8	0.020	0.017	-0.006
BL18	BL19	632.0	620.0	631.2	618.5	510.7	510.9	510.9	0.023	0.025	0.003
BL20	BL22	630.9	627.0	628.9	624.7	193.9	194.0	194.0	0.020	0.022	0.004
BL21	BL22	630.0	627.0	627.8	624.7	109.4	109.4	109.4	0.027	0.029	0.004
BL22	BL23	627.0	621.3	624.7	618.4	287.2	287.3	287.3	0.020	0.022	0.004
BL27	BL22	632.0	627.0	629.8	624.7	176.1	176.2	176.2	0.028	0.029	0.002
BL24	BL25	632.0	622.7	630.3	620.6	456.2	456.3	456.3	0.020	0.021	0.002
BL26	BL25	628.0	622.7	626.3	620.6	180.9	181.0	181.0	0.029	0.032	0.007
BL29	BL28	637.1	632.2	635.6	631.4	235.9	236.0	236.0	0.021	0.018	-0.005
BL30	BL29	639.6	637.1	638.0	635.6	133.4	133.4	133.4	0.019	0.018	-0.001
BL31	BL29	640.1	637.1	638.3	635.6	106.1	106.2	106.2	0.027	0.026	-0.004

[1] Refer to Sheet IIIE-B-1-12 for Evaluation Point locations. The "A" and "B" points represent the upgradient and downgradient endpoints, respectively.







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LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- CELL BOUNDARY
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 — SITE GRID
- 610 — EXISTING CONTOUR (SEE NOTE 1)
- 516 — TOP OF LINER CONTOUR
- 630 — AS-BUILT TOP OF LINER
- X660 — APPROXIMATE TOP OF PRE-SUBTITLE D LINER (SEE NOTE 8)
- CHANNEL CENTERLINE
- LEACHATE COLLECTION PIPE
- LEACHATE COLLECTION SUMP
- LEACHATE RISER PIPE
- BH1 — BOREHOLE DESIGNATION (FOR WCG ANALYSIS)
- (PWCG-1A) — BORING DESIGNATION (FOR DRILLING GEOLOGICAL INVESTIGATION)
- BH1A — TRANSLATED BORING PROFILE LOCATIONS (SEE NOTE 4)

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022. THE GRID SYSTEM IS BASED ON A SITE GRID SYSTEM. ELEVATIONS ARE BASED ON NAVD 88.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.
3. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY STANGER SURVEYING COMPANY, DATED APRIL 1995.
4. TRANSLATED BORING PROFILES DEVELOPED FROM ADJACENT BORING FOR USE IN SETTLE 3 MODEL.

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DESIGN BY: MB
REVIEWED BY: DEP

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS EXCAVATION PLAN W/ SETTLE3 TRANSLATED BORING LOCATIONS
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CHEROKEE COUNTY, TEXAS

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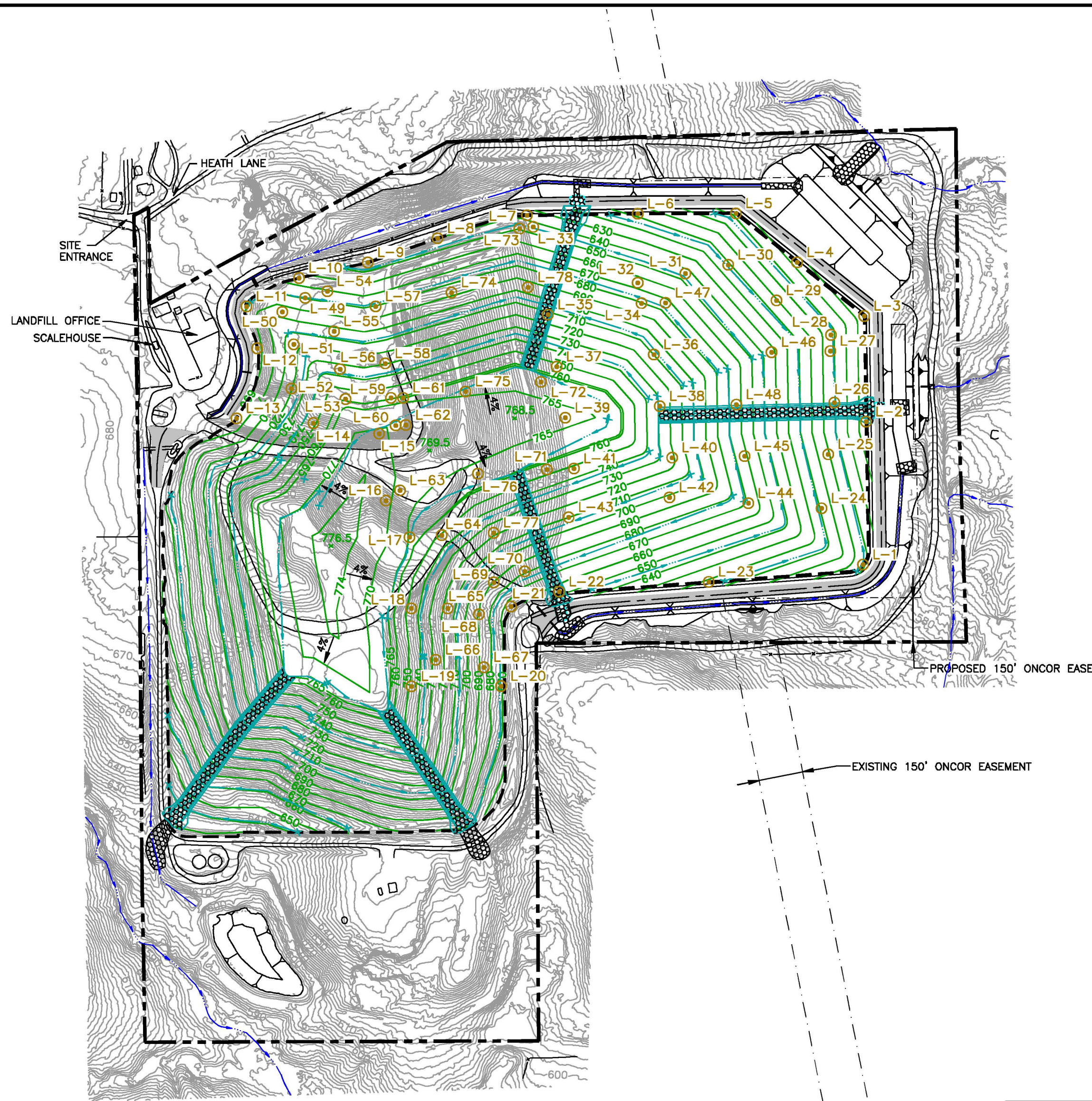

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SCALE IN FEET

LEGEND

PERMIT BOUNDARY
LIMIT OF WASTE
EXISTING EASEMENT
PROPOSED EASEMENT
E 10500 SITE GRID
610 EXISTING CONTOUR (SEE NOTE 1)
624 PROPOSED FINAL COVER CONTOUR
DRAINAGE SWALE
DRAINAGE LETDOWN
CHANNEL CENTERLINE
EXISTING PRE-SUBTITLE D FINAL COVER
L-1 LOAD ANALYSIS POINT

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS
FINAL COVER ANALYSIS POINT PLAN

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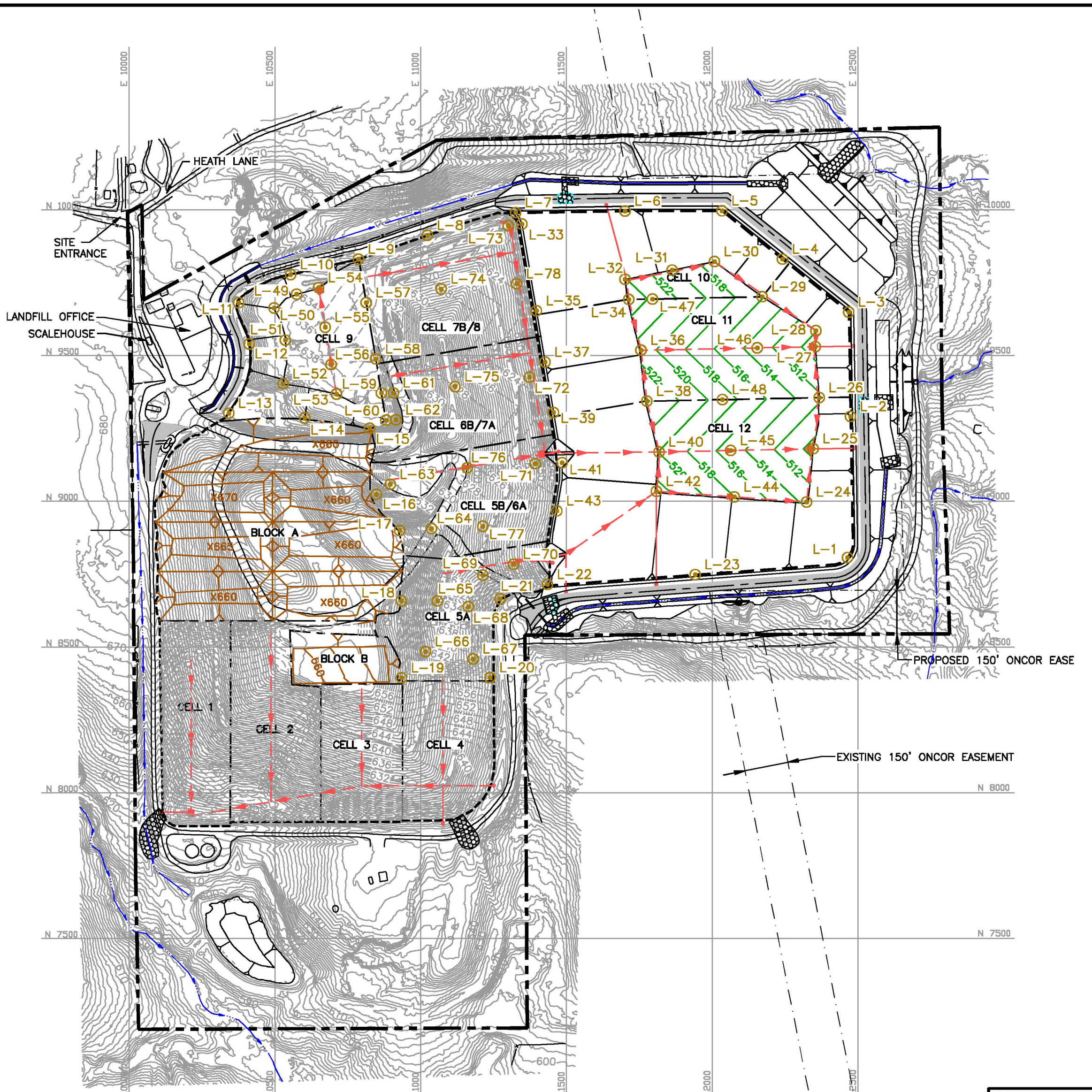
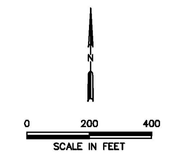


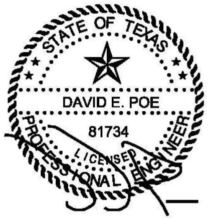


LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- CELL BOUNDARY
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 SITE GRID
- 610 EXISTING CONTOUR (SEE NOTE 1)
- 516 TOP OF LINER CONTOUR
- 630 AS-BUILT TOP OF LINER
- X660 APPROXIMATE TOP OF PRE-SUBTITLE D LINER (SEE NOTE 8)
- CHANNEL CENTERLINE
- LEACHATE COLLECTION PIPE
- LEACHATE COLLECTION SUMP
- LEACHATE RISER PIPE
- L-1 LOAD ANALYSIS POINT

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.



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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS
FINAL COVER ANALYSIS POINT PLAN
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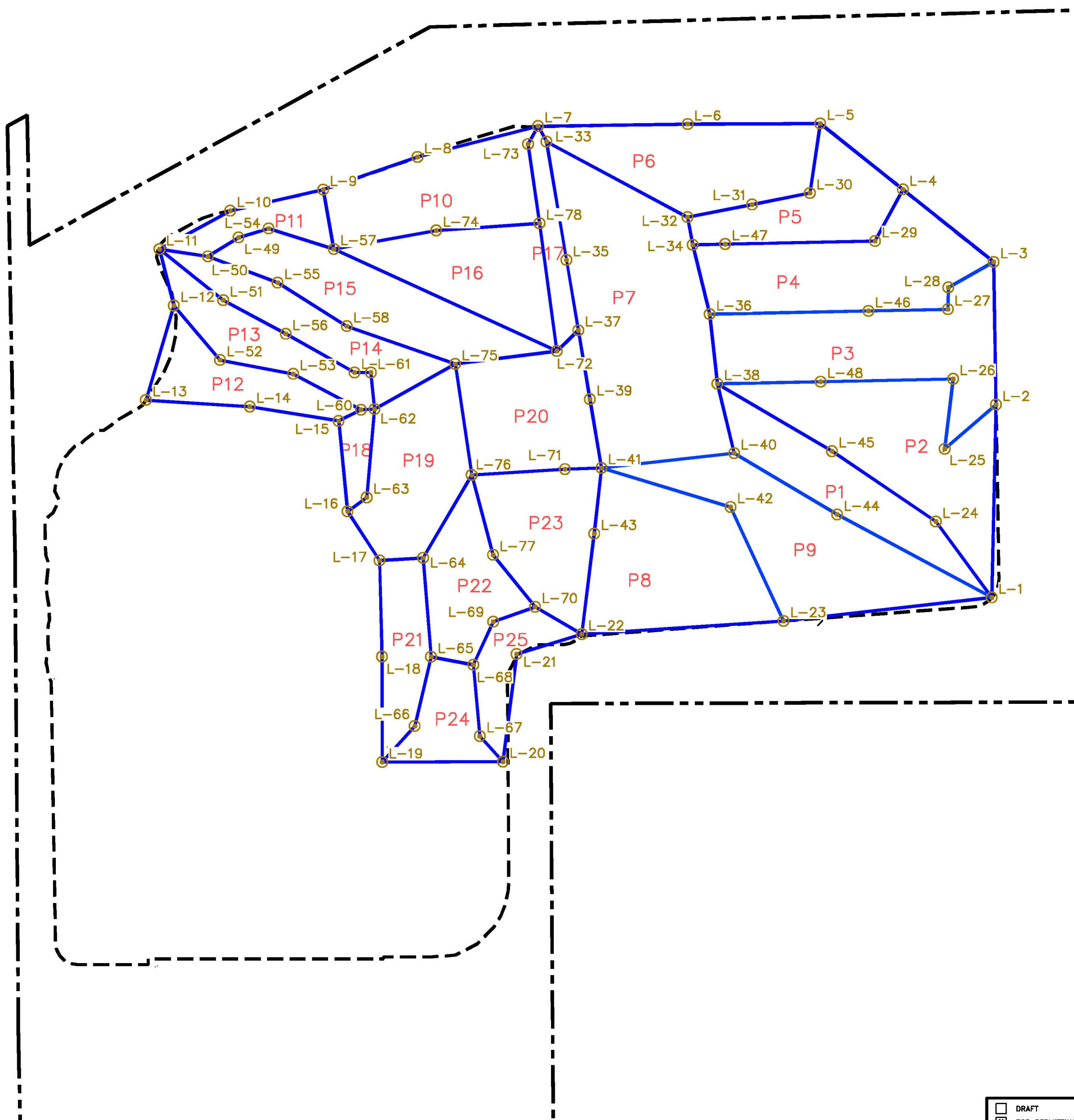

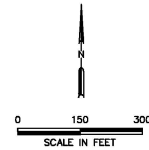

LEGEND

— - —	PERMIT BOUNDARY
- - -	LIMIT OF WASTE
⊙ L-1	LOAD ANALYSIS POINT
——	LOAD POLYGONS
P1	POLYGON NO.

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PINE HILL FARMS LANDFILL TX, LP

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS
SETTLEMENT ANALYSIS LOAD POLYGONS

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CHEROKEE COUNTY, TEXAS

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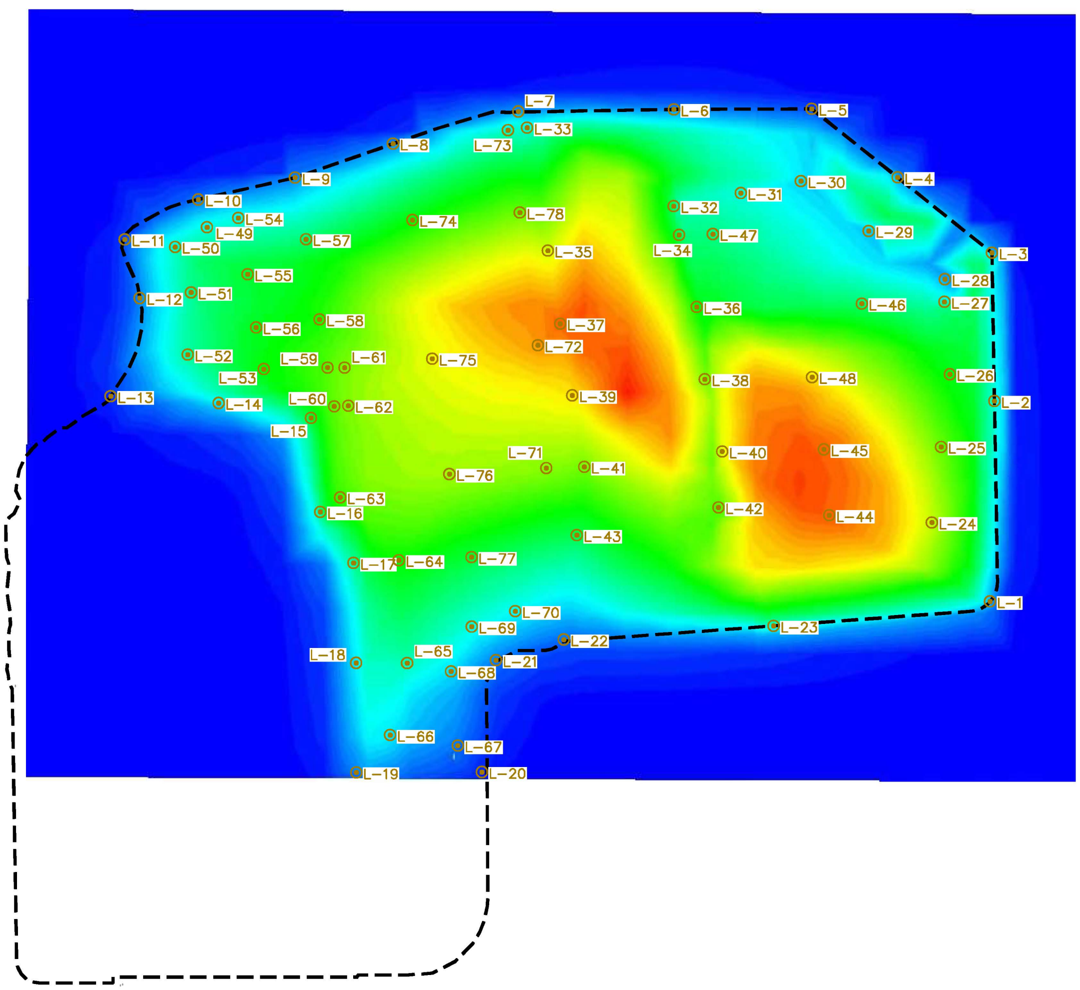

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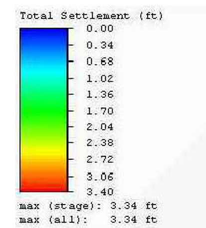
SCALE IN FEET

LEGEND

PERMIT BOUNDARY

LIMIT OF WASTE

L-1 LOAD ANALYSIS POINT



NOTES:

1. SETTLEMENT CONTOURS DEVELOPED USING SETTLE3 VERSION 5.011 RELEASED JUNE 30, 2021 (COPYRIGHT 2008).

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS
SETTLEMENT CONTOURS

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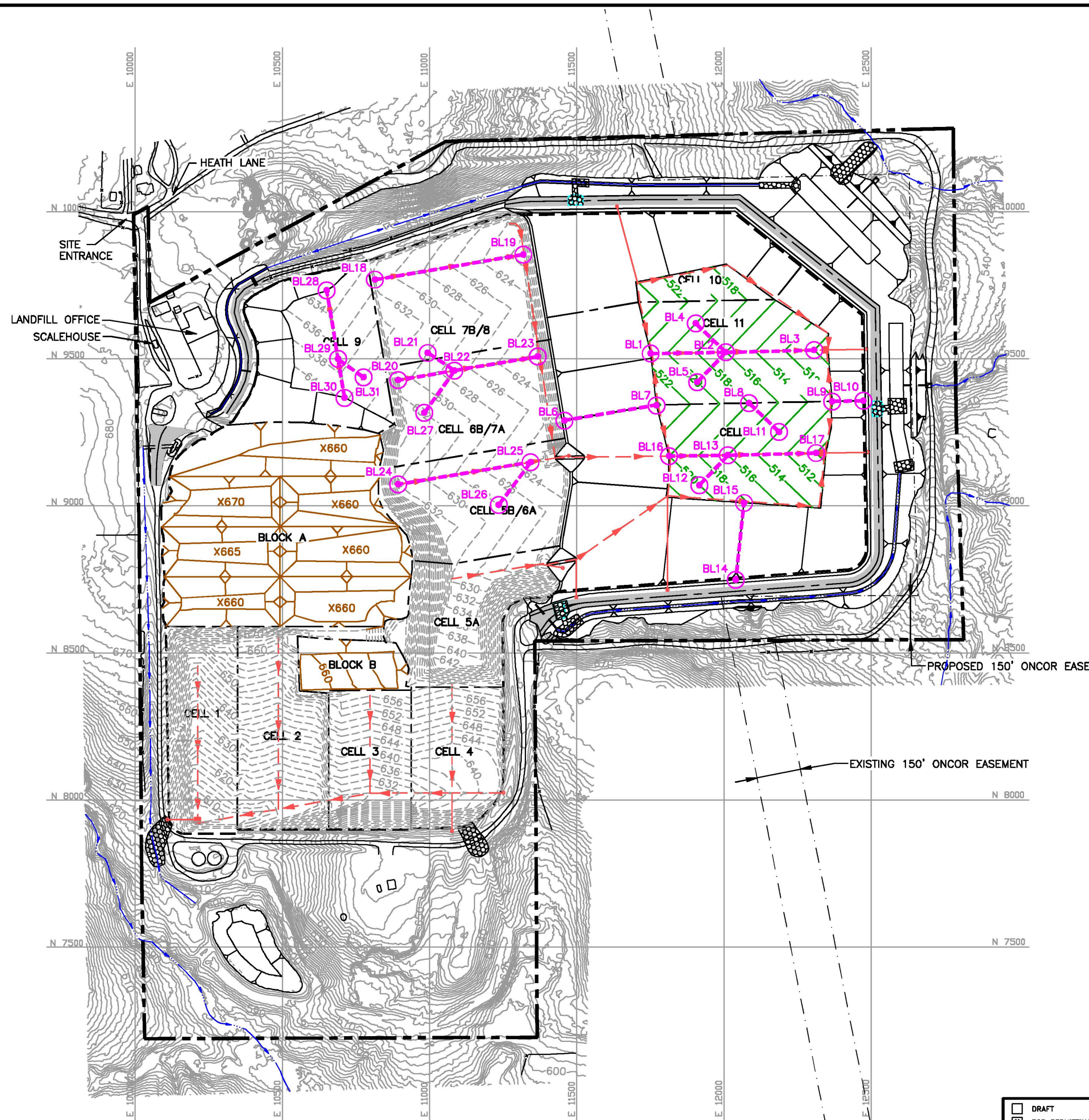

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- CELL BOUNDARY
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 — SITE GRID
- 610 — EXISTING CONTOUR (SEE NOTE 1)
- 516 — TOP OF LINER CONTOUR
- 630 — AS-BUILT TOP OF LINER
- X660 — APPROXIMATE TOP OF PRE-SUBTITLE D LINER (SEE NOTE 8)
- CHANNEL CENTERLINE
- LEACHATE COLLECTION PIPE
- LEACHATE COLLECTION SUMP
- LEACHATE RISER PIPE
- BL1 — BOTTOM LINER EVALUATION POINT
- BOTTOM LINER EVALUATION LINE

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022. THE GRID SYSTEM IS BASED ON A SITE GRID SYSTEM. ELEVATIONS ARE BASED ON NAVD 88.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.
3. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY STANGER SURVEYING COMPANY, DATED APRIL 1995.

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT ANALYSIS
BOTTOM LINER STRAIN CALCULATIONS

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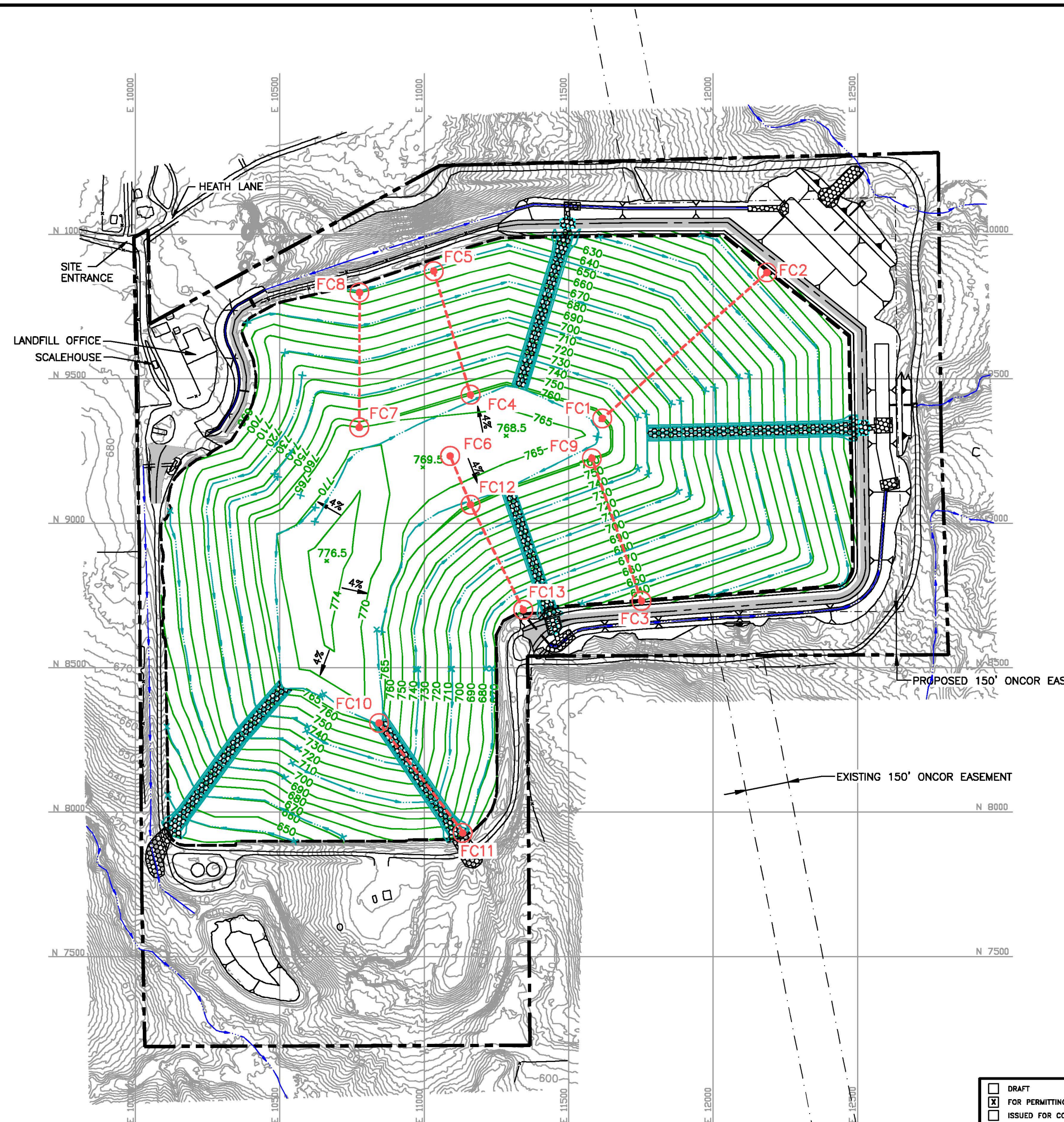

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 SITE GRID
- 610 EXISTING CONTOUR (SEE NOTE 1)
- 624 PROPOSED FINAL COVER CONTOUR
- DRAINAGE SWALE
- DRAINAGE LETDOWN
- CHANNEL CENTERLINE
- FC2 FINAL COVER EVALUATION POINT
- FINAL COVER EVALUATION LINE

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION IS 776.5 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 774.5 FT-MSL.
4. TYPICAL SIDESLOPES ARE 4H:1V. TYPICAL TOPSLOPE IS 4%.
5. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY LANDES & ASSOCIATES, INC. DATED MARCH 2011.

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REVISIONS

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MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT FINAL COVER ANALYSIS POINT PLAN

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Weaver Consultants Group

Royal Oaks Settlement
Weaver Consultants Group
Report Creation Date: 2024/01/02, 13:50:00

Table of Contents

Settle3 Analysis Information

Royal Oaks Settlement

Project Settings

Document Name	Royal Oaks Settlement Polygon Edits JBM.s3z
Project Title	Royal Oaks Settlement
Author	MB
Company	Weaver Consultants Group
Date Created	4/13/2023, 3:56:26 PM
Stress Computation Method	Boussinesq
Minimum settlement ratio for subgrade modulus	0.9
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 16.6242 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [ft]	0	3.20883
Total Consolidation Settlement [ft]	0	2.20097
Virgin Consolidation Settlement [ft]	0	2.20097
Recompression Consolidation Settlement [ft]	0	0
Immediate Settlement [ft]	0	1.95088
Loading Stress ZZ [ksf]	-3.23243e-05	14.9305
Loading Stress XX [ksf]	-3.32466	17.3896
Loading Stress YY [ksf]	-3.48696	18.2787
Total Stress ZZ [ksf]	-3.23243e-05	30.5067
Total Stress XX [ksf]	-2.84659	35.3795
Total Stress YY [ksf]	-2.66188	36.182
Modulus of Subgrade Reaction (Total) [ksf/ft]	-0.000148629	9.45165
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-0.000551454	28.2097
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000452518	267.086
Total Strain	-1.59382e-07	0.301528
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.0006	30.4993
Over-consolidation Ratio	1	1.03293
Void Ratio	0	0.7
Hydroconsolidation Settlement [ft]	0	0
Undrained Shear Strength	-6.87287e-07	0.401451

Loads

1. Fill Load: "LP-1"

Label	LP-1
Load Type	Flexible
Area of Load	83106.7 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
12464.9	8806.77	0.6594
12324.3	8997.4	5.691
12063.9	9173.97	9.6978
11776.2	9343.07	14.931
11818.1	9168.9	14.742
12076.6	9014.85	9.8658

2. Fill Load: "LP-2"

Label	LP-2
Load Type	Flexible
Area of Load	147715 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
12475.3	9290.54	0.6594
12346.7	9179.8	4.8132
12367.7	9355.27	3.9732
12035.3	9348.42	9.4248
11776.2	9343.07	14.931
12063.9	9173.97	9.6978
12324.3	8997.4	5.691
12464.9	8806.77	0.6594

3. Fill Load: "LP-3"

Label	LP-3
Load Type	Flexible
Area of Load	147139 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
12469.1	9648.86	0.6594
12356	9584.83	4.2546
12354.6	9530.04	4.3386
12155.2	9525.92	7.518
11756.6	9517.7	13.4442
11776.2	9343.07	14.931
12035.3	9348.42	9.4248
12367.7	9355.27	3.9732
12346.7	9179.8	4.8132
12475.3	9290.54	0.6594

4. Fill Load: "LP-4"

Label	LP-4
Load Type	Flexible
Area of Load	131043 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
12241.6	9830.57	0.6594
12172	9701.31	5.0484
11795.8	9693.55	9.87
11714.1	9691.87	10.4328
11756.6	9517.7	13.4442
12155.2	9525.92	7.518
12354.6	9530.04	4.3386
12356	9584.83	4.2546
12469.1	9648.86	0.6594

5. Fill Load: "LP-5"

Label	LP-5
Load Type	Flexible
Area of Load	74337.7 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
12034.4	9996.14	0.6594
12008.3	9821.79	5.6826
11863.2	9792.91	7.7532
11701.3	9761.04	9.1896
11714.1	9691.87	10.4328
11795.8	9693.55	9.87
12172	9701.31	5.0484
12241.6	9830.57	0.6594

6. Fill Load: "LP-6"

Label	LP-6
Load Type	Flexible
Area of Load	113773 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11326.3	9989.97	0.6594
11347.4	9950.69	1.1256
11701.3	9761.04	9.1896
11863.2	9792.91	7.7532
12008.3	9821.79	5.6826
12034.4	9996.14	0.6594
11701.5	9995	0.6594

7. Fill Load: "LP-7"

Label	LP-7
Load Type	Flexible
Area of Load	232406 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11347.4	9950.69	1.1256
11397.4	9653.57	4.8384
11427.1	9477.27	7.686
11456.2	9304.23	8.7612
11485.3	9131.68	7.3878
11818.1	9168.9	14.742
11776.2	9343.07	14.931
11756.6	9517.7	13.4442
11714.1	9691.87	10.4328
11701.3	9761.04	9.1896

8. Fill Load: "LP-8"

Label	LP-8
Load Type	Flexible
Area of Load	144248 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11435.9	8714.5	0.6594
11942.1	8747.73	0.6594
11809.1	9033.67	12.1464
11485.3	9131.68	7.3878
11467.3	8966.87	4.7208

9. Fill Load: "LP-9"

Label	LP-9
Load Type	Flexible
Area of Load	143341 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11942.1	8747.73	0.6594
12464.9	8806.77	0.6594
12076.6	9014.85	9.8658
11818.1	9168.9	14.742
11485.3	9131.68	7.3878
11809.1	9033.67	12.1464

10. Fill Load: "LP-10"

Label	LP-10
Load Type	Flexible
Area of Load	101800 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11326.3	9989.97	0.6594
11023.6	9912.15	0.6594
10787.1	9830.26	0.6594
10813.5	9681.09	2.5578
11070.7	9727.67	3.4566
11329.8	9745.69	4.2756
11301	9944.71	1.7304

11. Fill Load: "LP-11"

Label	LP-11
Load Type	Flexible
Area of Load	32566.7 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10787.1	9830.26	0.6594
10553.9	9777.19	0.6594
10376.1	9680.69	0.6594
10497.6	9662.57	3.3432
10574.7	9710.19	2.5074
10650	9732.7	2.2554
10813.5	9681.09	2.5578

12. Fill Load: "LP-12"

Label	LP-12
Load Type	Flexible
Area of Load	58453.9 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10412.3	9539.34	0.6594
10342.4	9302.15	0.6594
10602.7	9285.73	4.2714
10825.9	9249.96	6.7326
10881.5	9277.79	7.6062
10711.5	9367.6	7.3668
10528	9402.52	4.5444

13. Fill Load: "LP-13"

Label	LP-13
Load Type	Flexible
Area of Load	57860.9 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10376.1	9680.69	0.6594
10412.3	9539.34	0.6594
10528	9402.52	4.5444
10711.5	9367.6	7.3668
10881.5	9277.79	7.6062
10915.5	9279.27	8.232
10906.6	9371.19	8.2152
10865.7	9371.49	7.182
10693	9468.42	5.88
10536.2	9552.81	4.5654

14. Fill Load: "LP-14"

Label	LP-14
Load Type	Flexible
Area of Load	68181.1 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10376.1	9680.69	0.6594
10536.2	9552.81	4.5654
10693	9468.42	5.88
10865.7	9371.49	7.182
10906.6	9371.19	8.2152
10915.5	9279.27	8.232
11118.3	9392.89	8.694
10846.1	9487.62	5.2794
10672.9	9596.86	3.9606
10497.6	9662.57	3.3432

15. Fill Load: "LP-15"

Label	LP-15
Load Type	Flexible
Area of Load	104065 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10497.6	9662.57	3.3432
10672.9	9596.86	3.9606
10846.1	9487.62	5.2794
11118.3	9392.89	8.694
11373.4	9425.37	9.1728
10813.5	9681.09	2.5578
10650	9732.7	2.2554
10574.7	9710.19	2.5074

16. Fill Load: "LP-16"

Label	LP-16
Load Type	Flexible
Area of Load	87806.2 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10813.5	9681.09	2.5578
11373.4	9425.37	9.1728
11329.8	9745.69	4.2756
11070.7	9727.67	3.4566

17. Fill Load: "LP-17"

Label	LP-17
Load Type	Flexible
Area of Load	27755.7 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11326.3	9989.97	0.6594
11301	9944.71	1.7304
11329.8	9745.69	4.2756
11373.4	9425.37	9.1728
11427.1	9477.27	7.686
11397.4	9653.57	4.8384
11347.4	9950.69	1.1256

18. Fill Load: "LP-18"

Label	LP-18
Load Type	Flexible
Area of Load	15877.7 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10848.3	9022.97	7.5096
10896	9057.91	8.7612
10915.5	9279.27	8.232
10881.5	9277.79	7.6062
10825.9	9249.96	6.7326

19. Fill Load: "LP-19"

Label	LP-19
Load Type	Flexible
Area of Load	96401.9 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10928.4	8899.59	5.7666
11037.7	8906.02	7.1148
11159.9	9114.31	8.8326
11118.3	9392.89	8.694
10915.5	9279.27	8.232
10896	9057.91	8.7612
10848.3	9022.97	7.5096

20. Fill Load: "LP-25"

Label	LP-25
Load Type	Flexible
Area of Load	37995.6 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11238	8394.48	0.9492
11272.4	8664.79	0.6594
11435.9	8714.5	0.6594
11318.9	8782.99	3.2046
11213.5	8745.91	3.57
11163.6	8637.92	3.2004
11180	8458.54	2.5788

21. Fill Load: "LP-20"

Label	LP-20
Load Type	Flexible
Area of Load	95435.1 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11159.9	9114.31	8.8326
11393.5	9128.74	8.5218
11485.3	9131.68	7.3878
11456.2	9304.23	8.7612
11427.1	9477.27	7.686
11373.4	9425.37	9.1728
11118.3	9392.89	8.694

22. Fill Load: "LP-21"

Label	LP-21
Load Type	Flexible
Area of Load	50810.2 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10935	8393.67	5.04
11016.7	8484	5.0694
11057.9	8657.9	4.83
11037.7	8906.02	7.1148
10928.4	8899.59	5.7666

23. Fill Load: "LP-22"

Label	LP-22
Load Type	Flexible
Area of Load	67808.3 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11057.9	8657.9	4.83
11163.6	8637.92	3.2004
11213.5	8745.91	3.57
11318.9	8782.99	3.2046
11213.3	8913.73	5.5818
11159.9	9114.31	8.8326
11037.7	8906.02	7.1148

24. Fill Load: "LP-23"

Label	LP-23
Load Type	Flexible
Area of Load	89106.6 ft2
Elevation	520 ft
Installation Stage	Stage 1

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
11435.9	8714.5	0.6594
11467.3	8966.87	4.7208
11485.3	9131.68	7.3878
11393.5	9128.74	8.5218
11159.9	9114.31	8.8326
11213.3	8913.73	5.5818
11318.9	8782.99	3.2046

25. Fill Load: "LP-24"

Label	LP-24
Load Type	Flexible
Area of Load	42182.3 ft2
Elevation	520 ft
Installation Stage	Stage 1

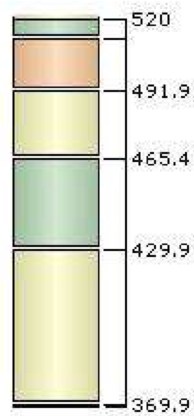
Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
10935	8393.67	2.88
11238	8394.48	0.5424
11180	8458.54	1.4736
11163.6	8637.92	1.8288
11057.9	8657.9	2.76
11016.7	8484	2.8968

Soil Layers

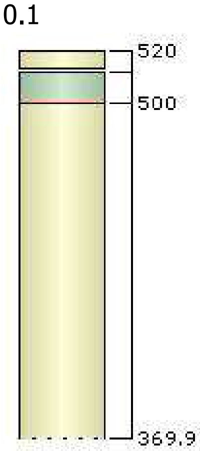
BH-1A

XY Location:		BH-1A: (11516.8, 9998.3)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	7.5	519.9
3	Silt/Silty Sand	20.5	512.4
4	Sand	26.5	491.9
5	Clay	35.5	465.4
6	Sand	60	429.9



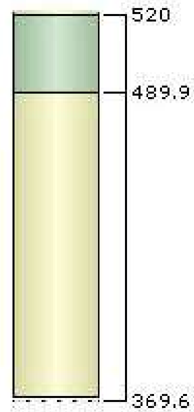
BH-2A

XY Location:		BH-2A: (12036.6, 10002.5)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	8	520
2	Clay	12	512
3	Silt/Silty Sand	0.1	500
4	Sand	130	499.9
5	Clay	0.1	369.9
6	Sand	0.1	369.8



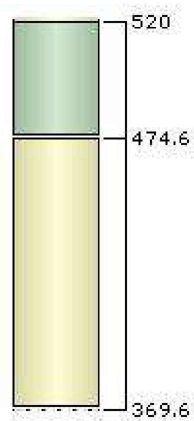
BH-3A

XY Location:		BH-3A: (12475.4, 9651.95)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	30	519.9
3	Silt/Silty Sand	0.1	489.9
4	Sand	0.1	489.8
5	Clay	0.1	489.7
6	Sand	120	489.6



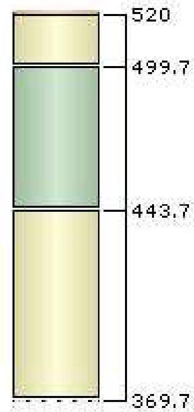
BH-4A

XY Location:		BH-4A: (12484.6, 9116.99)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	0.1	519.7
5	Clay	45	519.6
6	Sand	105	474.6



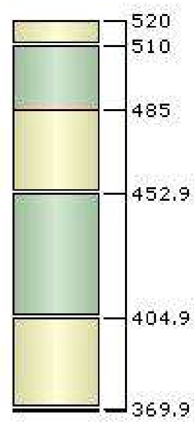
BH-5A

XY Location:		BH-5A: (12467.2, 8800.02)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	20	519.7
5	Clay	56	499.7
6	Sand	74	443.7



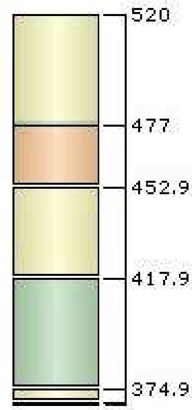
BH-6A

XY Location:		BH-6A: (11948.4, 8741.84)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	10	520
2	Clay	25	510
3	Silt/Silty Sand	0.1	485
4	Sand	32	484.9
5	Clay	48	452.9
6	Sand	35	404.9



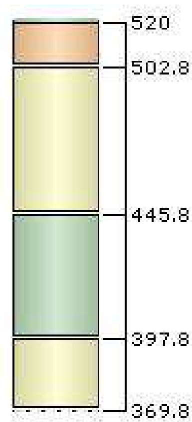
BH-7A

XY Location:		BH-7A: (11436.7, 8703.86)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	43	520
2	Clay	0.1	477
3	Silt/Silty Sand	24	476.9
4	Sand	35	452.9
5	Clay	43	417.9
6	Sand	5	374.9



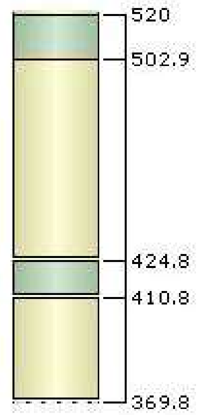
BH-9A

XY Location:		BH-9A: (10367.4, 9659.67)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	17	519.8
4	Sand	57	502.8
5	Clay	48	445.8
6	Sand	28	397.8



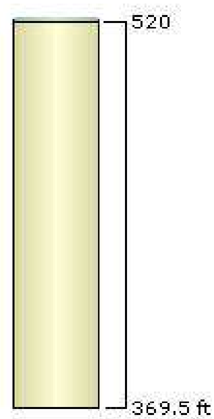
BH-10A

XY Location:		BH-10A: (11472, 9456.67)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	17	519.9
3	Silt/Silty Sand	0.1	502.9
4	Sand	78	502.8
5	Clay	14	424.8
6	Sand	41	410.8



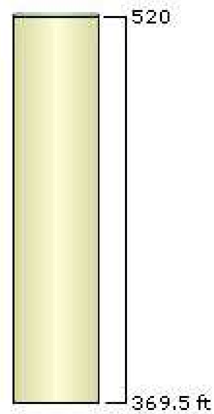
BH-1B

XY Location:		BH-1B: (11701.3, 9761.04)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	0.1	519.7
5	Clay	0.1	519.6
6	Sand	150	519.5



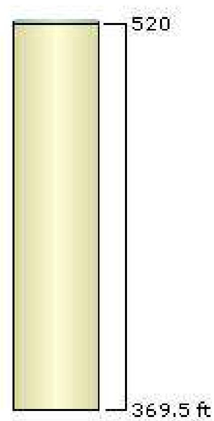
BH-2B

XY Location:		BH-2B: (12008.3, 9821.79)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	0.1	519.7
5	Clay	0.1	519.6
6	Sand	150	519.5



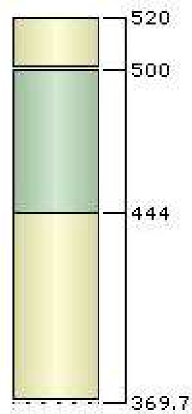
BH-3B

XY Location:		BH-3B: (12356.4, 9585.02)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	0.1	519.7
5	Clay	0.1	519.6
6	Sand	150	519.5



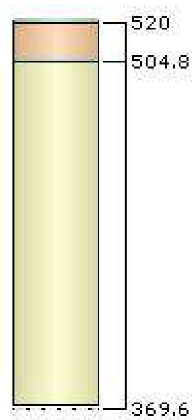
BH-5B

XY Location:		BH-5B: (12328.8, 8992.07)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	20	520
2	Clay	56	500
3	Silt/Silty Sand	0.1	444
4	Sand	0.1	443.9
5	Clay	0.1	443.8
6	Sand	74	443.7



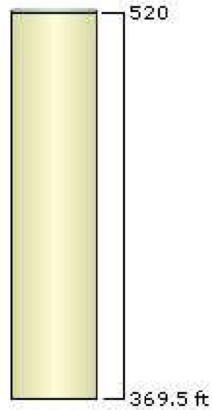
BH-7B

XY Location:		BH-7B: (11803.3, 9029.07)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	15	519.8
4	Sand	0.1	504.8
5	Clay	0.1	504.7
6	Sand	135	504.6



BH-10B

XY Location:		BH-10B: (11753.4, 9500)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Sand	0.1	520
2	Clay	0.1	519.9
3	Silt/Silty Sand	0.1	519.8
4	Sand	0.1	519.7
5	Clay	0.1	519.6
6	Sand	150	519.5



Soil Properties

Property	Sand	Clay	Silt/Silty Sand
Color			
Unit Weight [kips/ft3]	0.12	0.1	0.11
K0	1	1	1
Immediate Settlement	Enabled	Disabled	Enabled
Es [ksf]	1200	-	350
Esur [ksf]	120	-	110
Primary Consolidation	Disabled	Enabled	Disabled
Material Type		Non-Linear	
Cc	-	0.17	-
Cr	-	0.01	-
e0	-	0.7	-
OCR	-	1	-
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	BL1	11751.5, 9517.6	Auto: 61
2	BL2	12007, 9521.83	Auto: 61
3	BL3	12306.7, 9529.05	Auto: 61
4	BL4	11905, 9619.75	Auto: 61
5	BL5	11909.5, 9420.2	Auto: 61
6	BL6	11458.8, 9289.33	Auto: 65
7	BL7	11771.6, 9340.91	Auto: 61
8	BL8	12085.6, 9348.42	Auto: 65
9	BL9	12368, 9354.24	Auto: 65
10	BL10	12474.4, 9356.44	Auto: 49
11	BL11	12187.6, 9250.5	Auto: 65
12	BL12	11916.9, 9069.24	Auto: 61
13	BL13	12014.2, 9171.91	Auto: 65
14	BL14	12040.2, 8747.61	Auto: 45
15	BL15	12068.6, 9009.38	Auto: 65
16	BL16	11814, 9168.81	Auto: 61
17	BL17	12313.9, 9179.12	Auto: 73
18	BL18	10815.1, 9768.5	Auto: 77
19	BL19	11318.9, 9853.32	Auto: 77
20	BL20	10893.8, 9427	Auto: 65
21	BL21	10994.1, 9519.37	Auto: 65
22	BL22	11085, 9458.99	Auto: 65
23	BL23	11368.2, 9507.3	Auto: 65
24	BL24	10893.6, 9072.88	Auto: 69
25	BL25	11343.7, 9147.64	Auto: 69
26	BL26	11235.3, 9002.86	Auto: 69
27	BL27	10982.4, 9315.88	Auto: 65
28	BL28	10650.9, 9731.98	Auto: 77
29	BL29	10689.5, 9498.25	Auto: 69
30	BL30	10711.5, 9366.69	Auto: 69
31	BL31	10775.9, 9436.67	Auto: 69

Field Point Grid

Number of points	825
Expansion Factor	1

Grid Coordinates

X [ft]	Y [ft]
12954.7	10472.6
12954.7	7923.52
9872.22	7923.52
9872.22	10472.6

APPENDIX IIIE-B-2

FINAL COVER SETTLEMENT ANALYSIS

Includes pages IIIE-B-2-1 through IIIE-B-2-12

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

Required: Determine the post-settlement slope of the final cover system and verify that the strain induced on the final cover due to settlement is within acceptable limits.

Method:

A. Estimate primary settlement of waste below the final cover system.
B. Estimate secondary settlement of waste below the final cover system.
C. Estimate total settlement of waste below the final cover system.
D. Verify that strain induced on the final cover due to settlement is within acceptable limits.

Description of Contents:

- Sheets IIIE-B-2-3 thru IIIE-B-2-8 present example calculations.
- Table 1 presents the final cover settlement point parameters. and analysis results.
- Table 2 presents the strain calculations along the evaluation lines.
- Sheet IIIE-B-2-9 presents the analysis conclusions.
- Sheet IIIE-B-2-12 provides the final cover analysis points and evaluation lines supporting the strain calculations.

References:

1. Sowers, George F., Settlement of Solid Waste, *Proceedings of the Eighth International Conference on Soil Mechanics and Foundations Engineering, 1973* .
2. Quian, Xuede, R.M. Koerner, D. H. Gray, Geotechnical Aspects of Landfill Design and Construction, Prentice-Hall, Inc., New Jersey, 2002.
3. Koerner, Robert M., Designing with Geosynthetics, Third Edition. Prentice-Hall, New Jersey, 1994.
4. Acar, Yalcin B. & Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.
5. Zornberg, Jorge G., et al., *Retention of Free Liquids in Landfills Undergoing* Vertical Expansion, Journal of Geotechnical and Geoenvironmental Engineering, July 1999.
6. Fassett, Jeffrey B., et al., Geotechnical Properties of Municipal Solid Wastes and Their Use in Landfill Design, Waste Tech, 1994.
7. SETTLE3, Version 5.009, Copyright 2008-2021, Rockscience Inc.
8. Beggs, Ian D. et al, Assessment of Maximum Allowable Strains in Polyethylene and Polypropylene Geomembranes, Geo-Frontiers Congress, Austin, TX, 2005.

Solution:

A) Estimate primary settlement of waste below the final cover system.

MSW will undergo primary consolidation due to its own weight, final cover, equipment, etc. Primary consolidation occurs quickly, generally within the first month after loading. Therefore, the weight of the final cover system is the only remaining factor that contributes to primary consolidation. In addition, by the time the construction of the final cover is complete, settlement of the waste due to the weight of the final cover will be complete.

Primary settlement is calculated using the following equation:

$$S_p = \frac{H_o C_c}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_o}\right)$$

S_p = primary settlement, ft
H_o = waste thickness below the final cover system, ft
C_c = compression index
e_o = void ratio of the waste layer below final cover before settlement (i.e., before final cover placement)
$\Delta\sigma$ = change in loading/increase in overburden pressure, psf
σ'_o = overburden pressure acting at mid-height of refuse below the final cover, psf

For this site assume: C_c = 0.35 x e_o (Ref. 1, p. 210)

The compression index is a function of the void ratio. The compression index can range from C_c=0.15e_o to C_c=0.55e_o for fills that are low and high in organic content, respectively. An average compression index value was chosen because it is consistent with the types of waste accepted in the past. It is also representative of the minimal amount of settlement the site has experienced.

The average void ratio of waste below the final cover is estimated by determining the void ratio at the midpoint of the waste column below the final cover system. The void ratio is calculated for each settlement evaluation point using the following equation.

$e_o = 1.86 - 0.00102\ \sigma'_o$ (Ref. 5, p. 590)

where: σ'_o = overburden pressure in kPa

$\sigma'_o = 0.5\ \gamma_{msw}\ H_o$

$\Delta\sigma = \gamma_{cov}\ T_c$

γ_{msw} = unit weight of waste below the final cover system, pcf

γ_{cov} = unit weight of cover, pcf

T_c = thickness of final cover system, ft

Parameters:

γ_{cov} = 120 pcf

T_c = 3.5 feet (See Note 1, below)

γ_{msw} = varies (see Note 2, below)

Notes:
1. Tc value includes protective and final cover soils, intermediate cover, and grading soils
2. The value γ_{msw} is selected based on the midpoint of the waste thicknesses below the final cover system using the Unit Weight Profile for Waste/Daily Cover within an MSW Landfill chart developed from Ref. 4.

Example Calculations:

A) Estimate primary settlement of waste below the final cover system.

The settlement points analyzed are shown on Sheet IIIE-B-2-12. An example calculation of the estimated primary settlement is shown below for Evaluation Points FC12 and FC13. The estimated primary settlement for all evaluation points is shown in Table 1.

At Evaluation Point FC12:

Top of Final Cover Elevation (ft-msl)= 761.7

Bottom of Waste Elevation (ft-msl)= 627.5

H_o = 130.7 ft

γ_{msw} = 61.0 pcf

$\sigma'_o = 0.5\ \gamma_{msw}\ H_o$

σ'_o = 3986.4 psf

σ'_o = 190.9 kPa

$e_o = 1.86 - 0.00102\ \sigma'_o$

e_o = 1.67

$C_c = 0.35\ e_o$
$C_c =$ 0.58

$\Delta\sigma =$ 420.0 psf

$$S_p = \frac{130.7 \times 0.58}{1 + 1.67} \log\left(\frac{3986.4 + 420}{3986.4}\right)$$

$S_p =$ 1.2 ft

At Evaluation Point FC13:

Top of Final Cover Elevation (ft-msl)= 660.0
Bottom of Waste Elevation (ft-msl)= 651.4
$H_o =$ 5.1 ft
$\gamma_{msw} =$ 43.0 pcf

$\sigma'_o = 0.5\ \gamma_{msw}\ H_o$
$\sigma'_o =$ 109.7 psf
$\sigma'_o =$ 5.3 kPa

$e_o = 1.86 - 0.00102\ \sigma'_o$
$e_o =$ 1.85

$C_c = 0.35\ e_o$
$C_c =$ 0.65

$\Delta\sigma =$ 420.0 psf

$$S_p = \frac{5.1 \times 0.65}{1 + 1.85} \log\left(\frac{109.7 + 420}{109.7}\right)$$

$S_p =$ 0.8 ft

B) Estimate secondary settlement of waste below the final cover system.

Secondary consolidation continues at substantial rates for periods of time well beyond primary settlement. It is a combination of mechanical secondary compression, physico-chemical reaction, and bio-chemical decay. The settlement-log time relationship is similar to secondary compression of soils and can be expressed by:

$$S_c = \frac{H'_o\ \alpha}{1 + e'_o} \log\left(t_2/t_1\right)$$

(Ref. 2, p. 451)

Parameters:

S_c = secondary settlement, ft
α = secondary compression index
e'_o = void ratio of the waste layer below the final cover after primary settlement has occurred due to the final cover
H'_o = waste thickness below the final cover system after settlement, ft
t_1 = starting time of secondary settlement in years
t_2 = time at which settlement is determined in years

For this site assume: α = 0.03 x e'_o (Ref. 1, p. 210)

As reported by Sowers (Ref. 1), the secondary compression index is used to estimate waste decomposition. The secondary compression index ranges from $\alpha = 0.03e'_o$ to $\alpha = 0.09e'_o$ for conditions that are unfavorable and favorable to decay, respectively. An average secondary compression index value was chosen because it is consistent with the types of waste accepted in the past. It is also representative of the minimal amount of settlement the site has experienced.

The void ratio of the waste below the final cover at closure is a function of the overburden pressure caused by placement of the final cover system. The void ratio is calculated for each settlement evaluation point using the following equation.

$$e'_o = 1.86 - 0.00102\ \sigma''_o$$ (Ref. 5, p. 590)

where: σ''_o = overburden pressure in kPa

$\sigma''_o = 0.5\ \gamma'_{msw}\ H'_o$
γ'_{msw} = unit weight of waste below the final cover after primary settlement has occurred, pcf

For this site, the void ratio after primary settlement for the waste/cover soils below the final cover system varies between 1.5 to 1.9. Therefore, the secondary compression index will range between 0.09 to 0.11. Most literature sources report the secondary compression index in terms of the "modified secondary compression index" (Refs. 2, 6). The modified secondary compression index is defined by the following equation:

$$C'_\alpha = \frac{\alpha}{1 + e'_o}$$

The secondary compression index calculated for this site translates to a modified secondary compression index of 0.03 to 0.04 (for a void ratio of 1.5 to 1.9). These values are consistent with reported values for the modified secondary compression index which vary from 0.03 to 0.1 (Refs. 2, 6).

Time frame used for this analysis:

t_1 = 0.083 years
t_2 = 30.0 years (postclosure period)

An example calculation of the estimated secondary settlement using the above secondary settlement period is shown below for Evaluation Points FC12 and FC13. The estimated secondary settlement for all evaluation points is shown in Table 1.

At Evaluation Point FC12:

$H'_o = H_o - S_p$
$H_o' =$ 129.5 ft

$\sigma''_o = 0.5\ \gamma'_{msw}\ H'_o$
$\gamma'_{msw} =$ 61.0 pcf
$\sigma''_o =$ 3949.8 psf
$\sigma''_o =$ 189.1 kPa

$e'_o = 1.86 - 0.00102\ \sigma''_o$
$e'_o =$ 1.67

$\alpha = 0.03\ e'_o$
$\alpha =$ 0.05

$$S_c = \frac{H'_o\ \alpha}{1 + e'_o} \log(t_2/t_1)$$

$$S_c = \frac{129.5 \times 0.05}{1 + 1.67} \log\left(\frac{30}{0.083}\right)$$

$S_c =$ 6.2 ft

At Evaluation Point FC13:

$H'_o = H_o - S_p$

$H_o' =$ 4.3 ft

$\sigma''_o = 0.5\ \gamma'_{msw}\ H'_o$

$\gamma'_{msw} =$ 42.0 pcf

$\sigma''_o =$ 90.3 psf

$\sigma''_o =$ 4.3 kPa

$e'_o = 1.86 - 0.00102\ \sigma''_o$

$e'_o =$ 1.86

$\alpha = 0.03\ e'_o$

$\alpha =$ 0.06

$$S_c = \frac{H'_o\ \alpha}{1 + e'_o} \log (t_2/t_1)$$

$$S_c = \frac{4.3 \times 0.06}{1 + 4.3} \log\left(\frac{30}{0.083}\right)$$

$S_c =$ 0.2 ft

C) Estimate total settlement of waste below the final cover system.

Total settlement is the combination of primary and secondary settlement. An example calculation of the estimated total settlement is shown below for Evaluation Points FC12 and FC13. The estimated total settlement for all evaluation points is shown in Table 1.

At Evaluation Point FC12:				
Thickness of waste column, ft =	130.7	Primary Settlement =	1.2	ft
		Secondary Settlement =	6.2	ft
		Total Settlement =	7.4	ft
At Evaluation Point FC13:				
Thickness of waste column, ft=	5.1	Primary Settlement =	0.8	ft
		Secondary Settlement =	0.2	ft
		Total Settlement =	1.0	ft

D) Verify that strain induced on the final cover due to settlement is within acceptable limits.

Determine the post-settlement slope of the final cover system and verify the strain induced on the geocomposite due to settlement is within acceptable limits.

Note that negative values indicate the components are in compression.

$$\text{Strain} = \frac{L_f - L_o}{L_o} x100$$

(Reference 2, Page 472)

L_f = Final distance between evaluation points after total settlement (ft)
L_o = Initial distance between evaluation points before total settlement (ft)

An example calculation of the estimated strain is shown below for Evaluation Points FC12 and FC13. The estimated strain for all evaluation points is shown in Table 2.

Evaluation Point FC12 to Evaluation Point FC13:

Initial Distance:

Evaluation Point FC12 Elev. =	761.7	ft-msl
Evaluation Point FC13 Elev. =	660.0	ft-msl
Plan View Distance=	407.2	ft
L_o=	419.7	ft

Total Settlement:

Total Settlement Point FC12=	7.4	ft
Total Settlement Point FC13=	1.0	ft

Final Distance (after settlement):

Evaluation Point FC12 Elev. =	754.3	ft-msl
Evaluation Point FC13 Elev. =	659.0	ft-msl
Plan View Distance=	407.2	ft
L_f=	418.2	ft

Strain= -0.36%

Conclusions:

Strain is acceptable.

- Compacted clay component of final cover has the smallest average allowable tensile strain value which is 0.5 percent (Reference 2, Page 469).
- The allowable tensile strain for an LDPE and LLDPE geomembrane is 8 to 12 percent (Reference 8).
- The allowable tensile strain for a drainage geocomposite is more than 20 percent for the geotextile (reference 3, page 112) and 200 percent for the geonet (reference 3, page 400).
- The maximum calculated strain (-0.36%) represents compression versus tensile strain and is acceptable, therefore the system will be stable. No tensile strain was observed in the analysis results.

Prep By: MB
Date: 5/20/2024

ROYAL OAKS LANDFILL
0120-76-11-106
APPENDIX IIIE-B-2
FINAL COVER SETTLEMENT SUMMARY

Chkd By: DEP
Date: 5/20/2024

TABLE 1. FINAL COVER EVALUATION - SETTLEMENT SUMMARY[2]

Evaluation Point[1]	Initial Top of Final Cover Elevation (ft-msl)	Initial Top of Waste Elevation (ft-msl)	Bottom of Waste Elevation (ft-msl)	H_o (ft)	γ_{msw} (pcf)	σ'_o (psf)	$\Delta\sigma$ (psf)	e_o	C_c	S_p (ft)	H'_o (ft)	γ'_{msw} (pcf)	σ''_o (psf)	e'_o	α	S_c (ft)	Total Settlement (ft)	Post-Settlement Top of Final Cover Elevation (ft-msl)
FC1	761.7	758.2	577.8	180.4	68.0	6,133.6	420.0	1.56	0.55	1.1	179.3	68.0	6,096.2	1.56	0.05	8.4	9.5	752.2
FC2	570.0	566.5	561.3	5.2	43.0	111.8	420.0	1.85	0.65	0.8	4.4	42.0	92.4	1.86	0.06	0.2	1.0	569.0
FC3	630.0	626.5	625.2	1.3	42.0	27.3	420.0	1.86	0.65	0.4	0.9	42.0	18.9	1.86	0.06	0.0	0.4	629.6
FC4	761.7	758.2	626.3	131.9	61.0	4,023.0	420.0	1.66	0.58	1.2	130.7	61.0	3,986.4	1.67	0.05	6.3	7.5	754.2
FC5	650.0	646.5	627.9	18.6	44.0	409.2	420.0	1.84	0.64	1.3	17.3	44.0	380.6	1.84	0.06	0.9	2.2	647.8
FC6	769.0	765.5	630.2	135.3	62.0	4,194.3	420.0	1.66	0.58	1.2	134.1	61.0	4,090.1	1.66	0.05	6.4	7.6	761.4
FC7	765.0	761.5	644.0	117.5	59.0	3,466.3	420.0	1.69	0.59	1.3	116.2	59.0	3,427.9	1.69	0.05	5.6	6.9	758.1
FC8	650.0	646.5	634.6	11.9	43.0	255.9	420.0	1.85	0.65	1.1	10.8	43.0	232.2	1.85	0.06	0.5	1.6	648.4
FC9	761.7	758.2	594.3	163.9	66.0	5,408.7	420.0	1.60	0.56	1.1	162.8	66.0	5,372.4	1.60	0.05	7.7	8.8	752.9
FC10	765.0	761.5	652.9	108.6	58.0	3,149.4	420.0	1.71	0.60	1.3	107.3	58.0	3,111.7	1.71	0.05	5.2	6.5	758.5
FC11	670.0	666.5	655.5	11.0	43.0	236.5	420.0	1.85	0.65	1.1	9.9	43.0	212.9	1.85	0.06	0.5	1.6	668.4
FC12	761.7	758.2	627.5	130.7	61.0	3,986.4	420.0	1.67	0.58	1.2	129.5	61.0	3,949.8	1.67	0.05	6.2	7.4	754.3
FC13	660.0	656.5	651.4	5.1	43.0	109.7	420.0	1.85	0.65	0.8	4.3	42.0	90.3	1.86	0.06	0.2	1.0	659.0

[1] Refer to Sheet IIIE-B-2-12 for Evaluation Point locations (FC1 thru FC13).

[2]Settlement calculations in above table rounded to one significant figure.

TABLE 2. FINAL COVER EVALUATION - FINAL GRADES AND STRAIN SUMMARY

Evaluation Point[1]		Initial Top of Final Cover Elevation (ft-msl)		Post-Settlement Top of Final Cover Elevation (ft-msl)		Plan View Distance (ft)	L_o (ft)	L_f (ft)	Initial Slope (ft/ft)	Post-Settlement Slope (ft/ft)	Tensile Strain (%)
A	B	A	B	A	B						
FC1	FC2	761.7	570.0	752.2	569.0	762.7	786.5	784.4	0.25	0.24	-0.26
FC9	FC3	761.7	630.0	752.9	629.6	522.7	539.0	537.1	0.25	0.24	-0.37
FC4	FC5	761.7	650.0	754.2	647.8	446.8	460.5	459.3	0.25	0.24	-0.27
FC6	FC12	769.0	761.7	761.4	754.3	183.1	183.2	183.2	0.04	0.04	0.00
FC7	FC8	765.0	650.0	758.1	648.4	467.4	481.4	480.1	0.25	0.23	-0.26
FC10	FC11	765.0	670.0	758.5	668.4	479.7	489.0	488.1	0.20	0.19	-0.19
FC12	FC13	761.7	660.0	754.3	659.0	407.2	419.7	418.2	0.25	0.23	-0.36

[1] Refer to Sheet IIIE-B-2-12 for Evaluation Point locations. The "A" and "B" points represent the upgradient and downgradient endpoints, respectively.

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 SITE GRID
- 610 EXISTING CONTOUR (SEE NOTE 1)
- 624 PROPOSED FINAL COVER CONTOUR
- DRAINAGE SWALE
- DRAINAGE LETDOWN
- CHANNEL CENTERLINE
- FC2 FINAL COVER EVALUATION POINT
- FINAL COVER EVALUATION LINE

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION IS 776.5 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 774.5 FT-MSL.
4. TYPICAL SIDESLOPES ARE 4H:1V. TYPICAL TOPSLOPE IS 4%.
5. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY LANDES & ASSOCIATES, INC. DATED MARCH 2011.

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER
05/20/2024

- [ ] DRAFT
- [x] FOR PERMITTING PURPOSES ONLY
- [ ] ISSUED FOR CONSTRUCTION

DATE: 05/2024
FILE: 0120-76-11
CAD: IIIE-B-2-12 SETTELMENT.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: DEP

PREPARED FOR
PINE HILL FARMS LANDFILL TX, LP

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
SETTLE3 SETTLEMENT FINAL COVER ANALYSIS POINT PLAN

ROYAL OAKS LANDFILL
CHEROKEE COUNTY, TEXAS

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SHEET IIIE-B-2-12

O:\0120\76\EXPANSION 2023\PART III\IIIE\IIIE-B-2-12 SETTLMENT.dwg, byoung, 1:2

APPENDIX IIIE-B-3

FOUNDATION HEAVE ANALYSIS

Includes pages IIIE-B-3-1 through IIIE-B-3-4

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05/20/2024

ROYAL OAKS LANDFILL
0120-76-11-106
APPENDIX IIIE-B-3
FOUNDATION HEAVE

Required: Estimate the potential heave of the bottom of excavation resulting from the removal of overburden soils during liner construction.

Method: Heave will be analyzed for the proposed excavation in Sector 10 (southeast portion of the expansion area).

References:

1. Terzaghi, Karl and Peck, Ralph, Soil Mechanics in Engineering Principle, Third Edition, John Wiley and Sons, Inc, New York, 1996.
2. Das, Braja M., Principles of Geotechnical Engineering, Fourth Edition, PWS, Boston, 1998.
3. Day, Robert W., *Geotechnical Engineer's Portable Handbook* , McGraw-Hill, New York, 2000.
4. Dunn, I.S., Anderson, L.R., and Kiefer, F.W., Fundamentals of Geotechnical Analysis, 1st Edition, 1980.
5. Coduto, Donald P., Geotechnical Engineering Principles and Practices, 1999.
6. Acar, Yalcin B.& Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.

Foundation Heave Calculations

Estimate the potential heave of the excavation bottom in Sector 10.

Note: Evaluation location for the heave analysis is the shown as on Figure IIIE-B-3-4 (Heave Analysis Point 1).

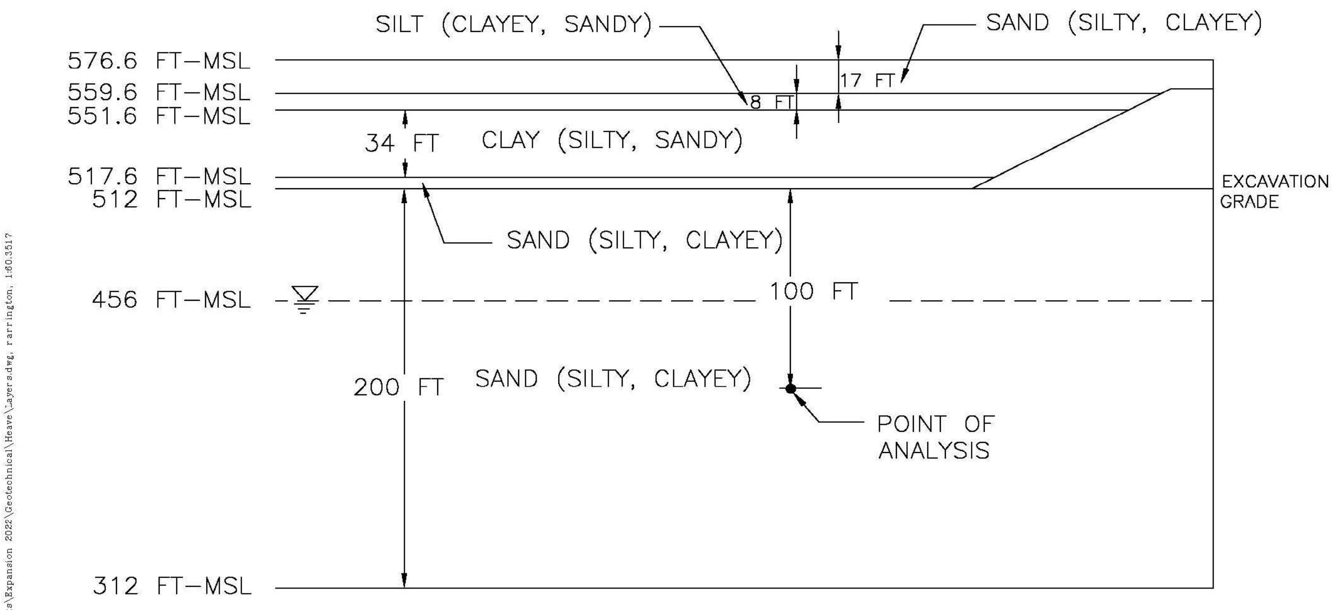
Excavation for liner construction will result in reduced overburden pressure on subgrade strata which may result in heave. Note that the heave within the marginally-elastic sands is expected to be minimal, and these calculations are conservative.

Method:

A. Select critical location for heave. The critical location is established as the location that has the estimated highest overburden pressure relief resulting from landfill excavation prior to liner installation. For this analysis it was assumed this point is in Sector 10 (Southeast corner of the expansion area).
B. Use unit weight values for the excavated soils and consolidation parameter values derived from available field and laboratory results and from estimates of similar materials.
C. Stratum elevations, thicknesses, and water table are shown on the below diagram for Boring PWCG-11.

Solution:

Diagram for Heave Analysis in Sector 10 (Southeast of the Expansion Area)



Definition of Terms/Variables:

e_o = initial void ratio
γ_d = Dry Unit Weight (pcf)
γ_{moist} = Moist Unit Weight (pcf)
γ_{sat} = Saturated Unit Weight (pcf)
γ_w = Unit Weight of Water (pcf)
γ_i = Assumed Unit Weight Stratum i (pcf)
D = Depth of Excavation
Di = Overburden depth of Stratum i (ft)
H_i = Thickness of soil layer (Stratum II thickness analyzed for heave)
P_o = Initial Average Effective Overburden Pressure (psf)
P_c = Preconsolidation Pressure (psf) (pressure in excess of overburden pressure, assumed zero)
ΔP = Change in Vertical Pressure (psf)
C_c = Compression Index
C_r = Recompression index (rebound portion of consolidation curve during unloading)

Based on the laboratory test results included in Appendix IIIE-C, the material properties of the soil overburden material to be excavated during liner construction are shown in following table:

	e_0	γ_d (pcf)	γ_m (pcf)	γ_{sat} (pcf)	C_c	C_r
Stratum I (Sand (Silty, Clayey)		101	120	127	na[2]	na[2]
Stratum II (Silt (Clayey, Sandy))		98	115	125	na[2]	na[2]
Stratum III (Clay (Silty, Sandy))	0.8	94	115	121	na[2]	na[2]
Stratum IV (Sand (Silty, Clayey))	0.6	115	120	135	0.08	0.038

[1]Average unit weight for four layers is used.

[2]Consolidation parameters are not needed for Stratums I, II and III.

[3]The Cr value for sand estimated as 50 percent of Cr value for clay. Note that this assumption is conservative, as true sands demonstrate minimal elastic uplift or heave during unloading. Heaving will occur in the interstitial clays and silts only.

The following parameters were used for Stratum VI heave calculations:

$H_i =$ 200 ft

$e_o =$ 0.6

$C_r =$ 0.0380

Estimate Potential Maximum Heave of the Excavation Bottom

The change in loading is due to the excavation of overburden soils.

$$\Delta P = D_I * \gamma_{I,\,moist} + D_{II} * \gamma_{II,\,moist} + D_{III} * \gamma_{III,\,moist} + D_{IV} * \gamma_{IV,\,moist} + D_V * \gamma_{V,\,sat}$$

$D_I =$ 17 ft (Sand)

$D_{II} =$ 8 ft (Silt)

$D_{III} =$ 34 ft (Clay)

$D_{IV} =$ 5.6 ft (Sand)

$\Delta P =$ 7,542 psf

Using the standard consolidation theory:

$S = C_r H_i \log ((P_o - \Delta P) / P_o)$ (at midpoint of Stratum III)

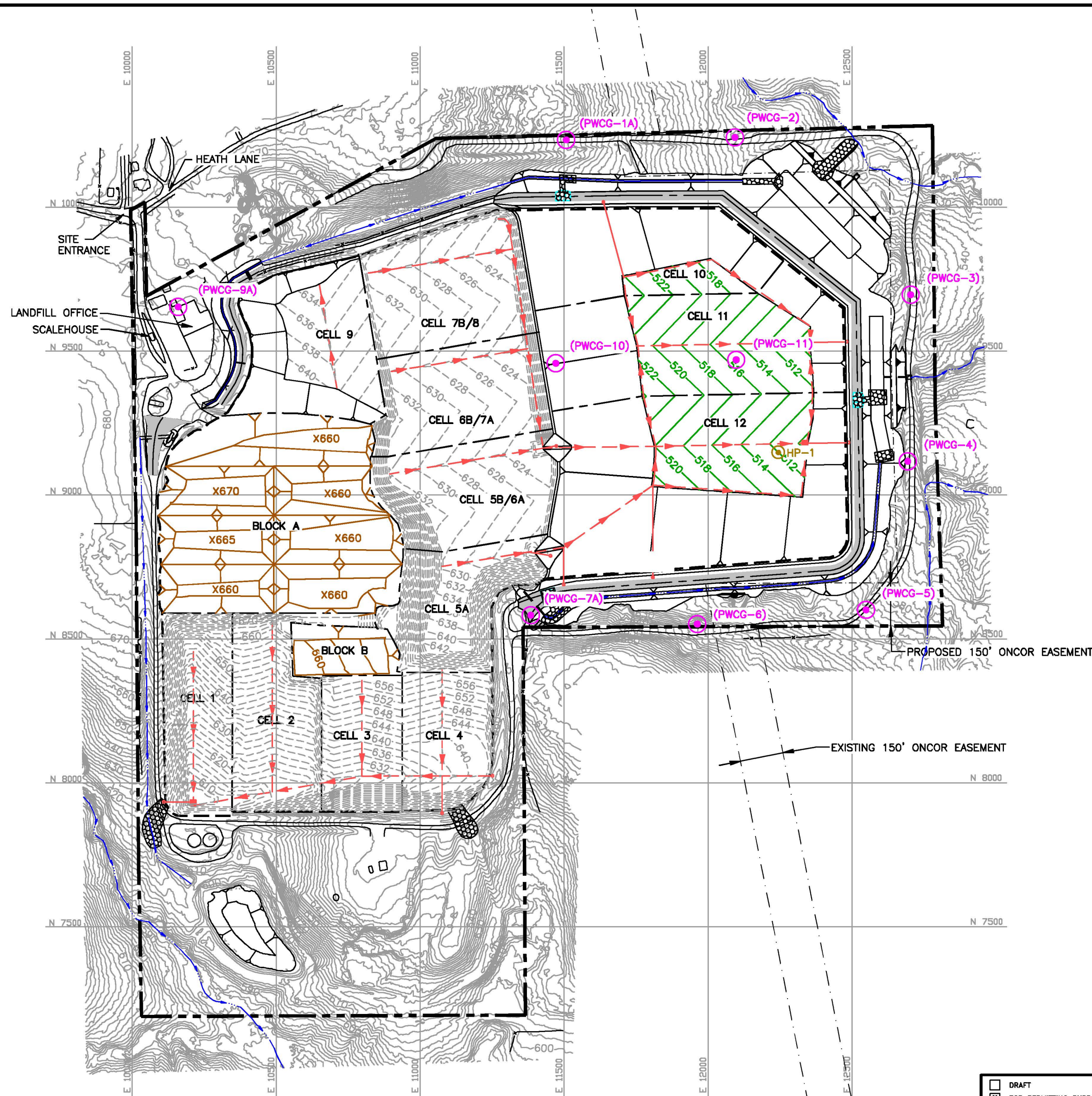
$P_o = ((H_i/2)*(\gamma_{III(sat)})) + \Delta P$ (assumed fully saturated foundation)

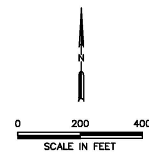
$P_o =$ 21,042.00 psf

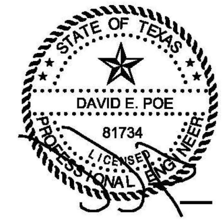
$S =$ -1.46 ft

Projected Heave[1] =	**-1.46**	**ft**	**or**	**-17.6**	**inches**

[1] Negative value represents heave or uplift of excavated foundation. Note that heave will be recovered during settlement of sector. As the settlement analysis conservatively does not incorporate actual preconsolidation stresses on formation, the actual heave and settlement will be less than calculated.







05/20/2024

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- CELL BOUNDARY
- EXISTING EASEMENT
- PROPOSED EASEMENT
- E 10500 — SITE GRID
- 610 — EXISTING CONTOUR (SEE NOTE 1)
- 516 — TOP OF LINER CONTOUR
- 630 — AS-BUILT TOP OF LINER
- X660 — APPROXIMATE TOP OF PRE-SUBTITLE D LINER (SEE NOTE 8)
- CHANNEL CENTERLINE
- LEACHATE COLLECTION PIPE
- LEACHATE COLLECTION SUMP
- LEACHATE RISER PIPE
- HP-1 — HEAVE ANALYSIS POINT
- (PWCG-1A) — BORING DESIGNATION (FOR DRILLING GEOLOGICAL INVESTIGATION)

NOTES:

1. EXISTING CONTOURS DEVELOPED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN NOVEMBER 10, 2022.
2. EXCAVATION SLOPES AND SLOPES OUTSIDE THE LIMIT OF WASTE (e.g., CHANNELS) ARE TYPICALLY 3H:1V.
3. PERMIT BOUNDARY WAS REPRODUCED FROM LEGAL DESCRIPTION PROVIDED BY STANGER SURVEYING COMPANY, DATED APRIL 1995.

☐ DRAFT ☒ FOR PERMITTING PURPOSES ONLY ☐ ISSUED FOR CONSTRUCTION	PREPARED FOR PINE HILL FARMS LANDFILL TX, LP	MAJOR PERMIT AMENDMENT SETTLE3 SETTLEMENT ANALYSIS HEAVE ANALYSIS POINT ROYAL OAKS LANDFILL CHEROKEE COUNTY, TEXAS
DATE: 05/2024 FILE: 0120-76-11 CAD: SHEET IIIE-B-3-4.DWG DRAWN BY: RAA DESIGN BY: MB REVIEWED BY: DEP	REVISIONS NO. / DATE / DESCRIPTION	
Weaver Consultants Group TBPE REGISTRATION NO. F-3727		WWW.WCGRP.COM — SHEET IIIE-B-3-4

0:\0120\76\EXPANSION 2023\PART III\IIIE\SHEET IIIE-B-3-4.dwg, byoung, 1:2

APPENDIX IIIE-C

LABORATORY TEST RESULTS

Includes pages IIIE-C-1 through IIIE-C-77

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

05/20/2024

LABORATORY TESTING

Introduction

This appendix presents the geotechnical laboratory test results for samples obtained by WCG during the 2023 geological investigation at the landfill. Some limited information was derived from the field and laboratory testing previously performed at the site, as summarized in Appendix IIIG-Geology Report. Copies of the lithological logs, geological sections, maps of regional geology, and in-depth description of the various strata is provided in Appendix IIIG-Geology Report and has not been reproduced for this appendix.

Geotechnical Data Summary

A summary of the geological field and laboratory testing is provided for each stratum in Section 3 of this appendix, including physical description of the individual stratum and a summary of laboratory testing results for the individual stratum. Further description and background information (e.g., logs, geological cross-sections) is provided in Appendix G – Geology Report.

ROYAL OAKS LANDFILL
TABLE IIIE-C-1
GEOTECHNICAL LABORATORY RESULTS

Boring ID	Test Interval	D1140	D2216	D4318	D4318	D4318	D4767 MOD[1]	D2435-B	D5084
		< #200 (%)	MC (%)	LL	PL	PI	Triaxial	Consol.	Vert. Perm (cm/sec)
PWCG-01A	55.5-60	100.9	29	64	21	43			1.60E-06
PWCG-01A	60-69	99.4	23.4	57	21	36	Effective: C=43.2 psf, φ=29.7 Total: C=1006 psf, φ=19.4		
PWCG-01A	102.5-112.5	12.6	16.2	--	--	--			
PWCG-01A	230-235	29.8	25.9	--	--	--			
PWCG-01A	295-300	100.2	21.1	46	18	28			3.60E-08
PWCG-02	10-15	70	20.2	50	20	30			
PWCG-02	25-30	95.4	30.7	--	--	--			
PWCG-02	60-64	22.9	13	--	--	--			
PWCG-02	115-120	34	22.1	--	--	--			
PWCG-03	15-20	83.5	14.4	41	19	22			
PWCG-03	40-51	55.8	4.3	--	--	--			
PWCG-03	95-100.5	29.2	27.9	--	--	--			
PWCG-03	120-135	20.7	21.7	--	--	--			
PWCG-03	230-231	94.5	32.7	52	25	27			
PWCG-03	232-234	93.3	22.9	63	21	42			1.4E-05 (230-231') 8.1E-09 (232-234')
PWCG-04	7.5-12.5	99.9	23	47	17	30			
PWCG-04	17.5-22.5	97.8	26.9	--	--	--			
PWCG-04	30-35	88	31.7	47	19	28			1.80E-06
PWCG-04	75-80	28.3	10.9	--	--	--			
PWCG-04	115-121.5	30.6	18	--	--	--			
PWCG-05	30-35	93.5	26.5	55	25	30		Cc = 0.17	2.90E-07
PWCG-05	70-75	81	21.6	28	10	18			
PWCG-05	90-95	66.2	11.2	--	--	--			
PWCG-06	10-15	17.1	19.3	--	--	--			
PWCG-06	35-40	86.4	18.4	38	12	26			7.10E-07
PWCG-06	75-85	20.5	15.9	39	21	18			2.20E-05
PWCG-06	135-140	75.8	12.5	--	--	--			
PWCG-06	185-190	40.7	14.5	--	--	--			
PWCG-07	44.5-47	58.2	45.2	--	--	--			
PWCG-07	60-65	98.6	31.1	51	18	33			
PWCG-07	99.5-104.5	21.5	24.7	--	--	--			6.10E-05
PWCG-07	127-132.5	94.1	16.1	NL	NP	--			
PWCG-07	354.5-359.5	99.7	20.4	53	21	32			3.50E-05
PWCG-07	359.5-360.5	--	--	--	--	--			2.30E-07
PWCG-07	360.5-362	--	--	--	--	--			1.60E+06

ROYAL OAKS LANDFILL
TABLE IIIE-C-1
GEOTECHNICAL LABORATORY RESULTS

Boring ID	Test Interval	D1140	D2216	D4318	D4318	D4318	D4767 MOD[1]	D2435-B	D5084
		< #200 (%)	MC (%)	LL	PL	PI	Triaxial	Consol.	Vert. Perm (cm/sec)
PCWG-08	187-199.5	**24.1**	**23.1**	**NL**	**NP**	**1**			**7.30E-06**
PCWG-09	125-130	**99.4**	**18.2**	**58**	**21**	**37**			**7.60E-07**
WCG-10	9-14.5	**39.5**	**34.8**	**58**	**32**	**26**			
WCG-10	24.5-27	**94.5**	**23.8**	**--**	**--**	**--**			
WCG-10	52-57	**28.3**	**22.1**	**--**	**--**	**--**			
WCG-10	92-99.5	**91.6**	**29.5**	**64**	**22**	**42**			**4.50E-08**
WCG-10	102-104.5	**92.9**	**17.3**	**44**	**25**	**19**		**Cc = 0.15**	
WCG-10	132-140	**77.3**	**16.8**	**--**	**--**	**--**			
WCG-11	17-27	**94**	**24.1**	**43**	**17**	**26**			**3.10E-08**
WCG-11	44.5-57	**91**	**20.8**	**53**	**28**	**25**	**Effective: C=835 psf, φ=19.2** **Total: C=1080 psf, φ=14.2**	**Cc = 0.17**	
WCG-11	64.5-69.5	**49.5**	**20.5**	**--**	**--**	**--**			
WCG-11	92-109.5	**34.1**	**9.4**	**--**	**--**	**--**			

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill

TRI Log #: 23-003876

Jeffrey A. Kuhn, Ph.D., P.E., 11/1/2023
Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)
-	Test Method	ASTM D2216	ASTM D1140
1	PWCG-01A (55.5-60)	29.0	100.9
2	PWCG-01A (60-69)	23.4	99.4
3	PWCG-01A (102.5-112.5)	16.2	12.6
4	PWCG-01A (230-235)	25.9	29.8
5	PWCG-01A (295-300)	21.1	100.2
7	PWCG-02 (25-30)	30.7	95.4

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Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

TRI Log #: 23-003876

Jeffrey A. Kuhn, Ph.D., P.E., 11/21/2023

Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)	Atterberg Limits		
				Liquid Limit	Plastic Limit	Plasticity Index
-	Test Method	ASTM D2216	ASTM D1140	ASTM D4318, Method A : Multipoint		
1	PWCG-01A (55.5-60)	29.0	100.9	64	21	43
2	PWCG-01A (60-69)	23.4	99.4	57	21	36
3	PWCG-01A (102.5-112.5)	16.2	12.6	-	-	-
4	PWCG-01A (230-235)	25.9	29.8	-	-	-
5	PWCG-01A (295-300)	21.1	100.2	46	18	28
6	PWCG-02 (10-15)	20.2	-	50	20	30
7	PWCG-02 (25-30)	30.7	95.4	-	-	-

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Client: Weaver Consultants Group
Project: 0120-076-11-106 Royal Oaks Landfill

TRI Log #: 23-003892

Jeffrey A. Kuhn, Ph.D., P.E., 12/1/2023
Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)	Atterberg Limits		
				Liquid Limit	Plastic Limit	Plasticity Index
-	Test Method	ASTM D2216	ASTM D1140	ASTM D4318, Method A : Multipoint		
1	PWCG-03 (15-20)	14.4	83.5	41	19	22
5	PWCG-03 (232-234)	22.9	93.3	63	21	42
6	PWCG-04 (7.5-12.5)	23.0	99.9	47	17	30
8	PWCG-04 (30-35)	31.7	88.0	47	19	28
10	PWCG-04 (115-121.5)	18.0	30.6	-	-	-

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Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

TRI Log #: 23-003895

Jeffrey A. Kuhn, Ph.D., P.E., 12/7/2023

Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)	Atterberg Limits			Liquidity Index (%)
				Liquid Limit	Plastic Limit	Plasticity Index	
-	Test Method	ASTM D2216	ASTM D1140	ASTM D4318, Method A : Multipoint			ASTM D1140
1	PWCG-05 (30-35)	26.5	93.5	55	25	30	5
2	PWCG-05 (70-75)	21.6	81.0	28	10	18	64
5	PWCG-06 (35-40)	18.4	86.4	38	12	26	25
6	PWCG-06 (75-85)	15.9	20.5	39	21	18	-28

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Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

TRI Log #: 23-003901

Jeffrey A. Kuhn, Ph.D., P.E., 11/1/2023

Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)
-	Test Method	ASTM D2216	ASTM D1140
2	PWCG-07 (60-65)	31.3	98.6
4	PWCG-07 (127-132.5)	16.1	94.1
5	PWCG-07 (239.5-244.5)	24.3	22.8
6	PWCG-07 (354.5-359.5)	20.4	99.7
7	PWCG-08 (187-199.5)	23.1	24.1
8	PWCG-09 (125-130)	18.2	99.4

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Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill

TRI Log #: 23-003901

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Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)	Atterberg Limits		
				Liquid Limit	Plastic Limit	Plasticity Index
-	Test Method	ASTM D2216	ASTM D1140	ASTM D4318, Method A : Multipoint		
2	PWCG-07 (60-65)	31.3	98.6	51	18	33
4	PWCG-07 (127-132.5)	16.1	94.1	-	-	-
5	PWCG-07 (239.5-244.5)	24.3	22.8	NL	NP	-
6	PWCG-07 (354.5-359.5)	20.4	99.7	53	21	32
7	PWCG-08 (187-199.5)	23.1	24.1	NL	NP	-
8	PWCG-09 (125-130)	18.2	99.4	58	21	37

Note: NL = No Liquid Limit; NP = No Plastic Limit

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Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

TRI Log #: 23-003912

Jeffrey A. Kuhn, Ph.D., P.E., 12/7/2023

Quality Review/Date

COC Line #	Sample Identification	Moisture Content (%)	Fines (%)	Atterberg Limits			Liquidity Index (%)
				Liquid Limit	Plastic Limit	Plasticity Index	
-	Test Method	ASTM D2216	ASTM D1140	ASTM D4318, Method A : Multipoint			ASTM D1140
1	WCG-10 (9-14.5)	34.8	39.5	58	32	26	11
4	WCG-10 (92-99.5)	29.5	91.6	64	22	42	18
5	WCG-10 (102-104.5)	17.3	92.9	44	25	19	-41
7	WCG-11 (17-27)	24.1	94.0	43	17	26	27
8	WCG-11 (44.5-57)	20.8	91.0	53	28	25	-29

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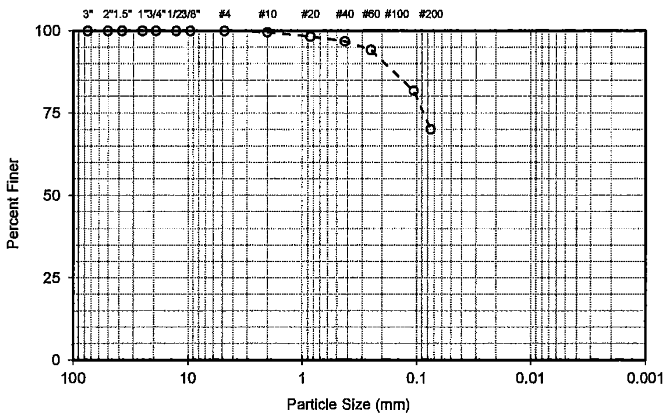
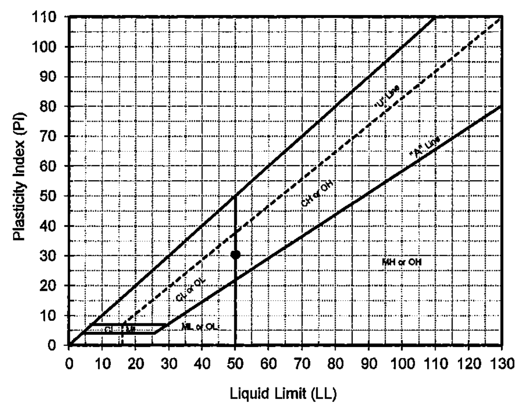
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client:	Weaver Consultants Group	TRI Log #:	23-003876.6
Project:	0120-076-11-01 Royal Oaks Landfill		
Sample ID:	PWCG-02 (10-15)		





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	30.0	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.6		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	98.3					
No. 40	0.420	96.8		mm		mm	
No. 60	0.250	94.2		0.002	--	0.002	--
No. 140	0.106	81.8		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	70.0	70.0	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	50
Plastic Limit	20
Plastic Index	30
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.3E-01	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
FAT CLAY (CH)

Moisture Content (%)	ASTM D2216	20.2
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Jeffrey A. Kuhn, Ph.D, P.E. 11/21/2023

Analysis & Quality Review/Date

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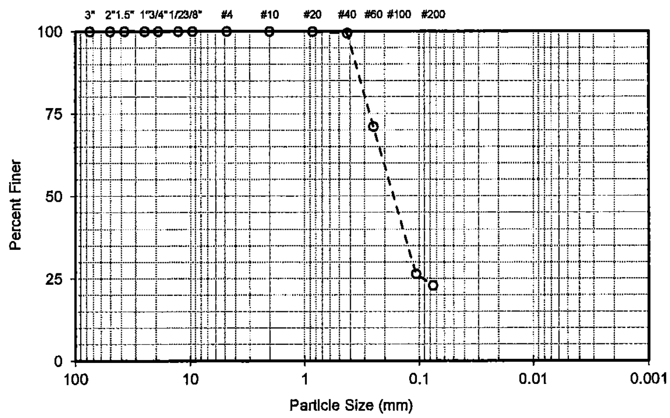
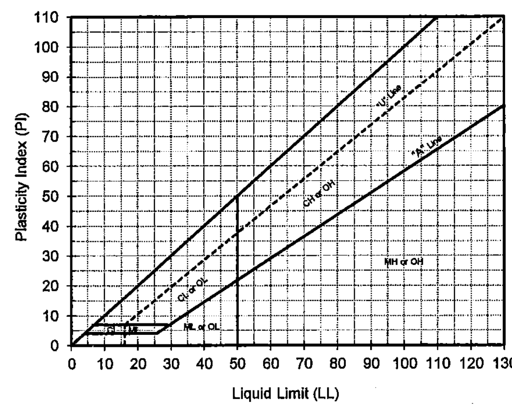
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client: Weaver Consultants Group — TRI Log #: 23-003876.8
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: PWCG-02 (60-64)





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	77.1	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	100.0		Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
No. 20	0.841	99.9					
No. 40	0.420	99.4					
No. 60	0.250	71.0		0.002	--	0.002	--
No. 140	0.106	26.3		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	22.9	22.9	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	3.2E-01	2.0E-01	1.7E-01	1.1E-01	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	13.0
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Jeffrey A. Kuhn, Ph.D, P.E. 11/1/2023
Analysis & Quality Review/Date

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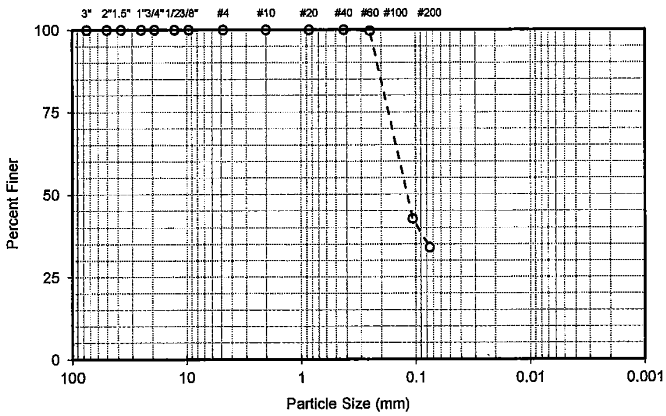
Particle Size, Atterberg Limit, and USCS Analyses for Soils

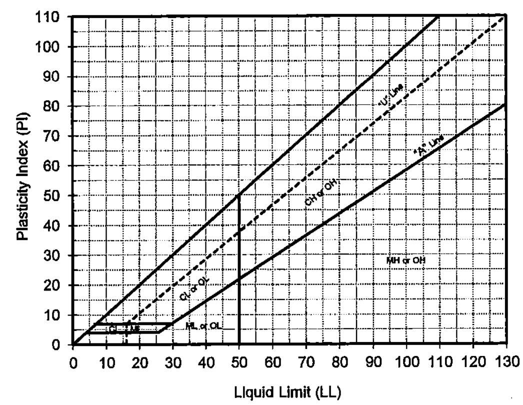
Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: PWCG-02 (115-120)

TRI Log #: 23-003876.9





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	66.0	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0		mm		mm	
No. 40	0.420	99.9					
No. 60	0.250	99.6		0.002	--	0.002	--
No. 140	0.106	42.6		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	34.0	34.0	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.0E-01	1.4E-01	1.2E-01	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	22.1
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Jeffrey A. Kuhn, Ph.D, P.E. 11/1/2023

Analysis & Quality Review/Date

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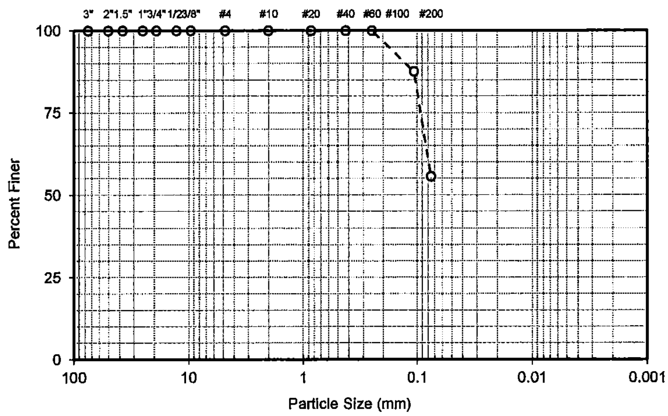
Particle Size, Atterberg Limit, and USCS Analyses for Soils

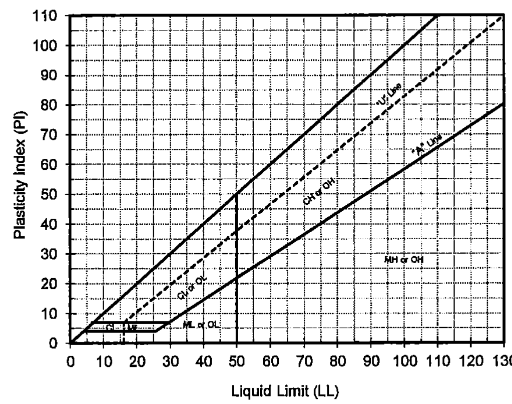
Client: Weaver Consultants Group

TRI Log #: 23-003892.2

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-03 (40-51)





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm		Fines	mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	44.4	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0					
No. 40	0.420	100.0		mm		mm	
No. 60	0.250	99.9		0.002	- -	0.002	- -
No. 140	0.106	87.5		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	55.6	55.6	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.0E-01	7.9E-02	- -	- -	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	4.3
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

Jeffrey A. Kuhn, Ph.D, P.E. 11/1/2023

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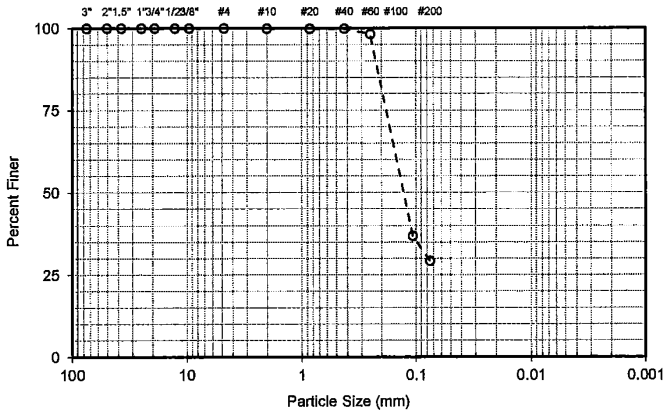
Particle Size, Atterberg Limit, and USCS Analyses for Soils

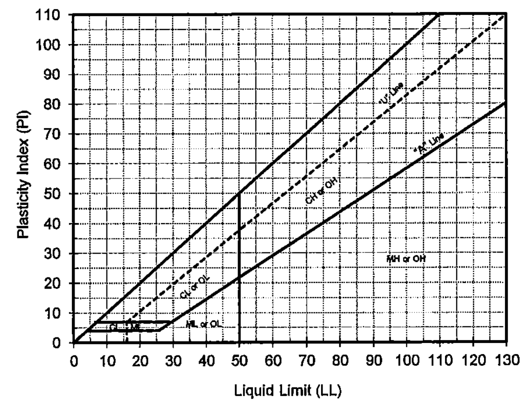
Client: Weaver Consultants Group

TRI Log #: 23-003892.3

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-03 (95-100.5)





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	70.8	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.9		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.8					
No. 40	0.420	99.8		mm		mm	
No. 60	0.250	98.2		0.002	- -	0.002	- -
No. 140	0.106	36.8		N m,2µm,d	-	N m,2µm,nd	-
No. 200	0.074	29.2	29.2	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.1E-01	1.5E-01	1.3E-01	7.8E-02	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	27.9
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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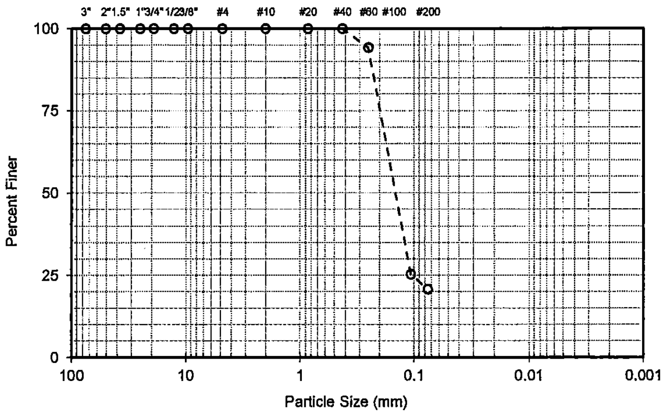
Particle Size, Atterberg Limit, and USCS Analyses for Soils

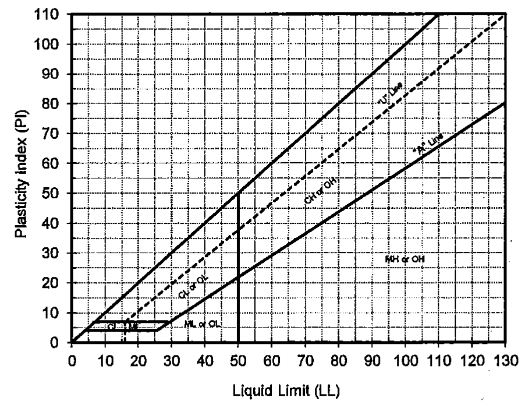
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-03 (120-135)

TRI Log #: 23-003892.4





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm		Fines	mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	79.3	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0					
No. 40	0.420	99.9		mm		mm	
No. 60	0.250	94.2		0.002	- -	0.002	- -
No. 140	0.106	25.2		N m,2µm,d	-	N m,2µm,nd	-
No. 200	0.074	20.7	20.7	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.2E-01	1.6E-01	1.4E-01	1.1E-01	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	21.7
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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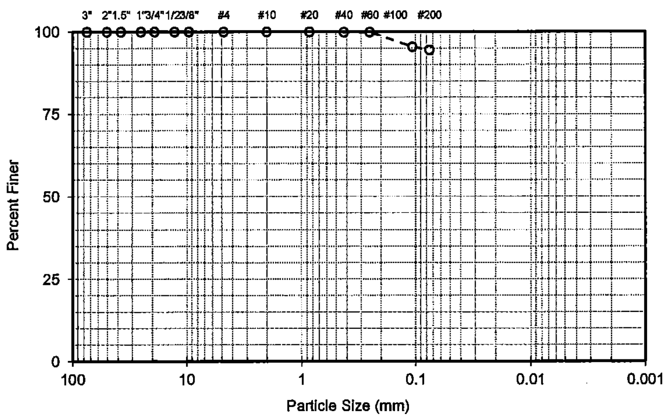
Particle Size, Atterberg Limit, and USCS Analyses for Soils

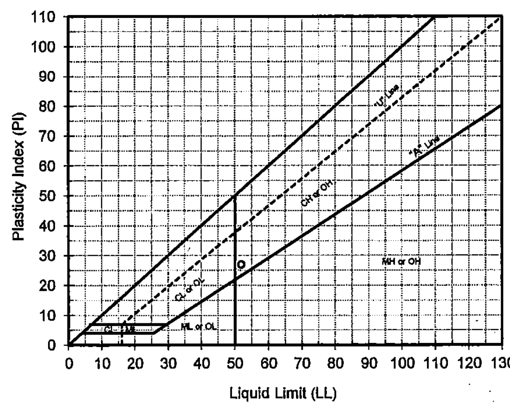
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-3 (230'-231')

TRI Log #: 23-001381.1





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	5.5	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0					
No. 40	0.420	99.9		mm		mm	
No. 60	0.250	99.9		0.002	--	0.002	--
No. 140	0.106	95.4		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	94.5	94.5	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	52
Plastic Limit	25
Plastic Index	27
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	--	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
Fat clay (CH)

Moisture Content (%)	ASTM D2216	32.7
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Kelby Broussard 5/5/2023

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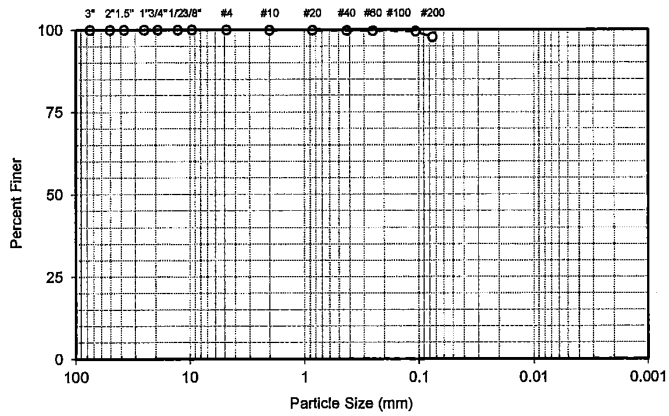
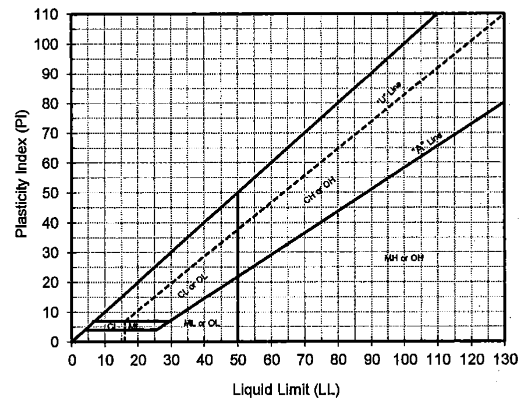
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client: Weaver Consultants Group
Project: 0120-076-11-106 Royal Oaks Landfill
Sample ID: PWCG-04 (17.5-22.5)

TRI Log #: 23-003892.7





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	2.2	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.9		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.8		mm		mm	
No. 40	0.420	99.7					
No. 60	0.250	99.7		0.002	--	0.002	--
No. 140	0.106	99.5		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	97.8	97.8	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	--	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	26.9
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

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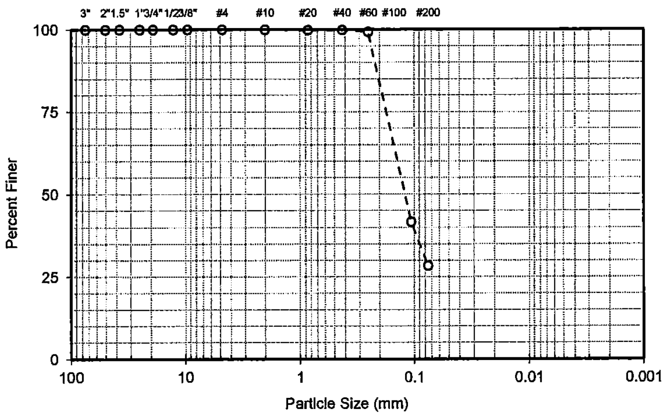
Particle Size, Atterberg Limit, and USCS Analyses for Soils

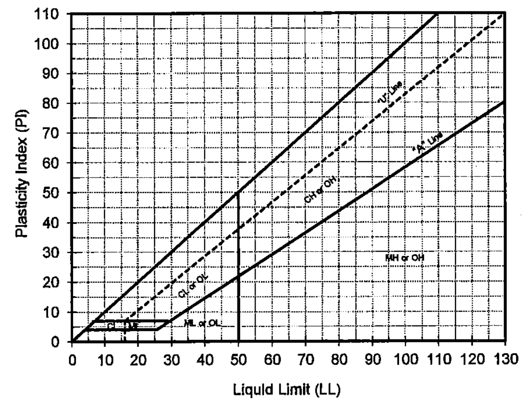
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-04 (75-80)

TRI Log #: 23-003892.9





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	71.7	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	99.9		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.8					
No. 40	0.420	99.8		mm		mm	
No. 60	0.250	99.2		0.002	- -	0.002	- -
No. 140	0.106	41.6		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	28.3	28.3	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.0E-01	1.4E-01	1.2E-01	7.8E-02	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	10.9
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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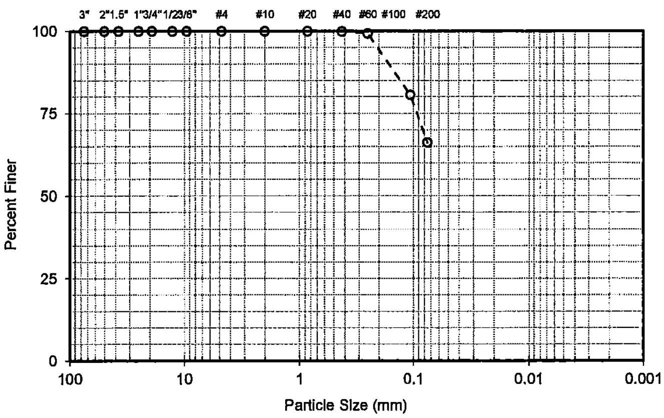
Particle Size, Atterberg Limit, and USCS Analyses for Soils

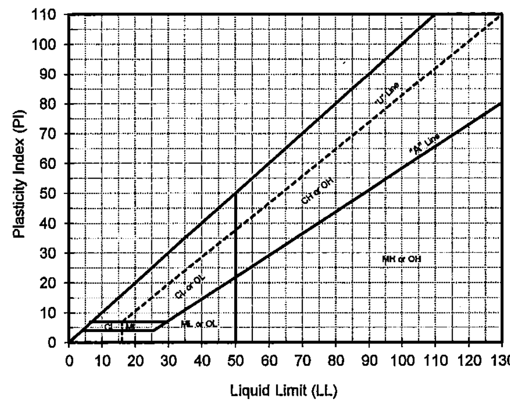
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-05 (90-95)

TRI Log #: 23-003895.3





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	33.8	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0		mm		mm	
No. 40	0.420	99.9					
No. 60	0.250	99.3		0.002	--	0.002	--
No. 140	0.106	80.6		N m,2µm,d	-	N m,2µm,nd	-
No. 200	0.074	66.2	66.2	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.3E-01	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	11.2
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Kelby Broussard 12/4/2023

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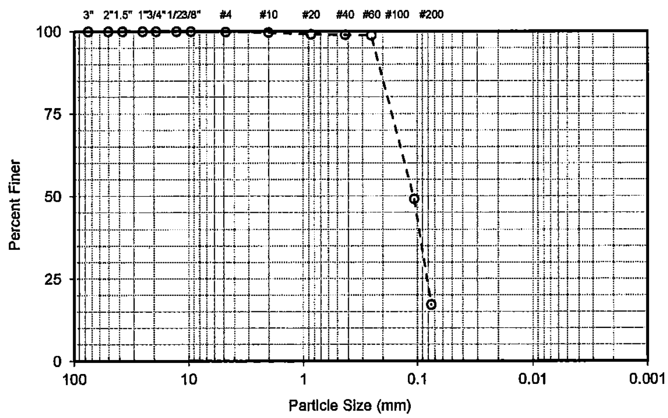
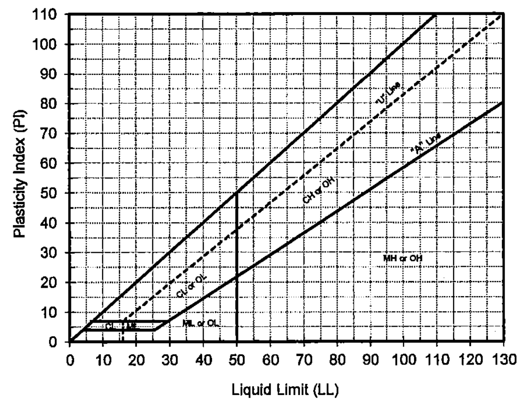
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client:	Weaver Consultants Group	TRI Log #: 23-003895.4
Project:	0120-076-11-106 Royal Oaks Landfill	67?
Sample ID:	PWCG-08 (10-15)	





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.2	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	99.8	82.7	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	99.7		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.1					
No. 40	0.420	98.9		mm		mm	
No. 60	0.250	98.7		0.002	--	0.002	--
No. 140	0.106	49.1		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	17.1	17.1	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.0E-01	1.3E-01	1.1E-01	8.6E-02	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	19.3
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Kelby Broussard 12/4/2023

Analysis & Quality Review/Date

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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

TRI Log #: 23-003895-6

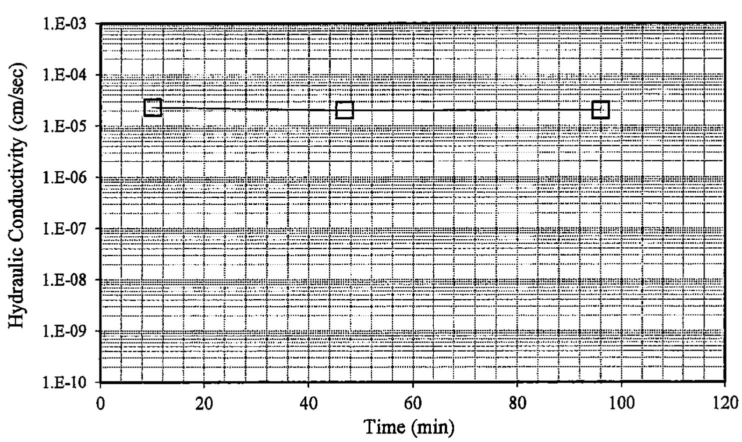
Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-06 (75-85)

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.03	4.10
Height (in)	3.69	3.72
Mass (g)	1549.2	1587.4
Sample Area (in^2)	12.76	13.23
Water Content (%)	16.4	21.5
Total Unit Weight (pcf)	125.5	122.9
Dry Unit Weight (pcf)	107.7	101.1
Specific Gravity (Assumed)	2.75	
Degree of Saturation	76.3	84.8
Void Ratio	0.59	0.70
Porosity	0.37	0.41
1 Pore Volume (cc)	286.8	331.0

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.98
Permeant	De-Aired Tap Water

Method C—Falling Head, rising tailwater elevation				
Time, t	Initial Gradient	Final Gradient	Inflow / Outflow	K_{20}
Min	-	-	-	cm/s
10.0	2.5	2.2	1.08	2.3E-05
46.8	2.2	1.4	1.06	2.0E-05
96.0	1.4	0.8	1.11	2.0E-05
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
Average, Last 4 Readings				**2.2E-05**



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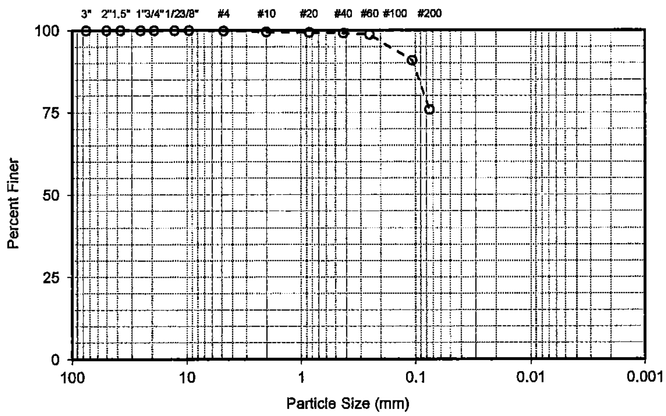
Particle Size, Atterberg Limit, and USCS Analyses for Soils

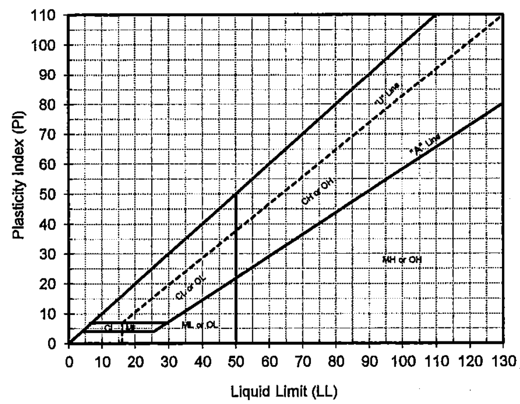
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-06 (135-140)

TRI Log #: 23-003895.7





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm		Fines	mm		mm	
3 in.	76.2	100.0	0.2	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	99.8	24.0	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.5		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.3					
No. 40	0.420	99.1		mm		mm	
No. 60	0.250	98.7		0.002	--	0.002	--
No. 140	0.106	90.9		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	75.8	75.8	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	9.3E-02	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	12.5
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Kelby Broussard 12/4/2023

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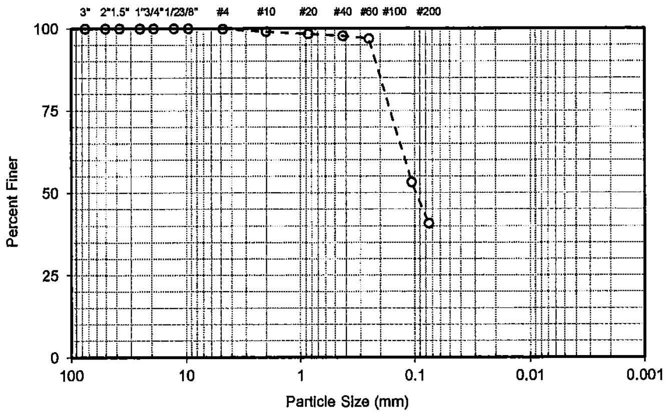
Particle Size, Atterberg Limit, and USCS Analyses for Soils

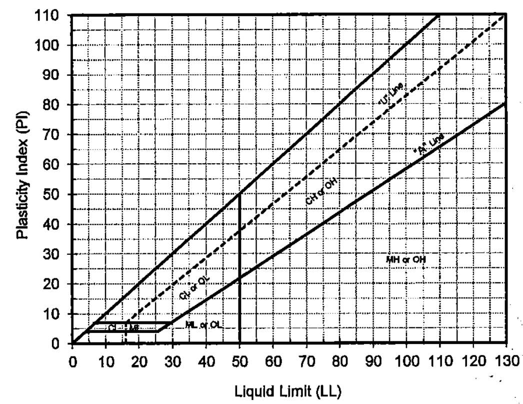
Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-06 (185-190)

TRI Log #: 23-003895.8





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm		Fines	mm		mm	
3 in.	76.2	100.0	0.1	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	99.9	59.2	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.1		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	98.3					
No. 40	0.420	97.7		mm		mm	
No. 60	0.250	96.9		0.002	--	0.002	--
No. 140	0.106	53.2		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	40.7	40.7	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.0E-01	1.2E-01	9.7E-02	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	14.5
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Kelby Broussard 12/4/2023

Analysis & Quality Review/Date

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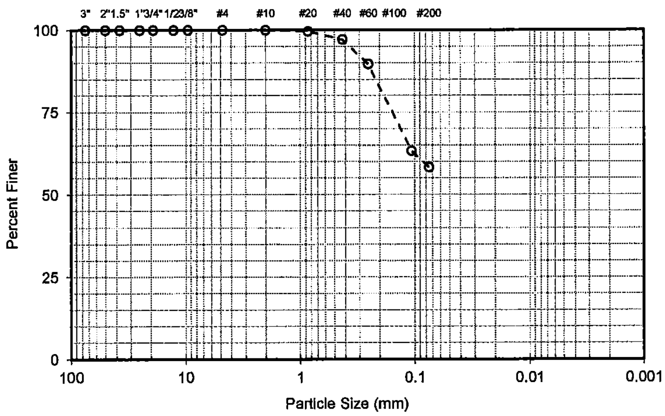
Particle Size, Atterberg Limit, and USCS Analyses for Soils

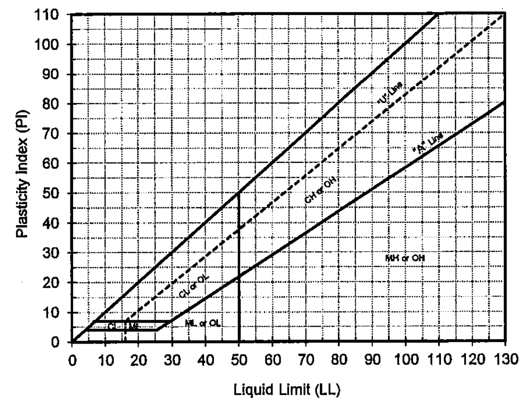
Client: Weaver Consultants Group

TRI Log #: 23-003901.1

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: PWCG-07 (44.5-47)





Mechanical Sieve ASTM D6913			
Sieve Designation		Percent Passing	Gravel / Sand / Fines
-	mm		
3 in.	76.2	100.0	0.0
2 in.	50.8	100.0	
1.5 in.	38.1	100.0	
1 in.	25.4	100.0	
3/4 in.	19.0	100.0	
1/2 in.	12.7	100.0	
3/8 in.	9.51	100.0	
No. 4	4.76	100.0	41.8
No. 10	2.00	100.0	
No. 20	0.841	99.6	
No. 40	0.420	97.1	
No. 60	0.250	89.7	
No. 140	0.106	63.2	
No. 200	0.074	58.2	58.2

Sedimentation (Hydrometer)			
ASTM D7928		ASTM D4221	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
- -	- -	- -	- -
- -	- -	- -	- -
- -	- -	- -	- -
- -	- -	- -	- -
- -	- -	- -	- -
- -	- -	- -	- -
- -	- -	- -	- -

Hydrometer Log-Linear Interpolation			
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
0.002	- -	0.002	- -
N m,2μm,d	-	N m,2μm,nd	-
Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.1E-01	8.5E-02	- -	- -	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	45.2
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

Jeffrey A. Kuhn, Ph.D, P.E. 11/1/2023

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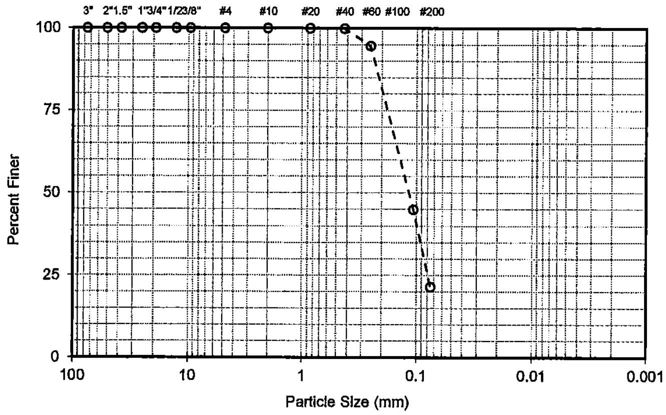
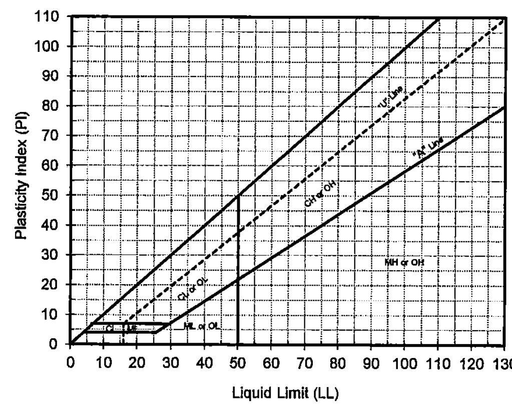
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client:	Weaver Consultants Group	TRI Log #:	23-003901.3
Project:	0120-076-11-01 Royal Oaks Landfill		
Sample ID:	PWCG-07 (99.5-104.5)		





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	78.5	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	100.0					
No. 40	0.420	99.9		mm		mm	
No. 60	0.250	94.6		0.002	- -	0.002	- -
No. 140	0.106	44.9		N m,2µm,d	-	N m,2µm,nd	-
No. 200	0.074	21.5	21.5	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	NL
Plastic Limit	NP
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	2.1E-01	1.4E-01	1.2E-01	8.5E-02	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	24.7
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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Analysis & Quality Review/Date

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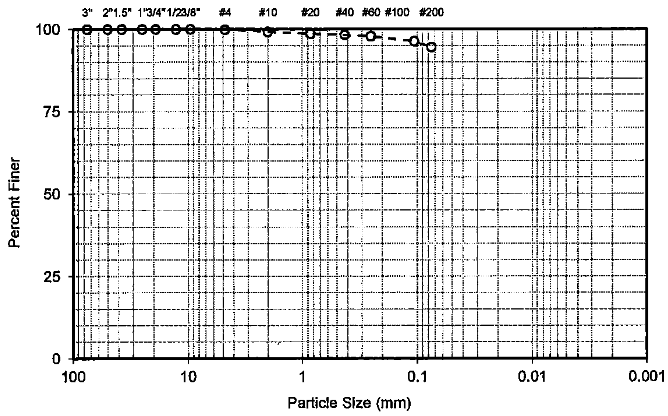
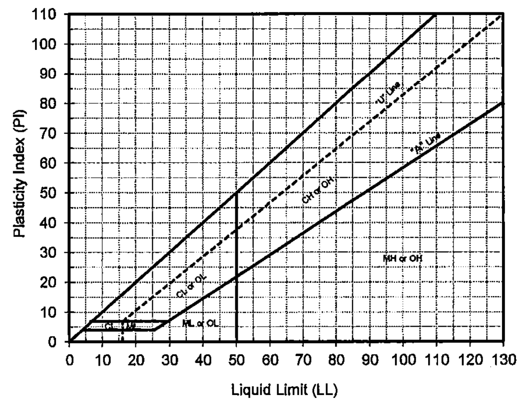
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: WCG-10 (24.5-27)
TRI Log #: 23-003912.2





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.2	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	99.8	5.3	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	99.1		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	98.6					
No. 40	0.420	98.2		mm		mm	
No. 60	0.250	97.8		0.002	--	0.002	--
No. 140	0.106	96.4		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	94.5	94.5	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	--	--	--	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	23.8
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Jeffrey A. Kuhn, Ph.D, P.E. 12/4/2023
Analysis & Quality Review/Date

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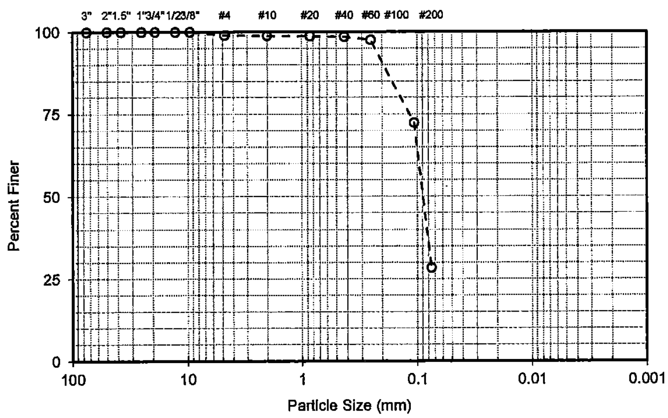
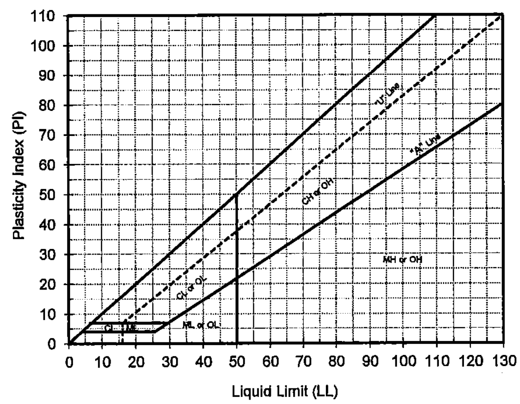
Particle Size, Atterberg Limit, and USCS Analyses for Soils

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: WCG-10 (52-57)

TRI Log #: 23-003912.3





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	1.1	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	98.9	70.6	Hydrometer Log-Linear Interpolation			
No. 10	2.00	98.8		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	98.7		mm		mm	
No. 40	0.420	98.6					
No. 60	0.250	97.5		0.002	--	0.002	--
No. 140	0.106	72.3		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	28.3	28.3	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.6E-01	9.6E-02	8.9E-02	7.6E-02	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	22.1
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

Jeffrey A. Kuhn, Ph.D, P.E. 12/4/2023
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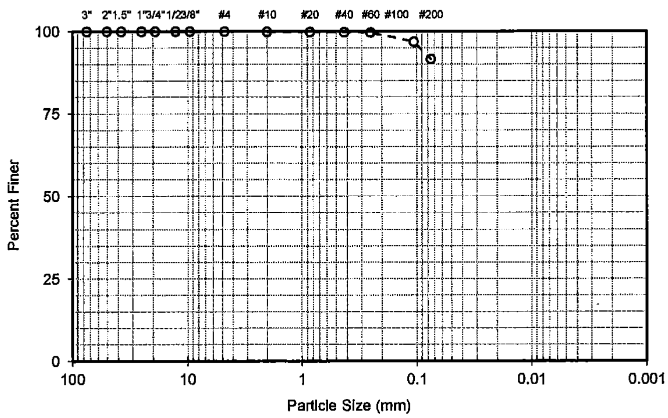
Particle Size, Atterberg Limit, and USCS Analyses for Soils

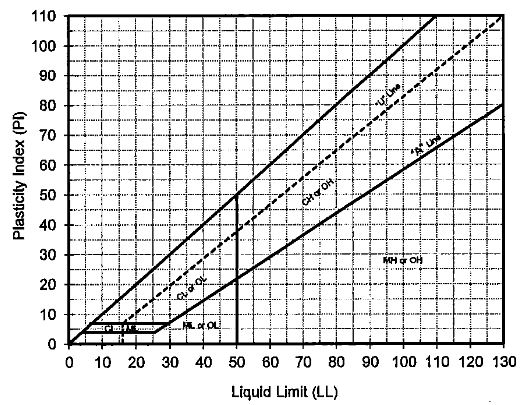
Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: WCG-10 (92-99.5)

TRI Log #: 23-003912.4





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	8.4	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	99.9		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.8					
No. 40	0.420	99.7		mm		mm	
No. 60	0.250	99.6		0.002	- -	0.002	- -
No. 140	0.106	96.8		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	91.6	91.6	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	- -	- -	- -	- -	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	29.5
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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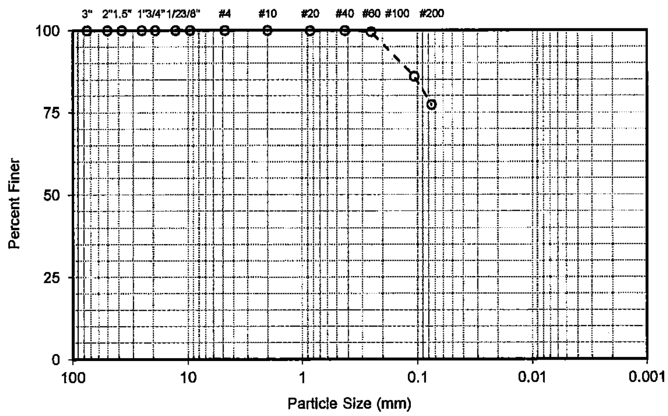
Particle Size, Atterberg Limit, and USCS Analyses for Soils

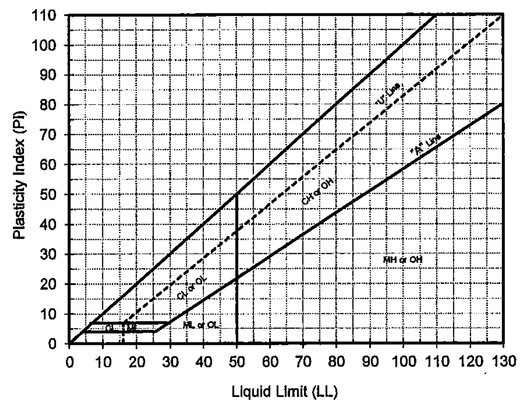
Client: Weaver Consultants Group

TRI Log #: 23-003912.6

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: WCG-10 (132-140)





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	- -	- -	- -	- -
2 in.	50.8	100.0		- -	- -	- -	- -
1.5 in.	38.1	100.0		- -	- -	- -	- -
1 in.	25.4	100.0		- -	- -	- -	- -
3/4 in.	19.0	100.0		- -	- -	- -	- -
1/2 in.	12.7	100.0		- -	- -	- -	- -
3/8 in.	9.51	100.0		- -	- -	- -	- -
No. 4	4.76	100.0	22.7	Hydrometer Log-Linear Interpolation			
No. 10	2.00	100.0		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.9					
No. 40	0.420	99.9		mm		mm	
No. 60	0.250	99.5		0.002	- -	0.002	- -
No. 140	0.106	86.0		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	77.3	77.3	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	- -
Plastic Limit	- -
Plastic Index	- -
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.0E-01	- -	- -	- -	- -

Cu	- -	Cc	- -

USCS Classification (ASTM D2487)
- -

Moisture Content (%)	ASTM D2216	16.8
Organic Content (%)	ASTM D2974-C	- -
Carbonate Content (%)	ASTM D4373	- -
Specific Gravity	ASTM D854	- -

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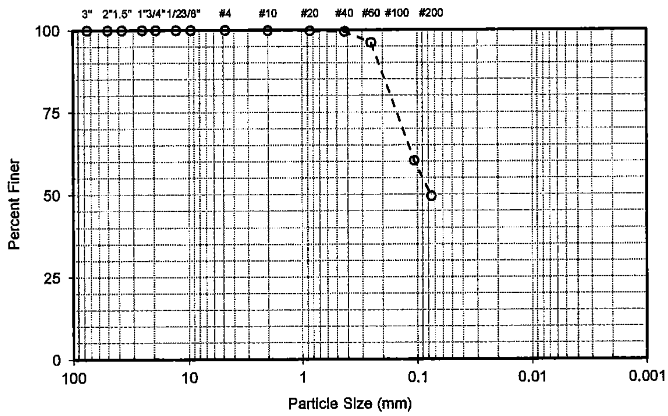
Particle Size, Atterberg Limit, and USCS Analyses for Soils

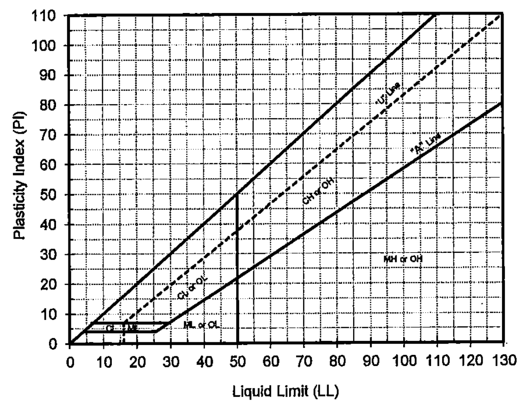
Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: WCG-11 (64.5-69.5)

TRI Log #: 23-003912.9





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand / Fines	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm			mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	50.5	Hydrometer Log-Linear Interpolation			
No. 10	2.00	99.9		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	99.7		mm		mm	
No. 40	0.420	99.5					
No. 60	0.250	96.2		0.002	--	0.002	--
No. 140	0.106	60.2		N m,2µm,d	-	N m,2µm,nd	-
No. 200	0.074	49.5	49.5	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.9E-01	1.1E-01	7.6E-02	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	20.5
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

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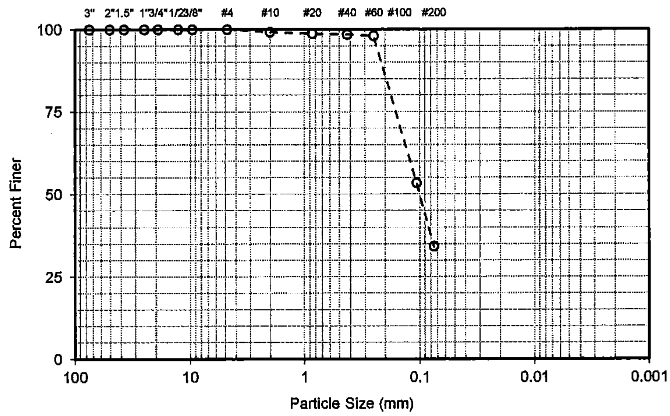
Particle Size, Atterberg Limit, and USCS Analyses for Soils

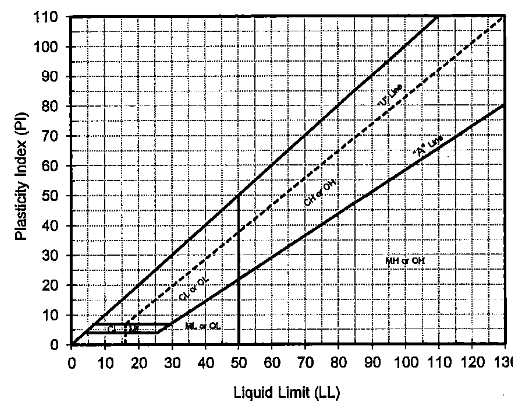
Client: Weaver Consultants Group

TRI Log #: 23-003912.10

Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: WCG-11 (92-109.5)





Mechanical Sieve				Sedimentation (Hydrometer)			
ASTM D6913				ASTM D7928		ASTM D4221	
Sieve Designation		Percent Passing	Gravel / Sand	Particle Size	Percent Passing	Particle Size	Percent Passing
-	mm		Fines	mm		mm	
3 in.	76.2	100.0	0.0	--	--	--	--
2 in.	50.8	100.0		--	--	--	--
1.5 in.	38.1	100.0		--	--	--	--
1 in.	25.4	100.0		--	--	--	--
3/4 in.	19.0	100.0		--	--	--	--
1/2 in.	12.7	100.0		--	--	--	--
3/8 in.	9.51	100.0		--	--	--	--
No. 4	4.76	100.0	65.9	**Hydrometer Log-Linear Interpolation**			
No. 10	2.00	99.2		Particle Size	Percent Passing	Particle Size	Percent Passing
No. 20	0.841	98.7					
No. 40	0.420	98.5		mm		mm	
No. 60	0.250	98.0		0.002	--	0.002	--
No. 140	0.106	53.4		N m,2μm,d	-	N m,2μm,nd	-
No. 200	0.074	34.1	34.1	Percent Dispersion		-	

Atterberg Limits	
ASTM D4318, Method A : Multipoint, Air Dried	
Liquid Limit	--
Plastic Limit	--
Plastic Index	--
(NL = No Liquid Limit, NP = No Plastic Limit)	

Particle Size Log-Linear Interpolation					
D_x	85	60	50	30	10
mm	1.9E-01	1.2E-01	1.0E-01	--	--

Cu	--	Cc	--

USCS Classification (ASTM D2487)
--

Moisture Content (%)	ASTM D2216	9.4
Organic Content (%)	ASTM D2974-C	--
Carbonate Content (%)	ASTM D4373	--
Specific Gravity	ASTM D854	--

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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

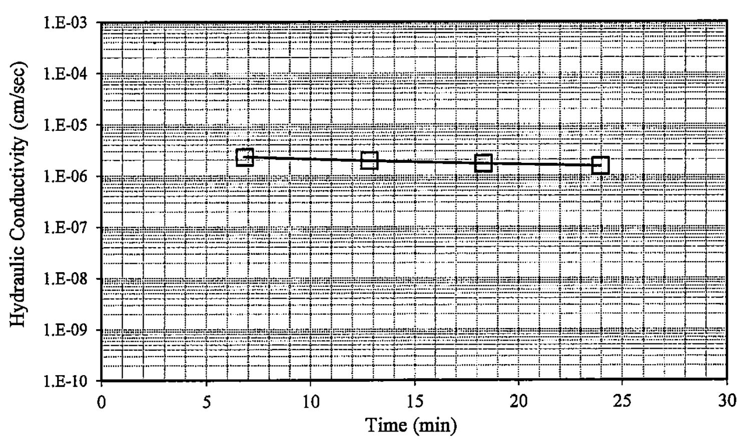
Sample ID: PWCG-01A (55.5-60)

TRI Log #: 23-003876.1

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	1.46	1.45
Height (in)	3.51	3.50
Mass (g)	180.9	186.2
Sample Area (in^2)	1.67	1.66
Water Content (%)	22.8	31.7
Total Unit Weight (pcf)	117.4	122.2
Dry Unit Weight (pcf)	95.6	92.8
Specific Gravity (Assumed)	2.75	
Degree of Saturation	78.8	102.6
Void Ratio	0.80	0.85
Porosity	0.44	0.46
1 Pore Volume (cc)	42.6	43.7

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.96
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
6.8	22.4	31.5	2.3E-06
12.8	13.5	19.0	2.0E-06
18.3	9.4	13.2	1.7E-06
24.0	7.0	9.8	1.5E-06
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**1.6E-06**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

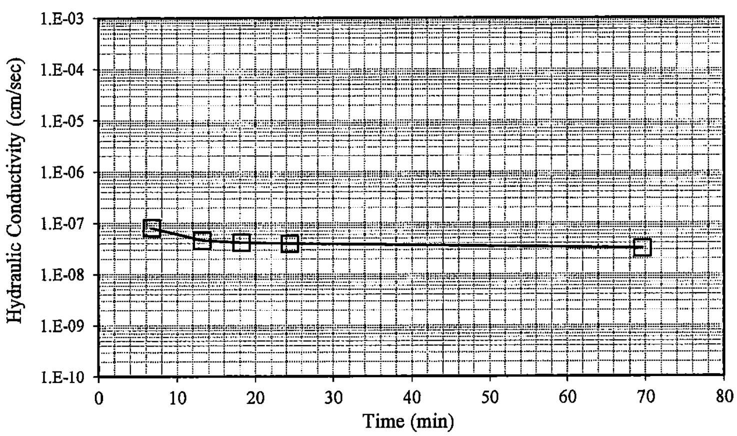
Sample ID: PWCG-01A (295-300)

TRI Log #: 23-003876.5

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.21	4.24
Height (in)	2.91	2.97
Mass (g)	1353.2	1401.0
Sample Area (in^2)	13.95	14.14
Water Content (%)	16.7	22.5
Total Unit Weight (pcf)	127.0	127.2
Dry Unit Weight (pcf)	108.8	103.8
Specific Gravity (Assumed)	2.75	
Degree of Saturation	79.6	94.9
Void Ratio	0.58	0.65
Porosity	0.37	0.40
1 Pore Volume (cc)	243.3	271.7

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.95
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
6.7	19.2	32.5	8.0E-08
13.2	16.1	27.4	4.6E-08
18.2	14.7	24.9	4.1E-08
24.4	13.7	23.3	4.0E-08
69.6	12.7	21.6	3.2E-08
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**3.6E-08**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-106 Royal Oaks Landfill

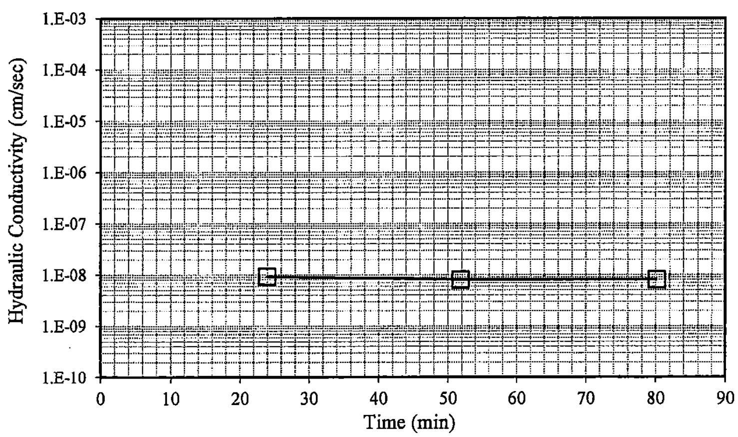
Sample ID: PWCG-03 (232-234)

TRI Log #: 23-003892.5

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	2.08	2.10
Height (in)	3.23	3.27
Mass (g)	375.8	385.1
Sample Area (in^2)	3.39	3.46
Water Content (%)	14.7	22.4
Total Unit Weight (pcf)	130.8	129.3
Dry Unit Weight (pcf)	114.0	105.7
Specific Gravity (Assumed)	2.75	
Degree of Saturation	80.2	98.7
Void Ratio	0.51	0.62
Porosity	0.34	0.38
1 Pore Volume (cc)	60.2	71.4

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.98
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
23.9	20.7	31.7	9.3E-09
51.8	20.4	31.3	8.1E-09
80.1	20.1	30.8	8.1E-09
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**8.1E-09**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group TRI Log #: 23-001381-1

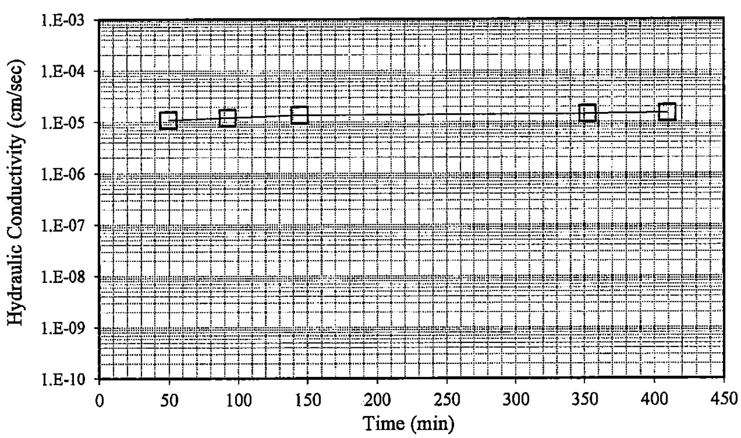
Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-3 (230'-231')

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	1.43	1.43
Height (in)	2.69	2.60
Mass (g)	137.4	137.1
Sample Area (in^2)	1.61	1.60
Water Content (%)	27.9	28.1
Total Unit Weight (pcf)	121.1	125.1
Dry Unit Weight (pcf)	94.6	97.6
Specific Gravity (Assumed)	2.75	
Degree of Saturation	94.4	102.0
Void Ratio	0.81	0.76
Porosity	0.45	0.43
1 Pore Volume (cc)	31.8	29.5

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.99
Permeant	De-Aired Tap Water

Method C—Falling Head, rising tailwater elevation				
Time, t	Initial Gradient	Final Gradient	Inflow / Outflow	K_{20}
Min	-	-	-	cm/s
49.4	3.2	3.1	0.83	1.1E-05
92.0	3.1	2.9	1.00	1.2E-05
144.3	2.9	2.7	1.17	1.4E-05
352.3	2.7	2.0	1.00	1.4E-05
409.3	2.0	1.9	0.83	1.5E-05
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
Average, Last 4 Readings				**1.4E-05**



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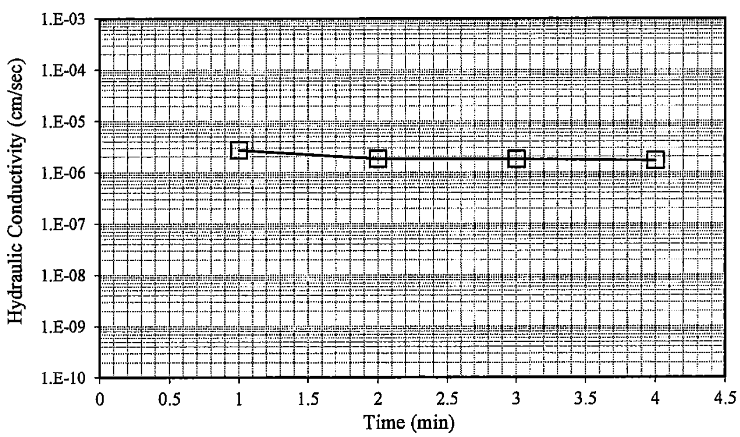
Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group
TRI Log #: 23-003892.8
Project: 0120-076-11-106 Royal Oaks Landfill
Sample ID: PWCG-04 (30-35)

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	2.75	2.70
Height (in)	2.89	2.94
Mass (g)	509.8	526.0
Sample Area (in^2)	5.93	5.72
Water Content (%)	22.7	36.0
Total Unit Weight (pcf)	113.3	119.1
Dry Unit Weight (pcf)	92.3	87.6
Specific Gravity (Assumed)	2.75	
Degree of Saturation	72.6	103.3
Void Ratio	0.86	0.96
Porosity	0.46	0.49
1 Pore Volume (cc)	129.7	134.9

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.96
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
1.0	23.9	40.9	2.7E-06
2.0	16.7	28.5	1.9E-06
3.0	13.0	22.2	1.8E-06
4.0	10.2	17.4	1.7E-06
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**1.8E-06**



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Hydraulic Conductivity (ASTM D5084)

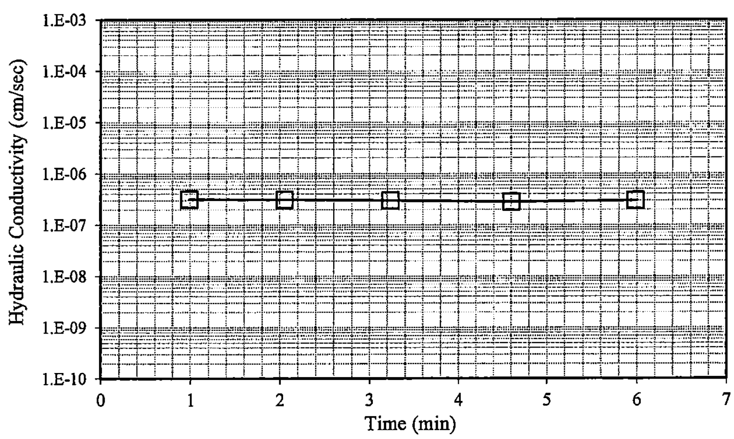
Client: Weaver Consultants Group
Project: 0120-076-11-106 Royal Oaks Landfill
Sample ID: PWCG-05 (30-35)

TRI Log #: 23-003895.1

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.16	4.20
Height (in)	4.35	4.44
Mass (g)	1913.2	1980.0
Sample Area (in^2)	13.61	13.88
Water Content (%)	22.5	42.0
Total Unit Weight (pcf)	123.0	122.3
Dry Unit Weight (pcf)	100.4	86.1
Specific Gravity (Assumed)	2.75	
Degree of Saturation	87.2	116.4
Void Ratio	0.71	0.99
Porosity	0.41	0.50
1 Pore Volume (cc)	402.4	503.4

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	1.00
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
1.0	16.7	18.9	3.2E-07
2.0	15.6	17.7	3.1E-07
3.2	14.6	16.5	3.0E-07
4.6	13.5	15.4	2.8E-07
6.0	12.5	14.2	3.0E-07
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**2.9E-07**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

TRI Log #: 23-003895.5

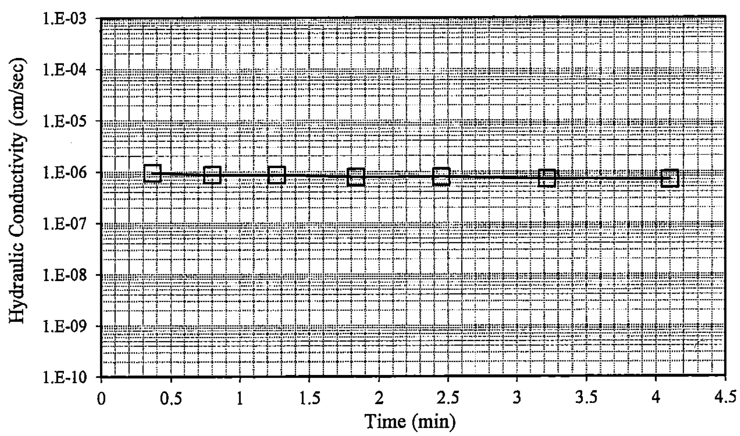
Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-06 (35-40)

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.17	4.20
Height (in)	4.23	4.25
Mass (g)	1902.0	1936.5
Sample Area (in^2)	13.66	13.82
Water Content (%)	18.8	21.3
Total Unit Weight (pcf)	125.3	125.5
Dry Unit Weight (pcf)	105.5	103.4
Specific Gravity (Assumed)	2.75	
Degree of Saturation	82.4	88.9
Void Ratio	0.63	0.66
Porosity	0.39	0.40
1 Pore Volume (cc)	364.7	382.7

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.99
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
0.4	14.6	17.0	9.4E-07
0.8	13.5	15.8	8.6E-07
1.3	12.5	14.6	8.7E-07
1.8	11.5	13.4	7.9E-07
2.5	10.4	12.2	8.0E-07
3.2	9.4	10.9	7.2E-07
4.1	8.3	9.7	7.1E-07
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**7.1E-07**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

TRI Log #: 23-003895-6

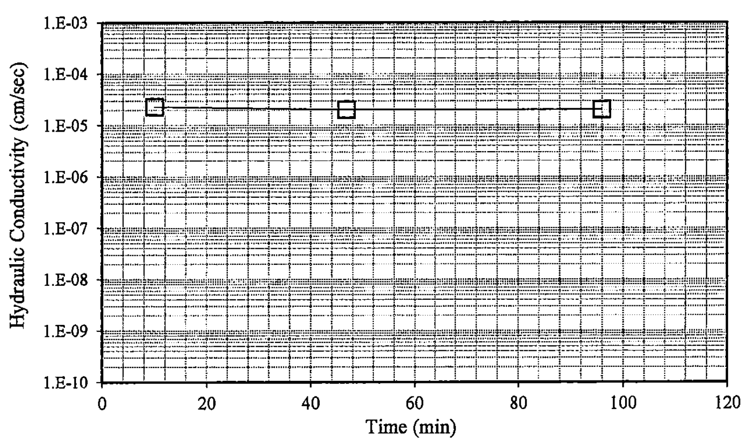
Project: 0120-076-11-106 Royal Oaks Landfill

Sample ID: PWCG-06 (75-85)

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.03	4.10
Height (in)	3.69	3.72
Mass (g)	1549.2	1587.4
Sample Area (in^2)	12.76	13.23
Water Content (%)	16.4	21.5
Total Unit Weight (pcf)	125.5	122.9
Dry Unit Weight (pcf)	107.7	101.1
Specific Gravity (Assumed)	2.75	
Degree of Saturation	76.3	84.8
Void Ratio	0.59	0.70
Porosity	0.37	0.41
1 Pore Volume (cc)	286.8	331.0

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.98
Permeant	De-Aired Tap Water

Method C—Falling Head, rising tailwater elevation				
Time, t	Initial Gradient	Final Gradient	Inflow / Outflow	K_{20}
Min	-	-	-	cm/s
10.0	2.5	2.2	1.08	2.3E-05
46.8	2.2	1.4	1.06	2.0E-05
96.0	1.4	0.8	1.11	2.0E-05
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
Average, Last 4 Readings				**2.2E-05**



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Hydraulic Conductivity (ASTM D5084)

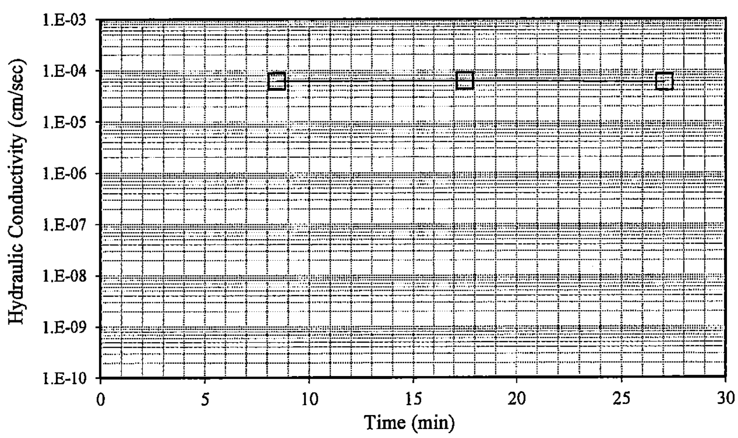
Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: PWCG-07 (99.5-104.5)

TRI Log #: 23-003901.3

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.38	4.22
Height (in)	1.95	2.01
Mass (g)	953.8	935.6
Sample Area (in^2)	15.06	14.00
Water Content (%)	23.2	21.5
Total Unit Weight (pcf)	124.0	126.4
Dry Unit Weight (pcf)	100.7	104.0
Specific Gravity (Assumed)	2.75	
Degree of Saturation	90.3	91.1
Void Ratio	0.70	0.65
Porosity	0.41	0.39
1 Pore Volume (cc)	198.5	182.1

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	1.10
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
-	0.0	0.0	-
-	0.0	0.0	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 4 Readings			**6.1E-05**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

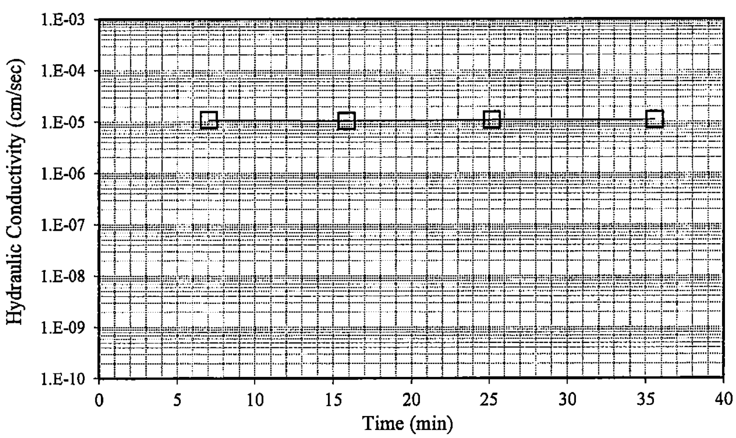
Sample ID: PWCG-07 (239.5-244.5)

TRI Log #: 23-003901.5

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.27	4.13
Height (in)	1.99	2.02
Mass (g)	910.2	893.2
Sample Area (in^2)	14.35	13.41
Water Content (%)	22.4	19.7
Total Unit Weight (pcf)	121.3	125.6
Dry Unit Weight (pcf)	99.1	104.9
Specific Gravity (Assumed)	2.75	
Degree of Saturation	84.2	85.2
Void Ratio	0.73	0.64
Porosity	0.42	0.39
1 Pore Volume (cc)	197.8	172.5

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.90
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
-	0.0	0.0	-
-	0.0	0.0	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 4 Readings			**1.1E-05**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

TRI Log #: 23-003901.6

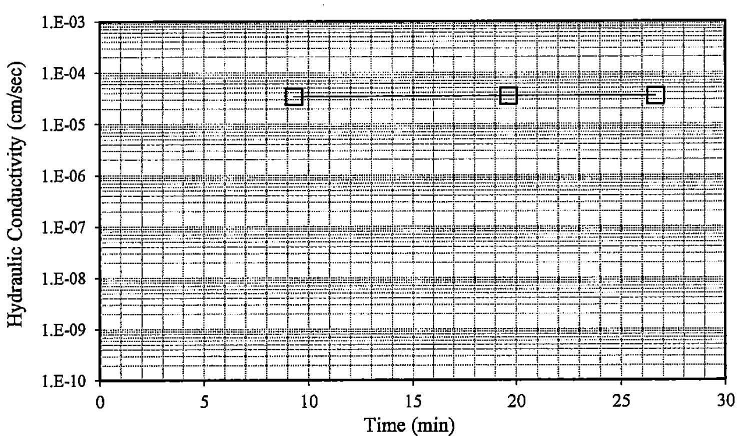
Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: PWCG-07 (354.5-359.5)

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.15	4.22
Height (in)	1.91	1.98
Mass (g)	815.4	857.2
Sample Area (in^2)	13.54	13.96
Water Content (%)	18.4	20.4
Total Unit Weight (pcf)	119.8	118.1
Dry Unit Weight (pcf)	101.2	98.1
Specific Gravity (Assumed)	2.75	
Degree of Saturation	72.7	75.0
Void Ratio	0.70	0.75
Porosity	0.41	0.43
1 Pore Volume (cc)	174.2	193.9

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	1.06
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
-	0.0	0.0	-
-	0.0	0.0	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 4 Readings			**3.5E-05**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

TRI Log #: 23-004746.1

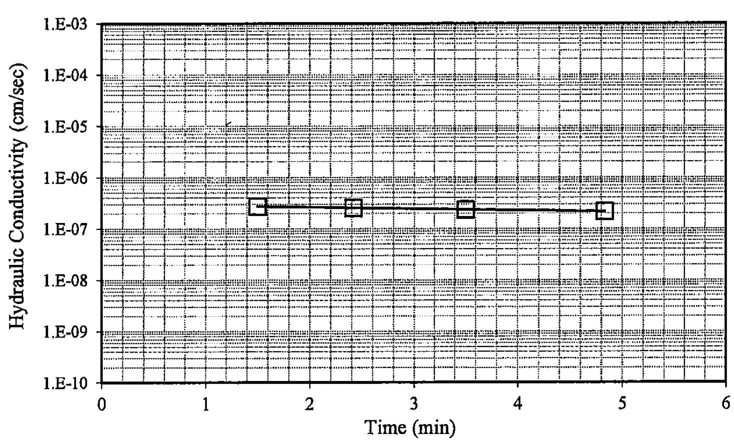
Project: 0120-076-11-01 Royal Oaks Landfill

Sample ID: PWCG-7 (359.5-360.5')

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.17	4.27
Height (in)	2.23	2.34
Mass (g)	1015.0	1086.3
Sample Area (in^2)	13.67	14.30
Water Content (%)	15.7	21.2
Total Unit Weight (pcf)	126.7	123.5
Dry Unit Weight (pcf)	109.5	101.9
Specific Gravity (Assumed)	2.75	
Degree of Saturation	75.9	85.3
Void Ratio	0.57	0.68
Porosity	0.36	0.41
1 Pore Volume (cc)	180.9	222.8

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.99
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
1.5	13.5	30.0	2.7E-07
2.4	11.5	25.3	2.5E-07
3.5	10.4	23.0	2.4E-07
4.8	9.4	20.7	2.2E-07
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**2.3E-07**



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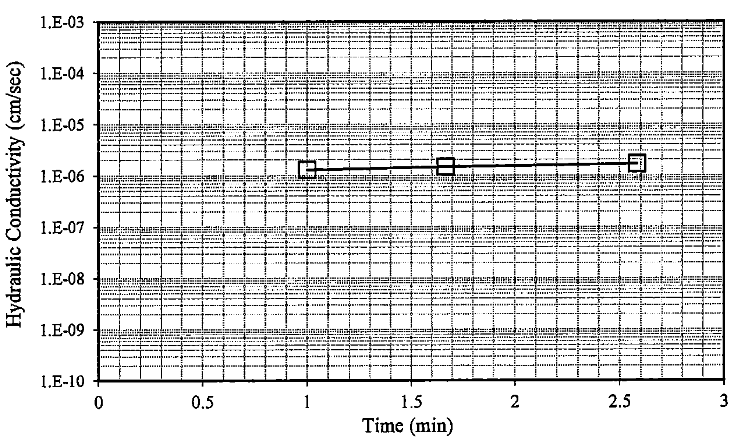
Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group
TRI Log #: 23-004746.2
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: PWCG-7 (360.5-362)

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.16	4.26
Height (in)	2.51	2.68
Mass (g)	1151.0	1236.8
Sample Area (in^2)	13.59	14.23
Water Content (%)	17.9	28.4
Total Unit Weight (pcf)	128.3	123.4
Dry Unit Weight (pcf)	108.8	96.1
Specific Gravity (Assumed)	2.75	
Degree of Saturation	85.3	99.5
Void Ratio	0.58	0.79
Porosity	0.37	0.44
1 Pore Volume (cc)	205.0	275.0

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.97
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
1.0	16.7	32.7	1.3E-06
1.7	10.4	20.5	1.5E-06
2.6	7.3	14.3	1.7E-06
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**1.6E-06**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

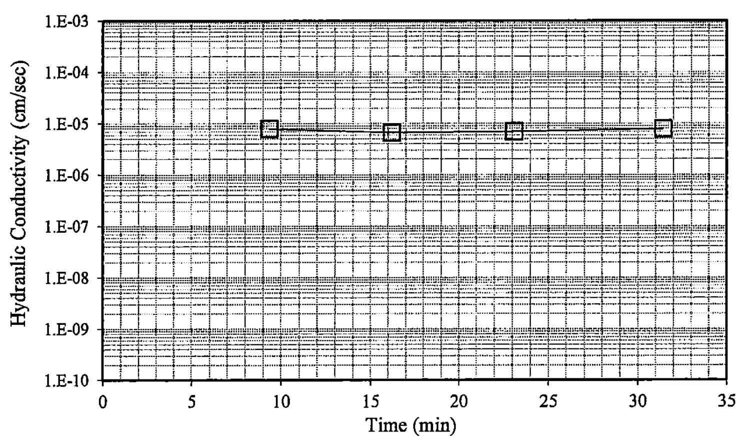
Sample ID: PWCG-08 (187-199.5)

TRI Log #: 23-003901.7

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.35	4.21
Height (in)	2.21	2.29
Mass (g)	1159.1	1120.9
Sample Area (in^2)	14.87	13.94
Water Content (%)	22.5	17.6
Total Unit Weight (pcf)	134.3	133.5
Dry Unit Weight (pcf)	109.7	113.5
Specific Gravity (Assumed)	2.75	
Degree of Saturation	109.5	94.8
Void Ratio	0.56	0.51
Porosity	0.36	0.34
1 Pore Volume (cc)	194.2	177.3

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.98
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
-	0.0	0.0	-
-	0.0	0.0	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 4 Readings			**7.3E-06**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

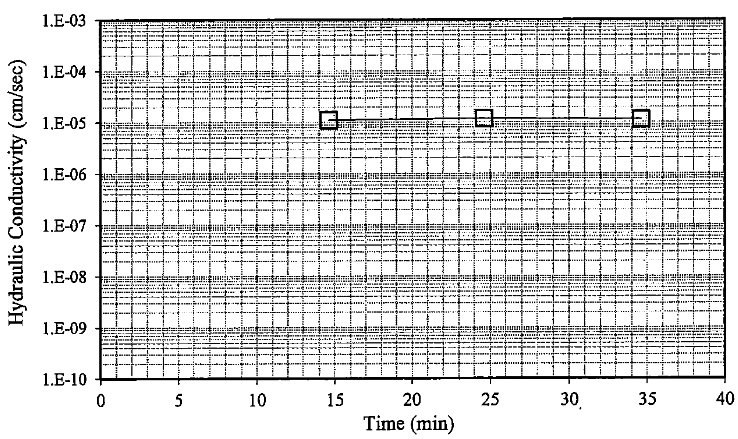
Sample ID: PWCG-09 (125-130)

TRI Log #: 23-003901.8

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	3.98	4.01
Height (in)	1.92	1.98
Mass (g)	769.0	790.8
Sample Area (in^2)	12.44	12.65
Water Content (%)	24.2	29.4
Total Unit Weight (pcf)	122.7	120.3
Dry Unit Weight (pcf)	98.8	93.0
Specific Gravity (Assumed)	2.75	
Degree of Saturation	90.3	95.7
Void Ratio	0.74	0.85
Porosity	0.42	0.46
1 Pore Volume (cc)	165.9	187.8

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.99
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
-	0.0	0.0	-
-	0.0	0.0	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 4 Readings			**7.6E-07**



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Hydraulic Conductivity (ASTM D5084)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

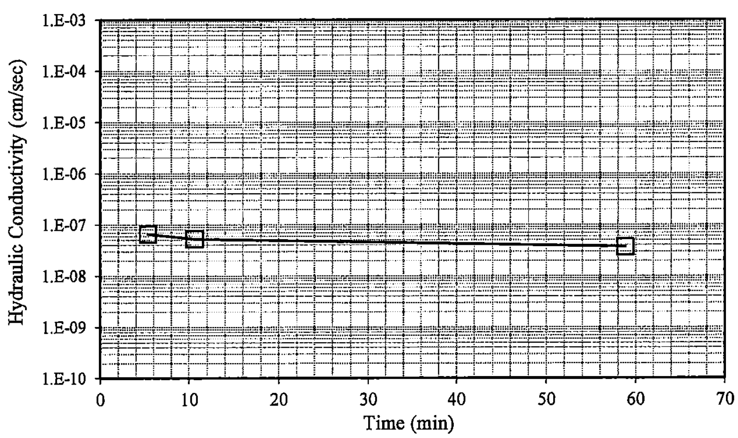
Sample ID: WCG-10 (92-99.5)

TRI Log #: 23-003912.4

Sample Condition	Initial Intact	Final Post-Test
Diameter (in)	4.13	4.20
Height (in)	3.10	3.16
Mass (g)	1304.7	1323.1
Sample Area (in^2)	13.41	13.85
Water Content (%)	31.7	32.6
Total Unit Weight (pcf)	119.6	115.1
Dry Unit Weight (pcf)	90.8	86.8
Specific Gravity (Assumed)	2.75	
Degree of Saturation	98.0	91.8
Void Ratio	0.89	0.98
Porosity	0.47	0.49
1 Pore Volume (cc)	320.8	354.3

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	0.98
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
5.4	15.2	24.2	6.5E-08
10.6	13.7	21.9	5.3E-08
58.8	12.7	20.3	3.7E-08
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**4.5E-08**



Kelby Broussard 10/20/2023

Analysis & Quality Review/Date

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Hydraulic Conductivity (ASTM D5084)

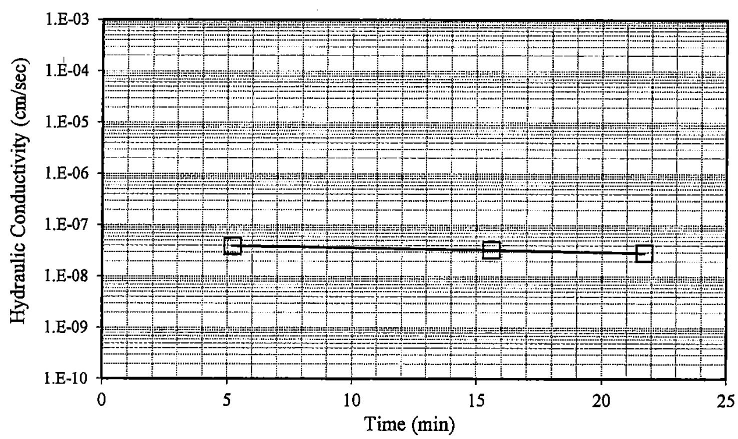
Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample ID: WCG-11 (17-27)

TRI Log #: 23-003912.7

Sample Condition	Initial	Final
	Intact	Post-Test
Diameter (in)	4.88	3.91
Height (in)	3.00	2.99
Mass (g)	1137.2	1172.3
Sample Area (in^2)	18.68	12.02
Water Content (%)	23.6	24.3
Total Unit Weight (pcf)	77.3	124.2
Dry Unit Weight (pcf)	62.6	99.9
Specific Gravity (Assumed)	2.75	
Degree of Saturation	37.2	93.1
Void Ratio	1.74	0.72
Porosity	0.64	0.42
1 Pore Volume (cc)	582.8	246.1

Eff. Confining Stress (psi)	5.0
Back-Pressure	80.0
B-Value Prior to Permeation	1.00
Permeant	De-Aired Tap Water

Method F—Constant Volume–Falling Head by mercury, rising tailwater elevation			
Manometer Constants		Aa (cm^2)	0.767
M1	0.0302	Ap (cm^2)	0.0314
M2	1.041	Z_p (cm)	0
Time, t	Trial Constant, Z_1	Gradient	K_{20}
Min	-	-	cm/s
5.2	13.0	21.5	3.9E-08
15.6	12.0	19.7	3.3E-08
21.7	10.4	17.2	2.9E-08
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
Average, Last 2 Readings			**3.1E-08**



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Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

M_c = 23.4 %200 = 99.4

LL= 57 PL= 21 PI= 36

TRI Log #: 23-003876.2
Test Method: ASTM D4767

Specimens				
Identification	1	2	3	4
Depth/Elev. (ft)	-	-	-	-
Eff. Consol. Stress (psi)	24.3	48.6	97.2	-
Initial Specimen Properties				
Avg. Diameter (in)	1.47	1.49	1.41	-
Avg. Height (in)	3.52	3.13	3.09	-
Avg. Water Content (%)	26.6	24.7	22.6	-
Bulk Density (pcf)	122.9	119.6	119.1	-
Dry Density (pcf)	97.1	95.8	97.1	-
Specific Gravity (Assumed)	2.75			
Saturation (%)	95.3	86.1	81.1	-
Void Ratio, n	0.77	0.79	0.77	-
B-Value, End of Saturation	0.98	0.96	0.98	-

Test Setup	
Specimen Condition	Undisturbed / Intact
Specimen Preparation	Trimmed
Mounting Method	Wet
Consolidation	Isotropic

Post-Consolidation / Pre-Shear				
Void Ratio	0.74	0.78	0.67	-

Shear / Post-Shear				
Rate of Strain (%/hr)	0.25	0.25	0.25	-
Avg. Water Content (%)	27.1	23.5	25.0	-

At Failure								
Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$				Ratio, $(\sigma_1'/\sigma_3')_{max}$			
Axial Strain at Failure (%), $\varepsilon_{a,f}$	9.9	6.6	5.0	-	4.2	4.8	5.0	-
Minor Effective Stress (psi), $\sigma_3'_f$	23.6	33.4	59.3	-	18.7	29.6	59.3	-
Principal Stress Difference (psi), $(\sigma_1-\sigma_3)_f$	39.7	77.4	114.8	-	36.4	71.6	114.8	-
Pore Water Pressure, Δu_f (psi)	-0.2	15.2	37.9	-	5.5	19.1	37.9	-
Major Effective Stress (psi), $\sigma_1'_f$	63.3	110.7	174.1	-	55.1	101.2	174.1	-
Secant Friction Angle (degrees)	27.1	32.5	29.4	-	29.5	33.2	29.4	-
Effective Friction Angle (degrees)	29.7				28.5			
Effective Cohesion (psi)	0.3				2.6			

Note: The presented M-C parameters are based on a linear regression in modified stress space, across all assigned effective consolidation stresses. This fit does not purported to capture typical curvature of envelopes that may, in particular, be observed across broader range in effective stresses. Please note that the stresses associated with peak principal stress ratio and peak principal stress difference are presented in tabular form on the first page of the report. There are alternate interpretations to theses two failure criterion including but not limited to strain compatibility and post-peak.

43.2 psf

	EFF.	TOTAL
φ	29.7	19.4
c	43.2	1066

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Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

TRI Log #: 23-003876.2
Test Method: ASTM D4767

R / "Total Stress" Envelope			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Friction Angle (deg)	ϕ_R	19.4	20.1
Cohesion (psi)	c_R	7.4	5.2

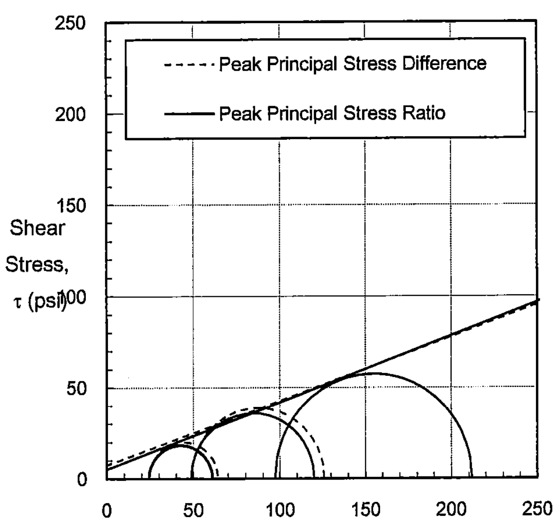
1066 psf

Kc = Kf Envelope, Effective Stress Envelope (Duncan et al. 1990)			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	ϕ'	29.7	28.5
Effective Cohesion (psi)	c'	0.3	2.6

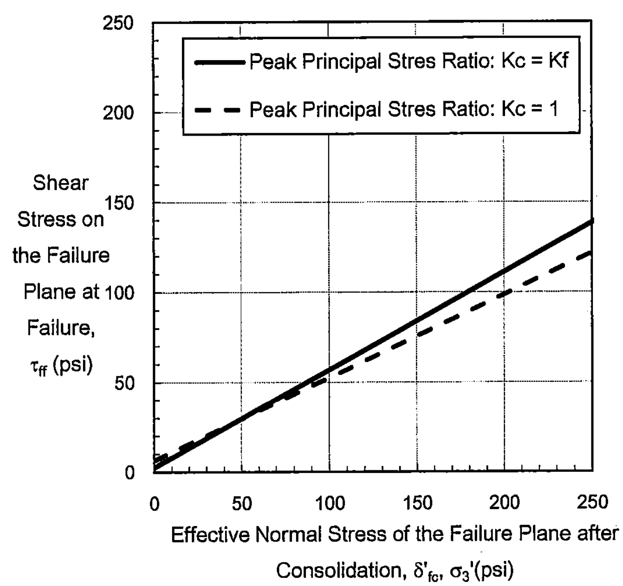
Kc = 1 (τ_{ff} vs σ'_{fc}) Envelope, Total Stress Envelope (Duncan et al. 1990)			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Friction Angle (deg)	$d_{Kc=1}$	23.3	24.7
Cohesion (psi)	$\psi_{Kc=1}$	9.1	6.5

1310 psf

R / "Total Stress" Envelope



Three-Stage Rapid Drawdown Envelopes



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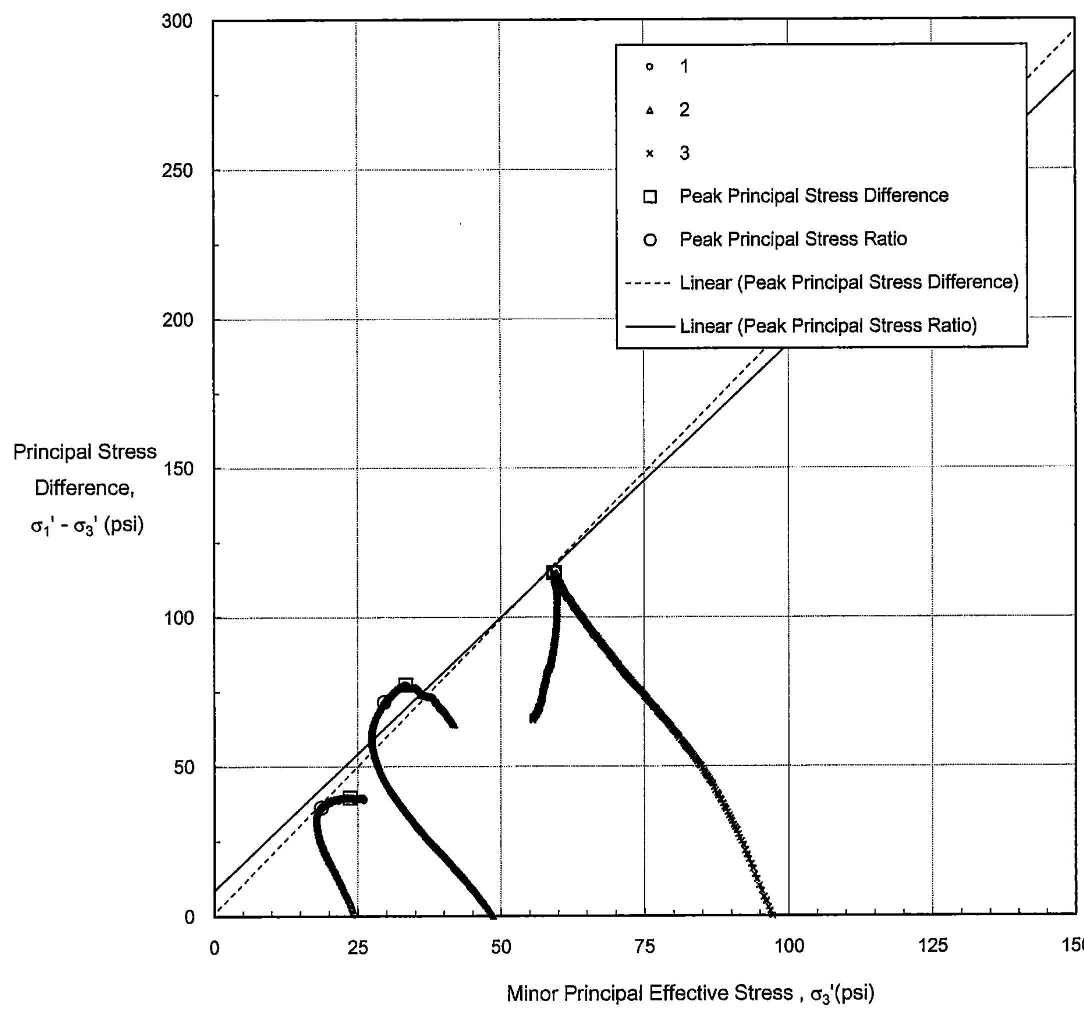
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

TRI Log #: 23-003876.2
Test Method: ASTM D4767

Modified Mohr-Coulomb



Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	29.7	28.5
Effective Cohesion (psi)	0.3	2.6

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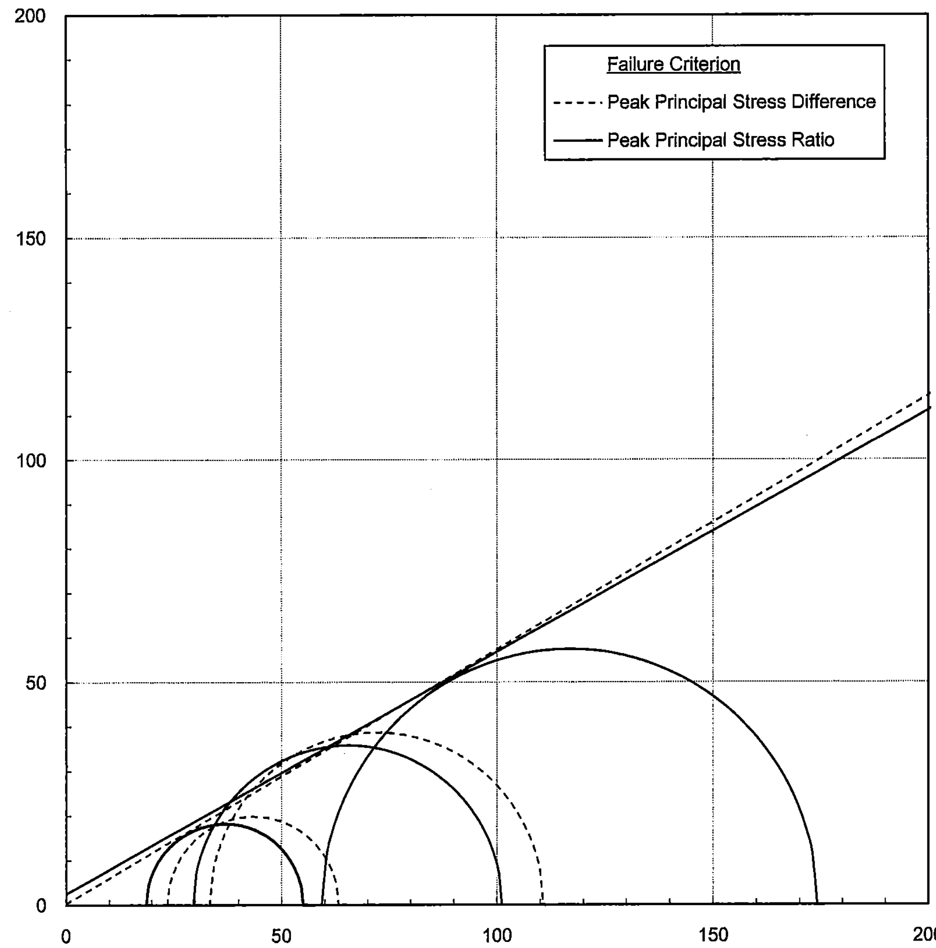
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

TRI Log #: 23-003876.2
Test Method: ASTM D4767

Mohr-Coulomb



Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	29.7	28.5
Effective Cohesion (psi)	0.3	2.6

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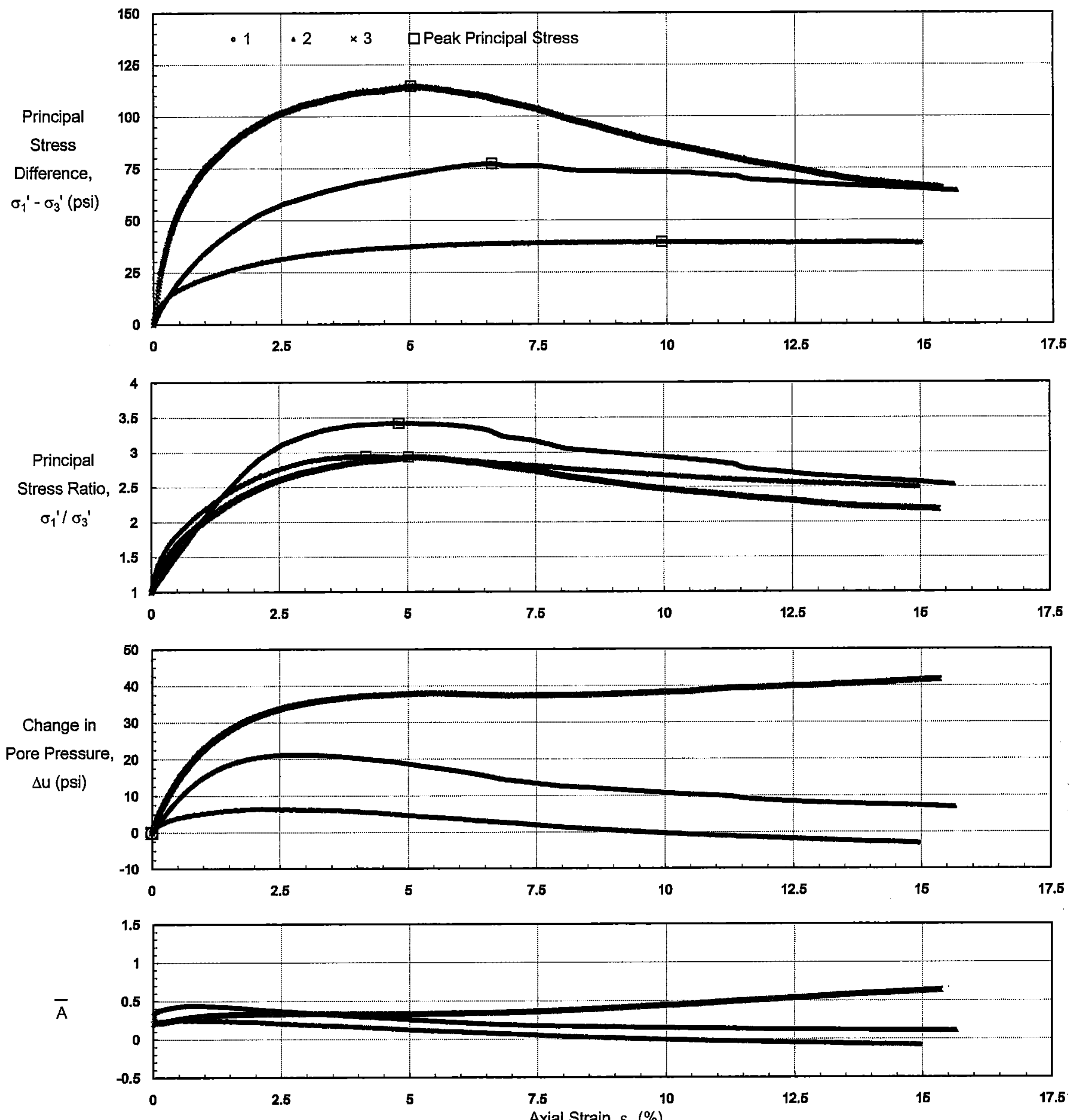
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

TRI Log #: 23-003876.2
Test Method: ASTM D4767



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Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

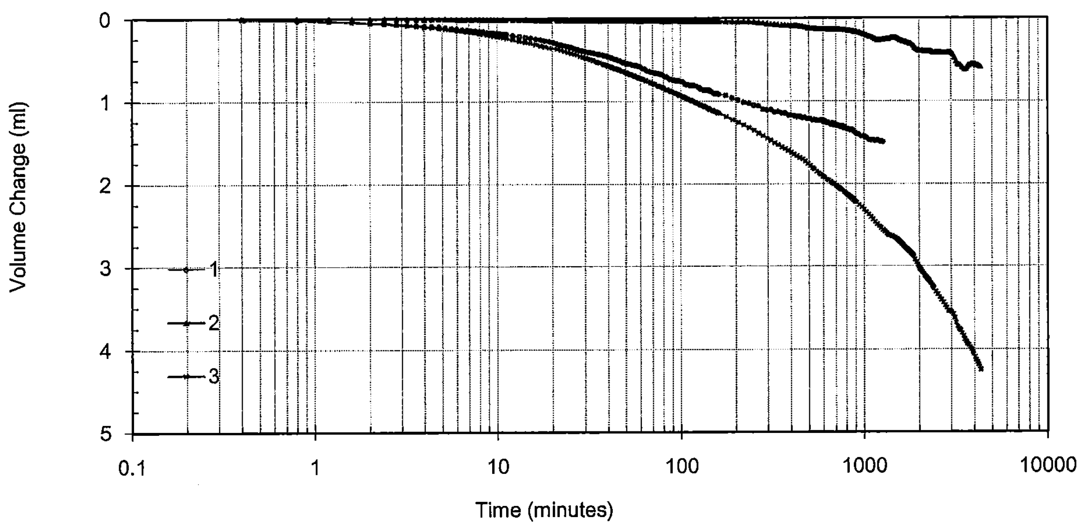
TRI Log #: 23-003876.2
Test Method: ASTM D4767

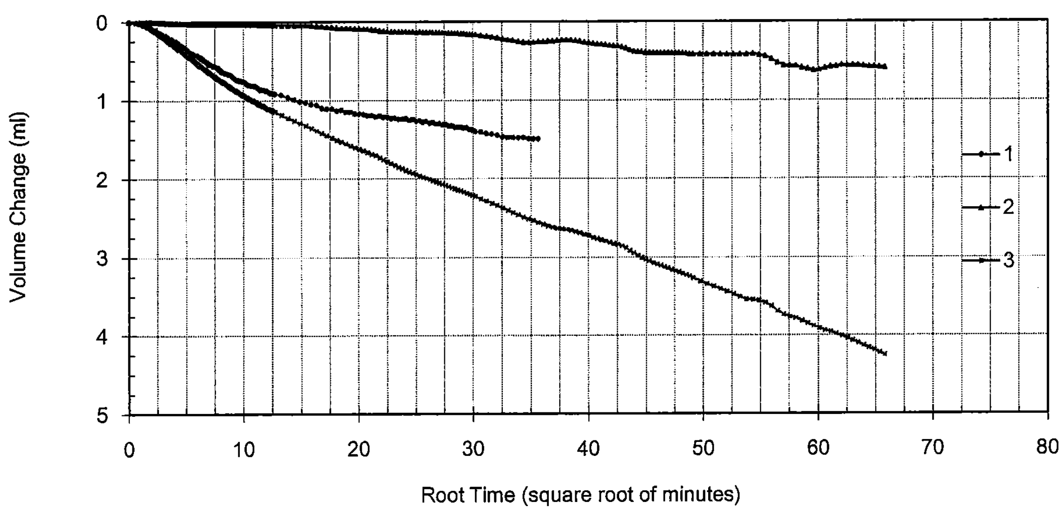
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: PWCG-01A (60-69)

TRI Log #: 23-003876.2
Test Method: ASTM D4767

Consolidation





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Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767

Specimens				
Identification	1	2	3	4
Depth/Elev. (ft)	-	-	-	-
Eff. Consol. Stress (psi)	24.3	48.6	97.2	-
Initial Specimen Properties				
Avg. Diameter (in)	1.40	1.44	1.42	-
Avg. Height (in)	3.45	3.39	3.54	-
Avg. Water Content (%)	24.0	23.7	23.5	-
Bulk Density (pcf)	125.1	121.7	123.2	-
Dry Density (pcf)	100.9	98.4	99.7	-
Specific Gravity (Assumed)	2.75			
Saturation (%)	94.2	87.7	89.8	-
Void Ratio, n	0.70	0.74	0.72	-
B-Value, End of Saturation	0.95	0.95	0.99	-

Test Setup	
Specimen Condition	Undisturbed / Intact
Specimen Preparation	Trimmed
Mounting Method	Wet
Consolidation	Isotropic

Post-Consolidation / Pre-Shear				
Void Ratio	0.62	0.62	0.60	-

Shear / Post-Shear				
Rate of Strain (%/hr)	0.25	0.25	0.25	-
Avg. Water Content (%)	26.6	25.0	25.2	-

At Failure								
Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$				Ratio, $(\sigma_1'/\sigma_3')_{max}$			
Axial Strain at Failure (%), $\varepsilon_{a,f}$	9.2	5.7	5.7	-	4.0	5.0	4.9	-
Minor Effective Stress (psi), $\sigma_3'{}_f$	19.7	34.9	67.8	-	15.4	33.6	66.1	-
Principal Stress Difference (psi), $(\sigma_1-\sigma_3)_f$	33.7	53.2	81.9	-	31.1	52.2	80.7	-
Pore Water Pressure, Δu_f (psi)	4.6	13.7	29.4	-	8.9	15.0	30.9	-
Major Effective Stress (psi), $\sigma_1'{}_f$	53.4	88.2	149.7	-	46.5	85.8	146.8	-
Secant Friction Angle (degrees)	27.4	25.6	22.1	-	30.2	25.9	22.3	-
Effective Friction Angle (degrees)	19.2				19.0			
Effective Cohesion (psi)	5.8				6.3			

Note: The presented M-C parameters are based on a linear regression in modified stress space, across all assigned effective consolidation stresses. This fit does not purported to capture typical curvature of envelopes that may, in particular, be observed across broader range in effective stresses. Please note that the stresses associated with peak principal stress ratio and peak principal stress difference are presented in tabular form on the first page of the report. There are alternate interpretations to theses two failure criterion including but not limited to strain compatibility and post-peak.

835 psf

	EFF.	TOTAL
ϕ	19.2	14.2
c	835	1080

Jeffrey A. Kuhn, Ph.D., P.E., 11/8/2023
Analysis & Quality Review/Date

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Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767

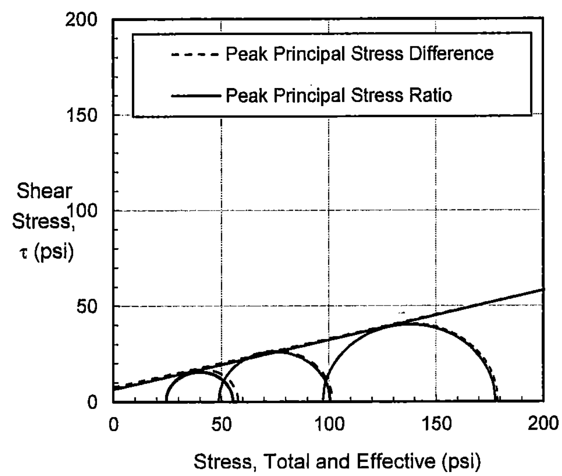
R / "Total Stress" Envelope			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Friction Angle (deg)	ϕ_R	14.2	14.5
Cohesion (psi)	c_R	7.5	6.5

1080 psf

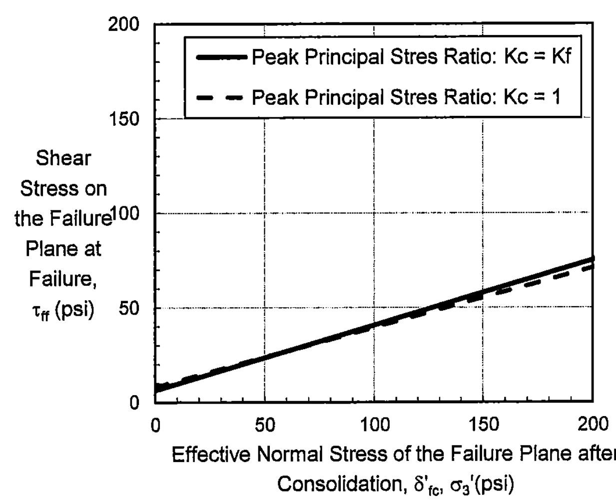
Kc = Kf Envelope, Effective Stress Envelope (Duncan et al. 1990)			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	ϕ'	19.2	19.0
Effective Cohesion (psi)	c'	5.8	6.3

Kc = 1 (τ_{ff} vs σ'_{fc}) Envelope, Total Stress Envelope (Duncan et al. 1990)			
Failure Criterion: Peak Principal Stress		Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Friction Angle (deg)	$d_{Kc=1}$	17.1	17.5
Cohesion (psi)	$\psi_{Kc=1}$	9.1	8.0

R / "Total Stress" Envelope



Three-Stage Rapid Drawdown Envelopes



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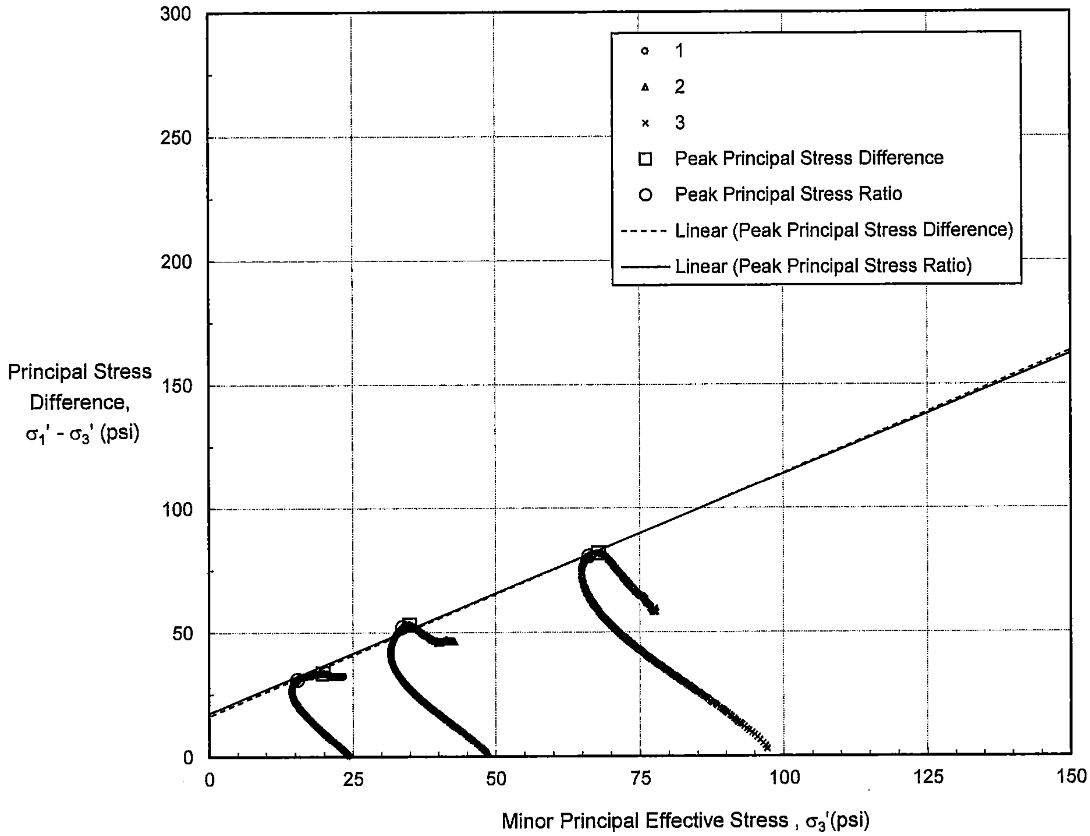
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767

Modified Mohr-Coulomb



Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	19.2	19.0
Effective Cohesion (psi)	5.8	6.3

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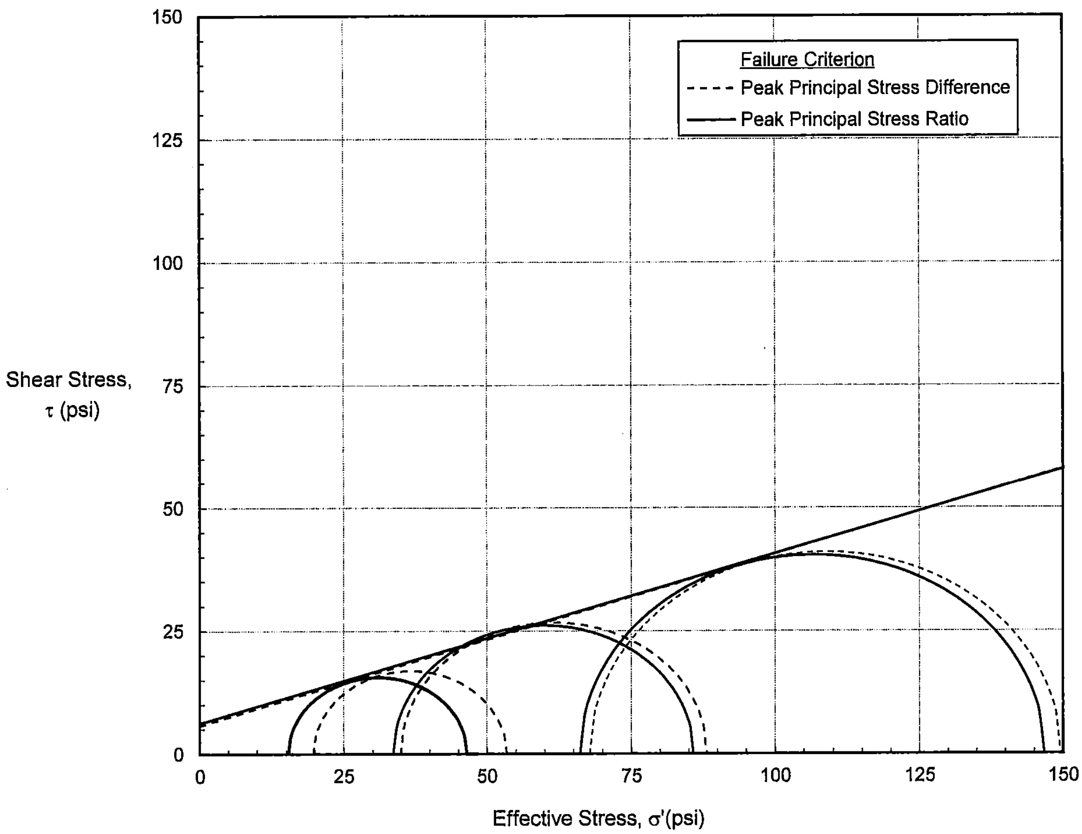
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767

Mohr-Coulomb



Failure Criterion: Peak Principal Stress	Difference, $(\sigma_1'-\sigma_3')_{max}$	Ratio, $(\sigma_1'/\sigma_3')_{max}$
Effective Friction Angle (deg)	19.2	19.0
Effective Cohesion (psi)	5.8	6.3

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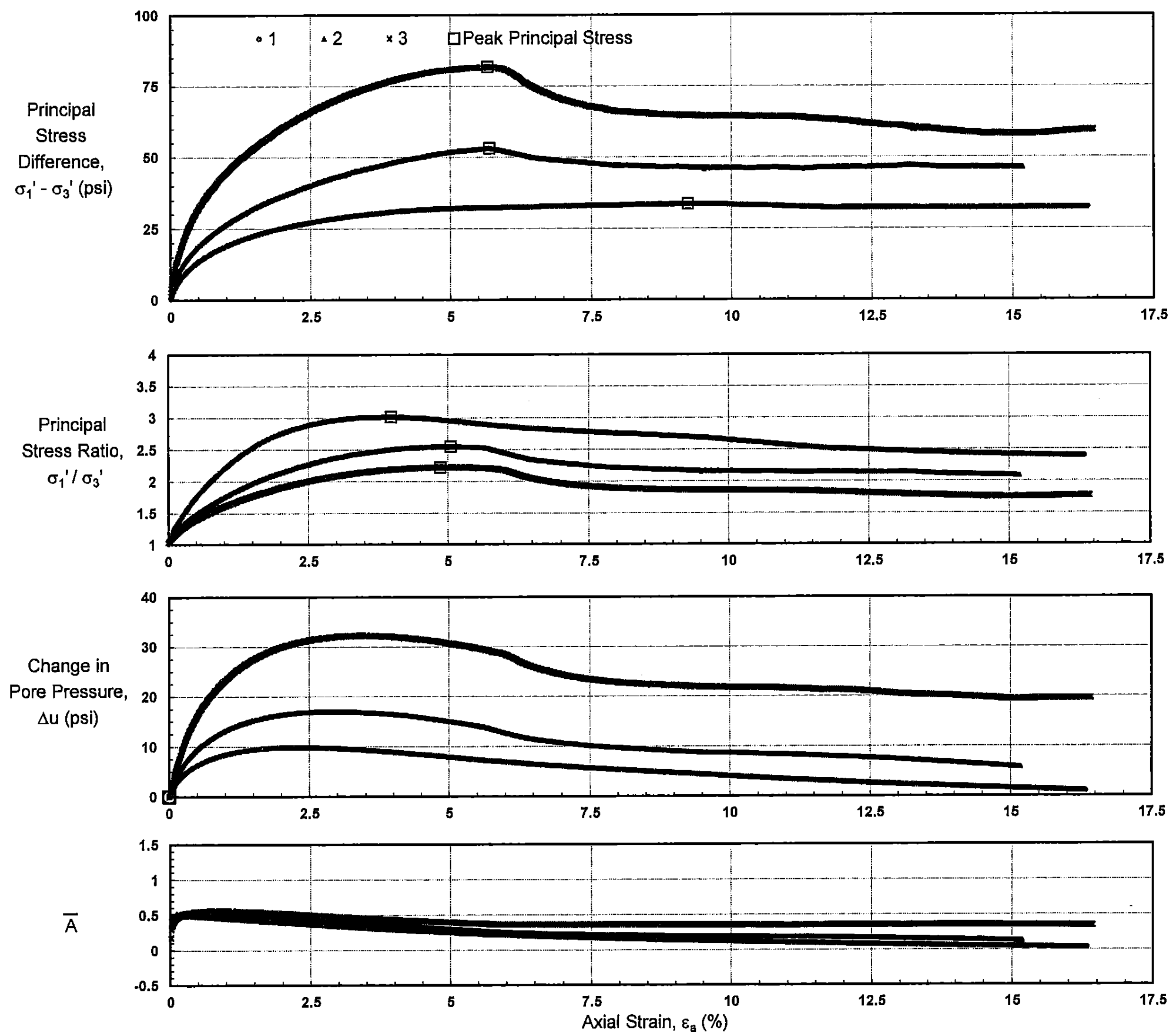
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767



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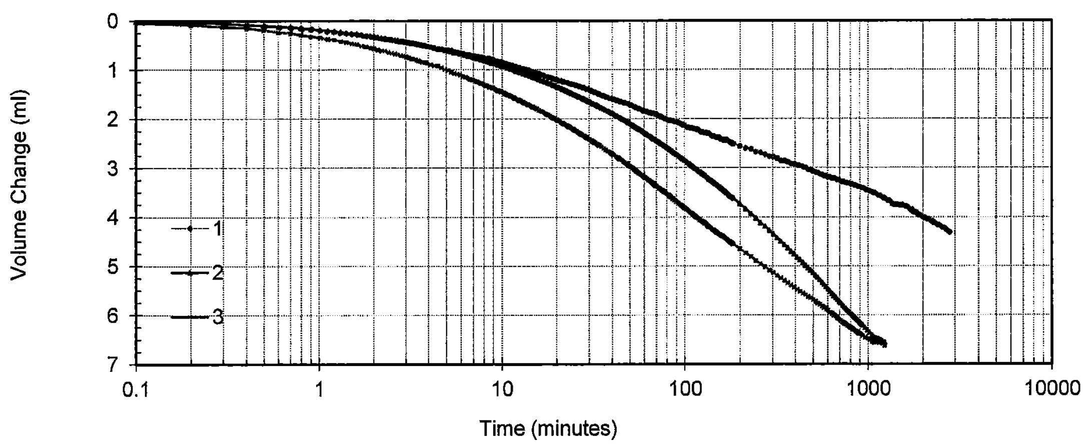
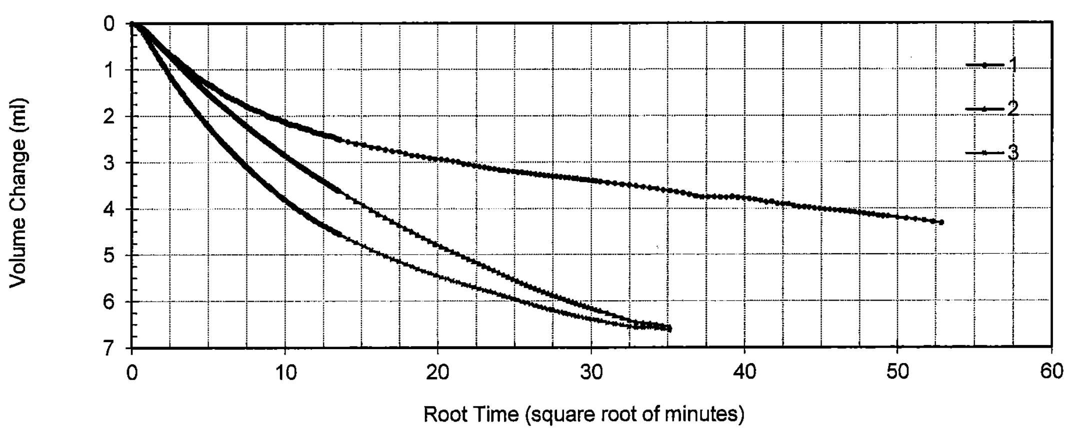
Consolidated-Undrained Triaxial Compression

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Sample: WCG-11 (44.5-57)

TRI Log #: 23-003912.8
Test Method: ASTM D4767

Consolidation





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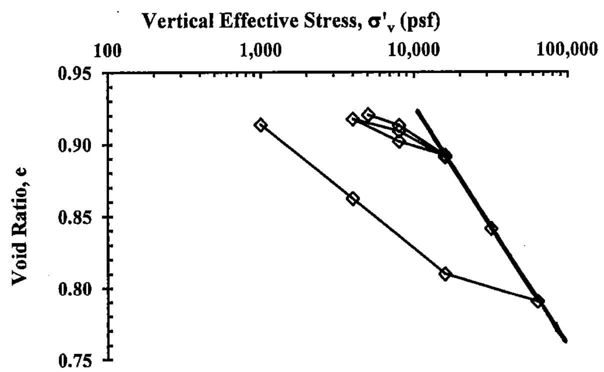
One-Dimensional Consolidation Properties of Soil (ASTM D2435)

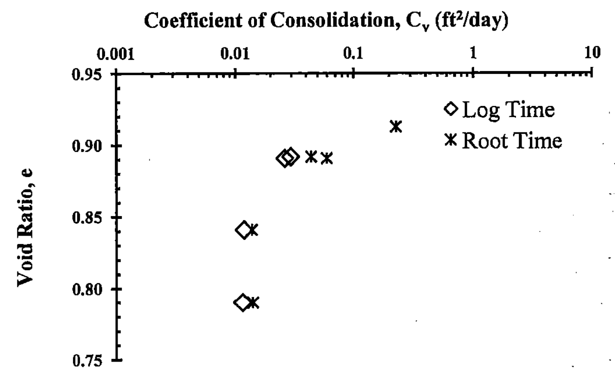
Client: Weaver Consultants Group
Project: 0120-076-11-106 Royal Oaks Landfill
Specimen: PWCG-05 (30-35)

TRI Log No.: 23-003895-01
Jeffrey A. Kuhn, Ph.D., P.E., 11/20/2023
Quality Review/Date

Soil Specimen Properties	Initial	Final
Water Content (%)	36.7	26.6
Diameter (in)	2.49	2.49
Height (in)	1.00	1.00
Dry Unit Weight, γ_o lb_f/ft^3	89.4	89.7
Specific Gravity (Assumed)	2.75	
Void Ratio, e	0.92	0.91
Degree of Saturation (%)	≈100	-

Test Setup		
Specimen Condition	•	Intact / "Undisturbed"
		Compacted/Remolded/Reconstituted
Test Water		Extracted Pore Water
	•	Potable Tap Water
		Demineralized Water
		Saline Water
		Other





Stage	σ'v	log (σ'v)	e	Strain, ε	δe / δlog (σ'v)	C_α	t_{50} (min)		C_v (ft^2/day)	
(#)	(psf)	log (psf)	(-)	(%)		(x1,000)	Log Time	Root Time	Log Time	Root Time
1	5,044	3.70	0.920	0.00	-	-	-	-	-	-
2	8,000	3.90	0.913	0.39	-0.037	-	-	2.1	-	2.3E-01
3	16,000	4.20	0.892	1.48	-0.070	-	16.1	10.9	3.0E-02	4.4E-02
4	8,000	3.90	0.902	0.97	-0.032	-	-	-	-	-
5	4,000	3.60	0.917	0.15	-0.052	-	-	-	-	-
6	8,000	3.90	0.909	0.59	-0.028	-	-	-	-	-
7	16,000	4.20	0.891	1.54	-0.060	-	18.1	8.0	2.6E-02	5.9E-02
8	32,000	4.51	0.841	4.13	-0.166	-	38.3	32.9	1.2E-02	1.4E-02
9	64,000	4.81	0.790	6.79	-0.169	-	37.3	30.5	1.1E-02	1.4E-02
10	16,000	4.20	0.809	5.77	-0.032	-	-	-	-	-
11	4,000	3.60	0.862	3.03	-0.087	-	-	-	-	-
12	1,000	3.00	0.913	0.36	-0.085	-	-	-	-	-
13	-	-	-	-	-	-	-	-	-	-
14	-	-	-	-	-	-	-	-	-	-
15	-	-	-	-	-	-	-	-	-	-
16	-	-	-	-	-	-	-	-	-	-
17	-	-	-	-	-	-	-	-	-	-
18	-	-	-	-	-	-	-	-	-	-

$$C_c = \frac{.841 - .79}{\log\left(\frac{64}{32}\right)} \simeq 0.17$$

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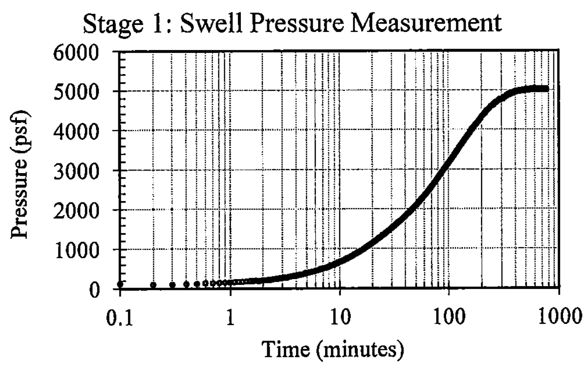
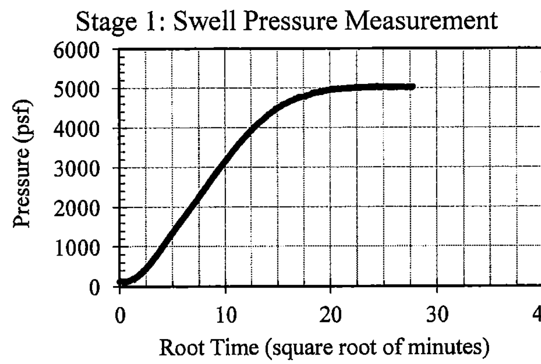
One-Dimensional Consolidation Properties of Soil (ASTM D2435)

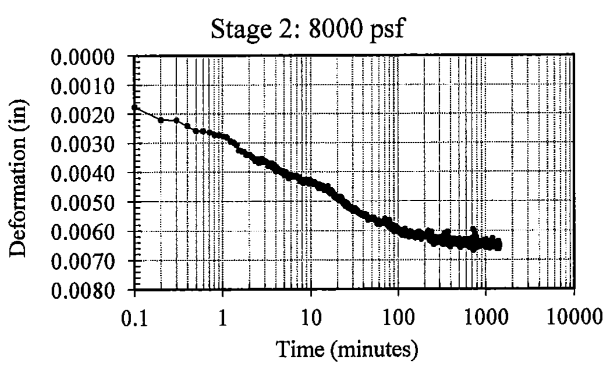
Client: Weaver Consultants Group
Project: 0120-076-11-106 Royal Oaks Landfill
Specimen: PWCG-05 (30-35)

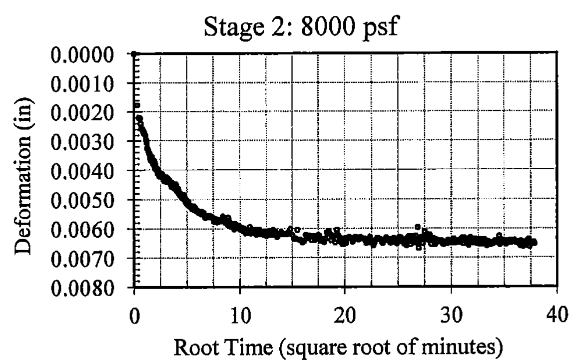
TRI Log No.: 23-003895-01

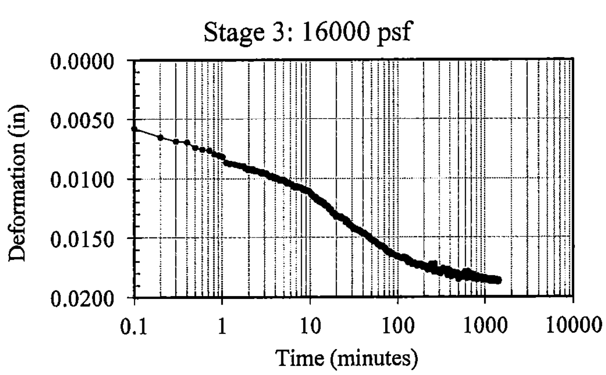
Jeffrey A. Kuhn, Ph.D., P.E., 11/20/2023
Quality Review/Date

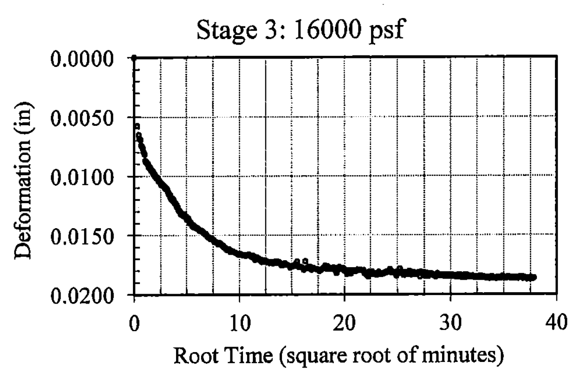












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One-Dimensional Consolidation Properties of Soil (ASTM D2435)

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Specimen: PWCG-05 (30-35)

TRI Log No.: 23-003895-01
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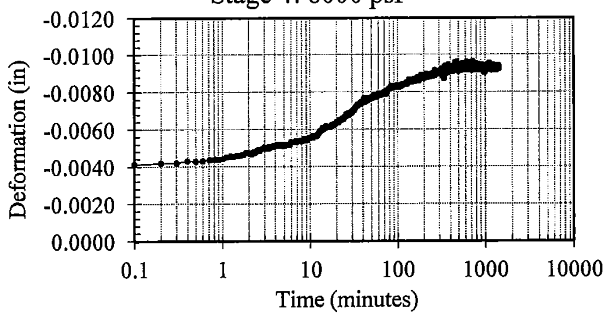
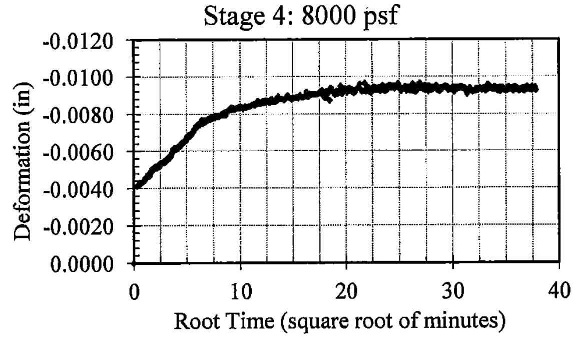
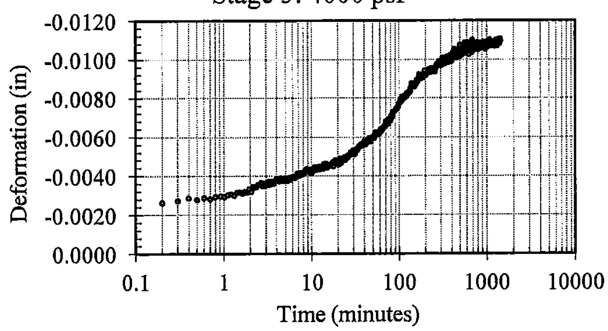
Stage 4: 8000 psf



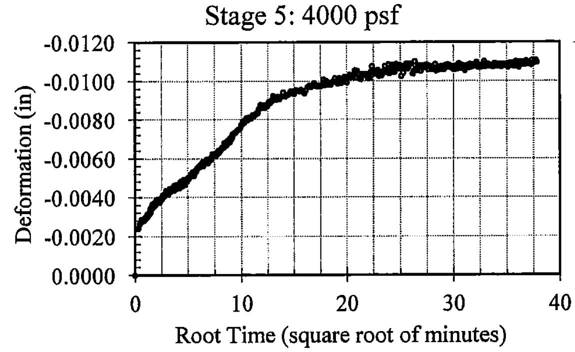
Stage 4: 8000 psf



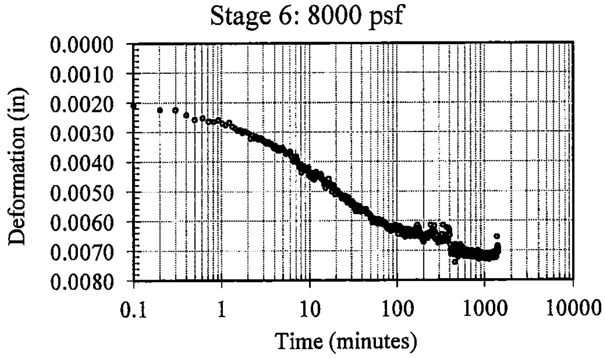
Stage 5: 4000 psf



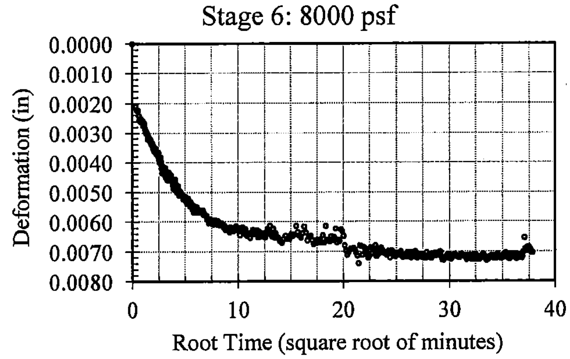
Stage 5: 4000 psf



Stage 6: 8000 psf



Stage 6: 8000 psf



One-Dimensional Consolidation Properties of Soil (ASTM D2435)

Client: Weaver Consultants Group
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Specimen: PWCG-05 (30-35)

TRI Log No.: 23-003895-01
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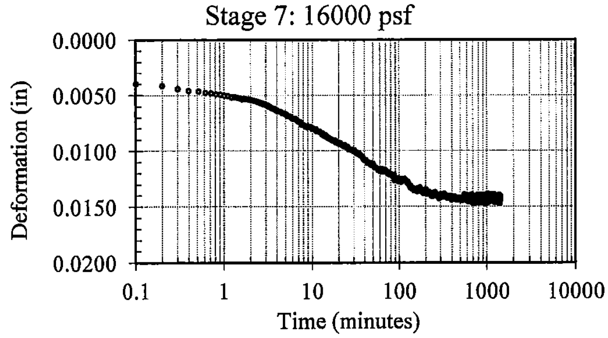
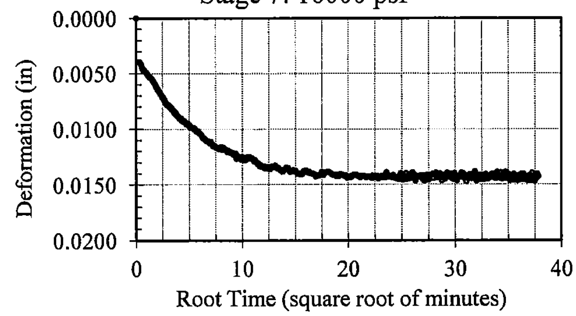
Quality Review/Date

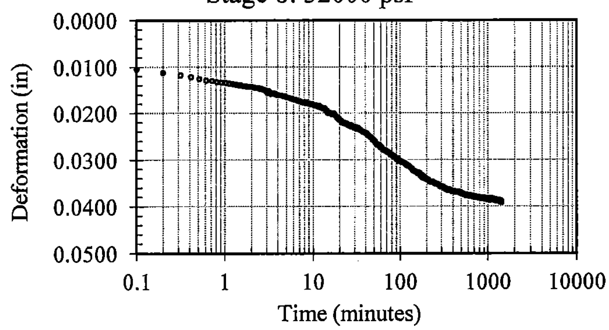
Stage 7: 16000 psf



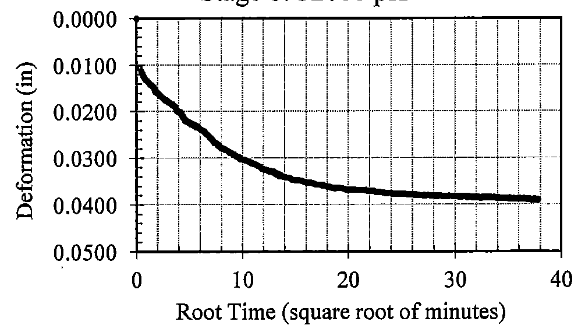
Stage 7: 16000 psf



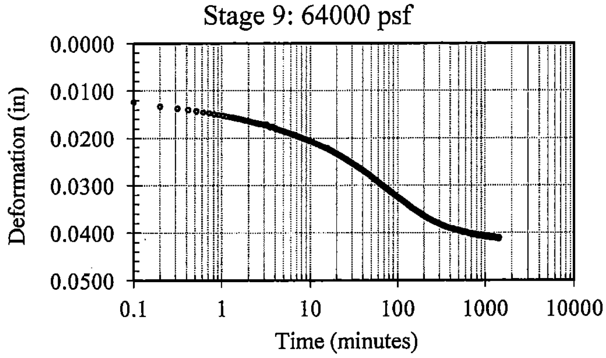
Stage 8: 32000 psf



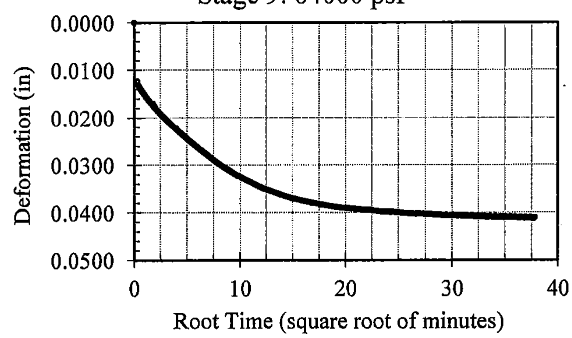
Stage 8: 32000 psf



Stage 9: 64000 psf



Stage 9: 64000 psf



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Specimen: PWCG-05 (30-35)

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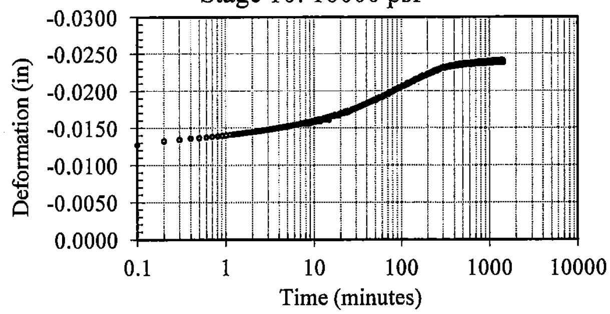
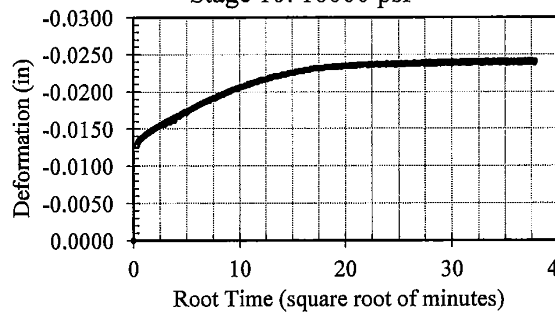
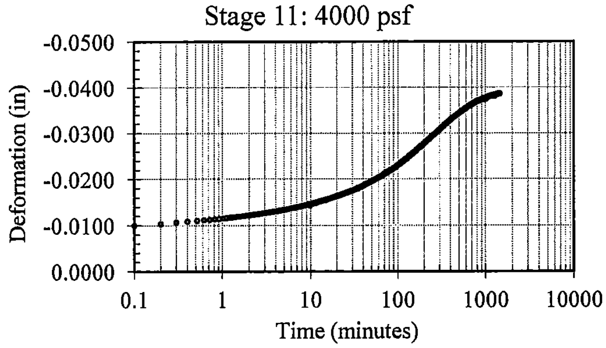
Stage 10: 16000 psf



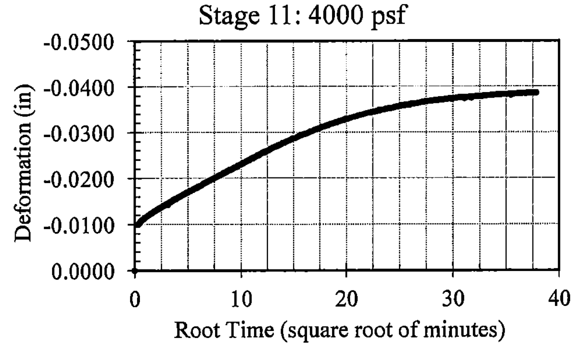
Stage 10: 16000 psf



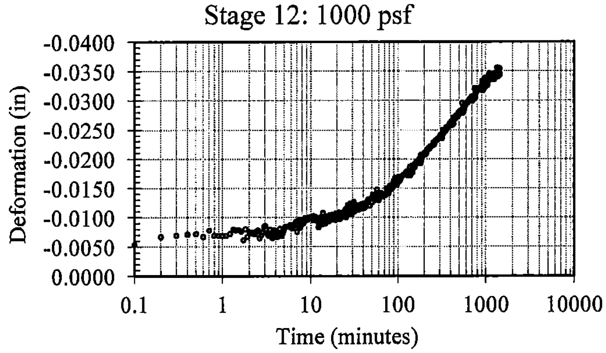
Stage 11: 4000 psf



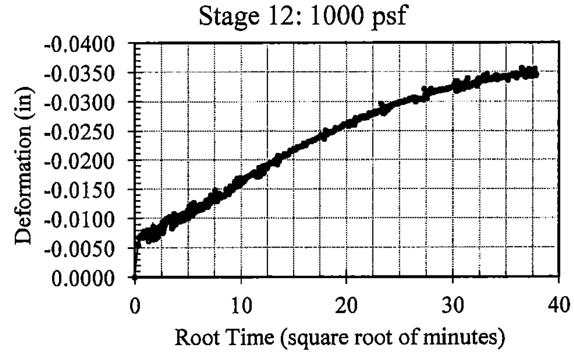
Stage 11: 4000 psf



Stage 12: 1000 psf



Stage 12: 1000 psf



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One-Dimensional Consolidation Properties of Soil (ASTM D2435)

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Specimen: WCG-10 (102-104.5)

TRI Log No.: 23-003912.5
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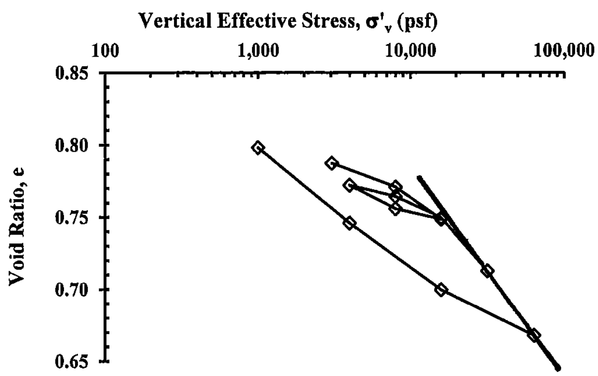
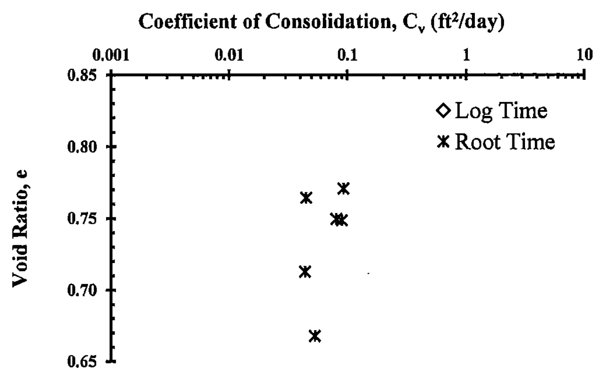
Soil Specimen Properties	Initial	Final
Water Content (%)	23.1	26.2
Diameter (in)	2.50	2.50
Height (in)	1.00	1.01
Dry Unit Weight, γ_o lb_f/ft^3	96.1	95.4
Specific Gravity (Assumed)	2.75	
Void Ratio, e	0.79	0.80
Degree of Saturation (%)	80.7	-

Test Setup		
Specimen Condition	•	Intact / "Undisturbed"
		Compacted/Remolded/Reconstituted
Test Water		Extracted Pore Water
	•	Potable Tap Water
		Demineralized Water
		Saline Water
		Other





Stage	σ'_v	log (σ'_v)	e	Strain, ε	$\frac{\delta e}{\delta \log(\sigma'_v)}$	C_α	t_{50} (min)		C_v (ft^2/day)	
(#)	(psf)	log (psf)	(-)	(%)		(x1,000)	Log Time	Root Time	Log Time	Root Time
1	3,052	3.48	0.787	0.00	-	-	-	-	-	-
2	8,000	3.90	0.771	0.93	-0.040	-	-	5.2	-	9.3E-02
3	16,000	4.20	0.749	2.17	-0.074	-	-	5.2	-	9.1E-02
4	8,000	3.90	0.756	1.76	-0.024	-	-	-	-	-
5	4,000	3.60	0.772	0.85	-0.054	-	-	-	-	-
6	8,000	3.90	0.764	1.29	-0.026	-	-	10.7	-	4.5E-02
7	16,000	4.20	0.750	2.11	-0.049	-	-	5.8	-	8.1E-02
8	32,000	4.51	0.713	4.17	-0.122	-	-	10.3	-	4.4E-02
9	64,000	4.81	0.668	6.69	-0.150	-	-	8.0	-	5.4E-02
10	16,000	4.20	0.700	4.90	-0.053	-	-	-	-	-
11	4,000	3.60	0.746	2.31	-0.077	-	-	-	-	-
12	1,000	3.00	0.798	-0.60	-0.087	-	-	-	-	-
13	-	-	-	-	-	-	-	-	-	-
14	-	-	-	-	-	-	-	-	-	-
15	-	-	-	-	-	-	-	-	-	-
16	-	-	-	-	-	-	-	-	-	-
17	-	-	-	-	-	-	-	-	-	-
18	-	-	-	-	-	-	-	-	-	-

$$C_c = \frac{.713 - .668}{\log\left(\frac{64}{32}\right)} \doteq \underline{0.15}$$

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One-Dimensional Consolidation Properties of Soil (ASTM D2435)

Client: Weaver Consultants Group

Project: 0120-076-11-01 Royal Oaks Landfill

Specimen: WCG-10 (102-104.5)

TRI Log No.: 23-003912.5

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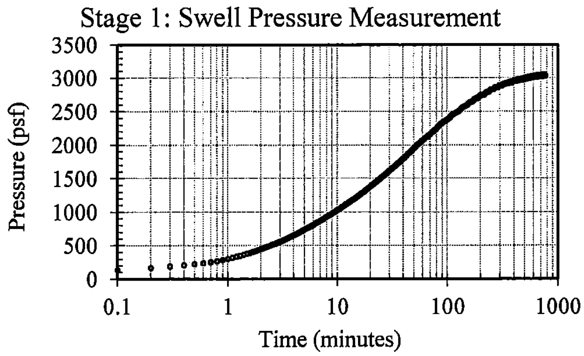
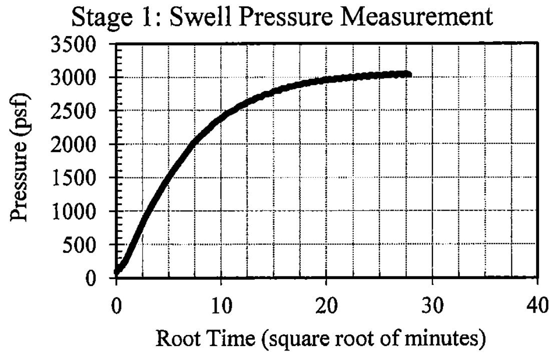
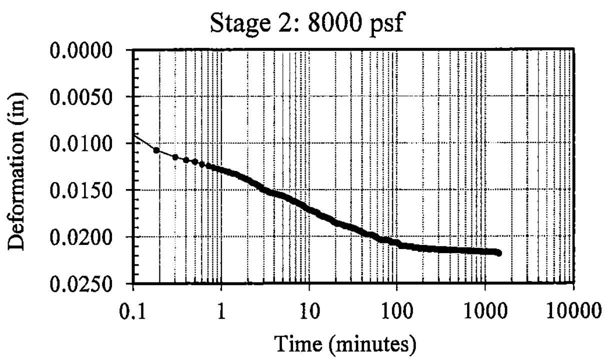
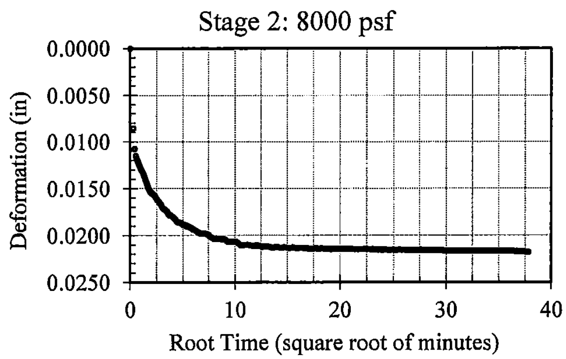
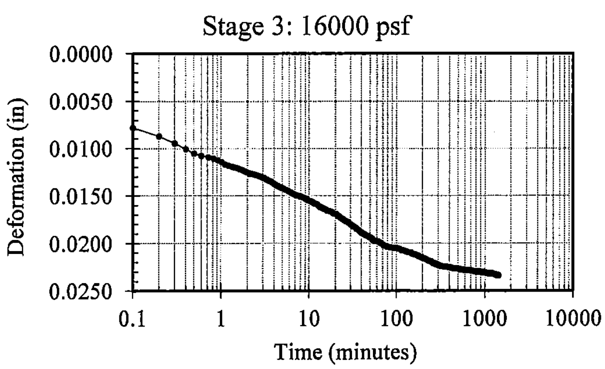
Quality Review/Date

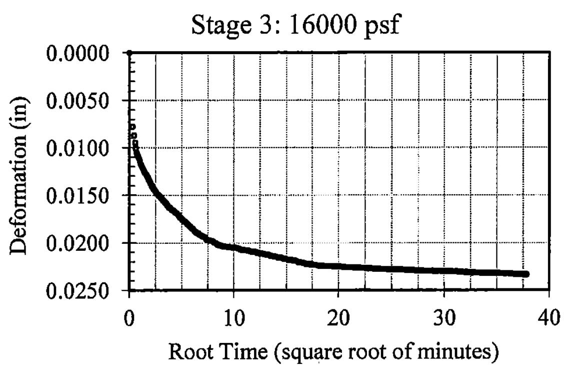












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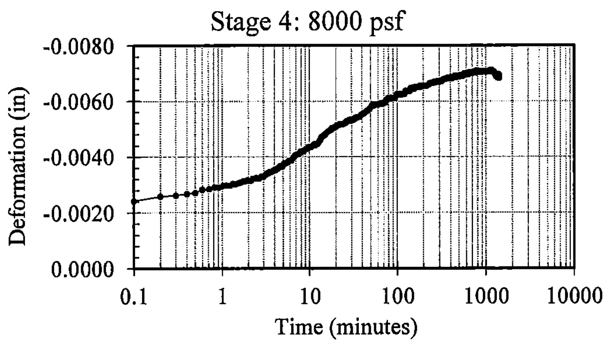

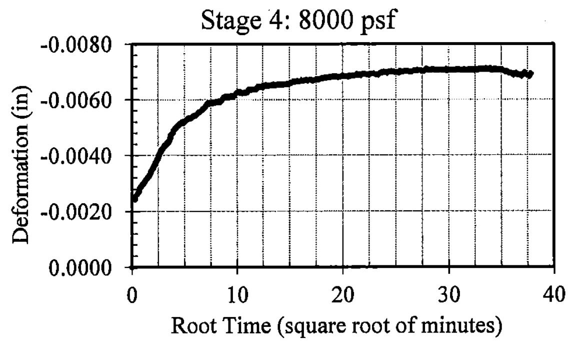

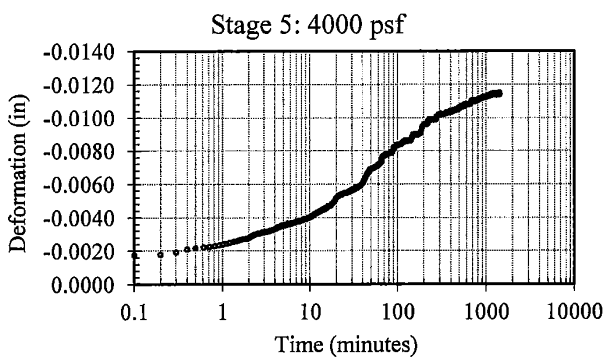

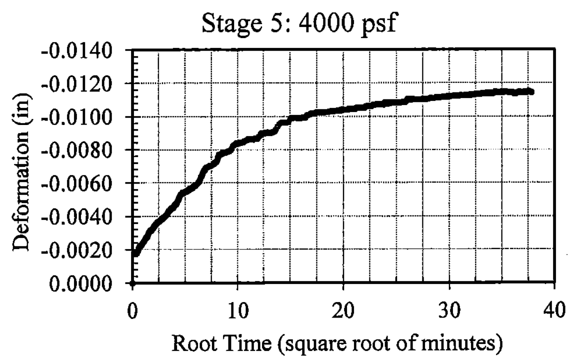

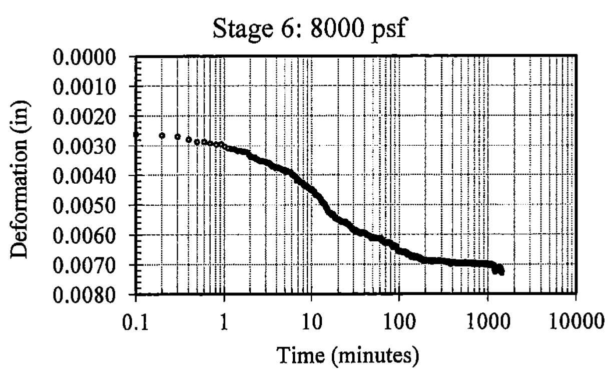

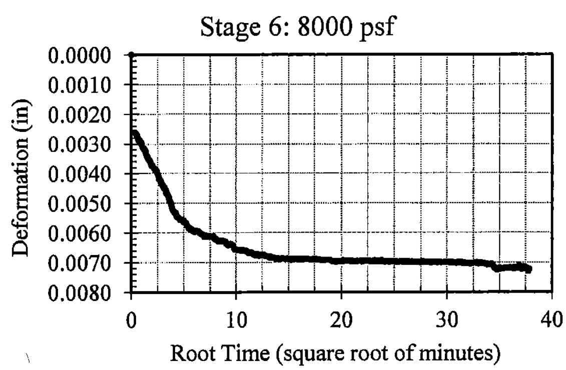

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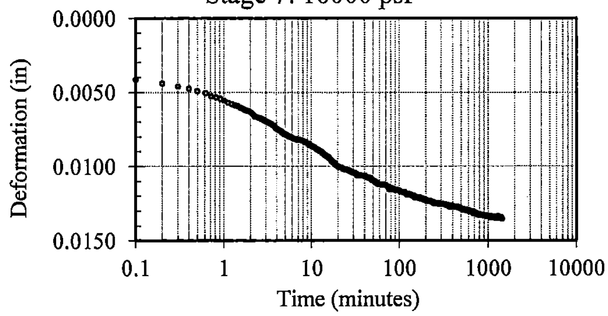
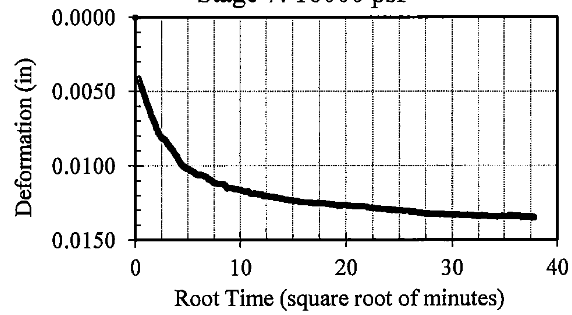
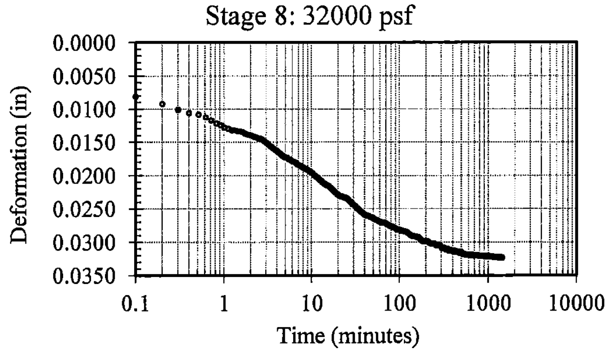
Stage 7: 16000 psf



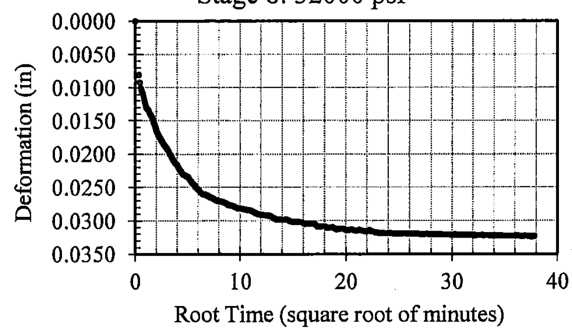
Stage 7: 16000 psf



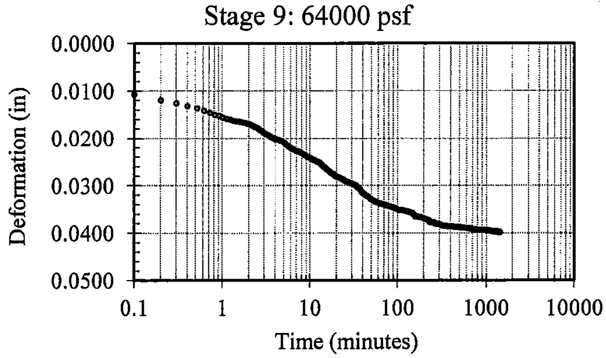
Stage 8: 32000 psf



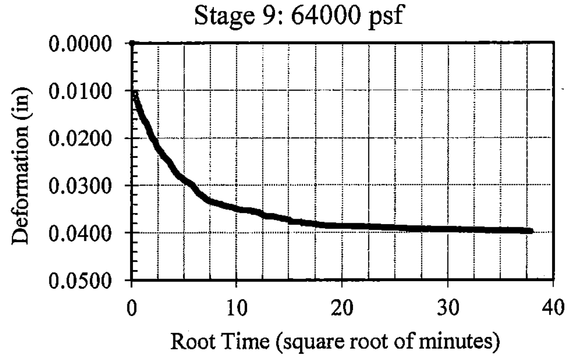
Stage 8: 32000 psf



Stage 9: 64000 psf



Stage 9: 64000 psf



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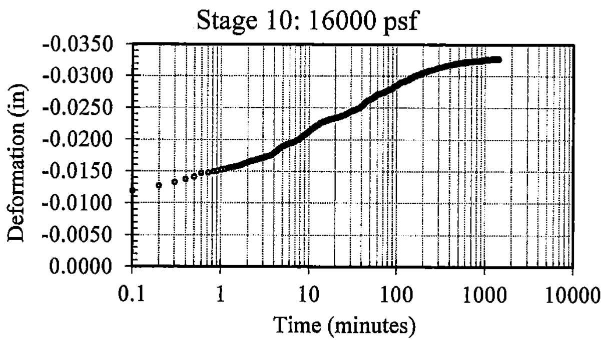

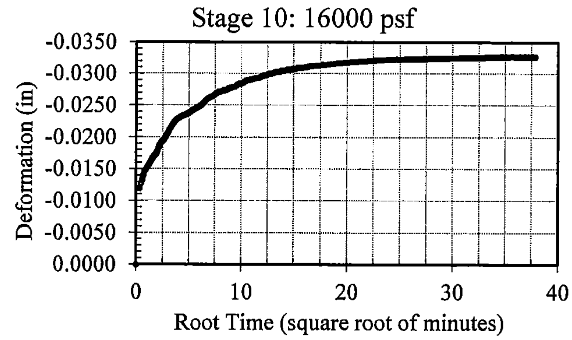

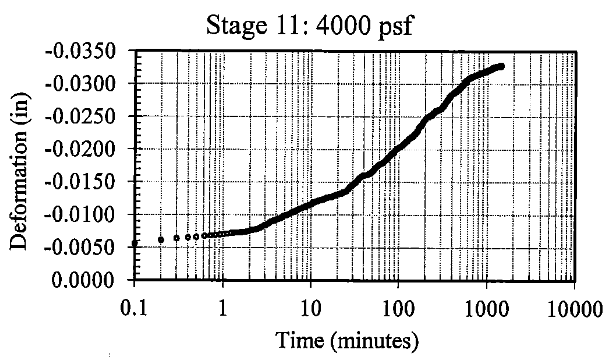

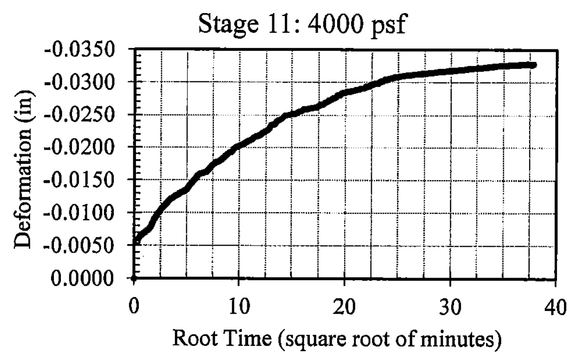

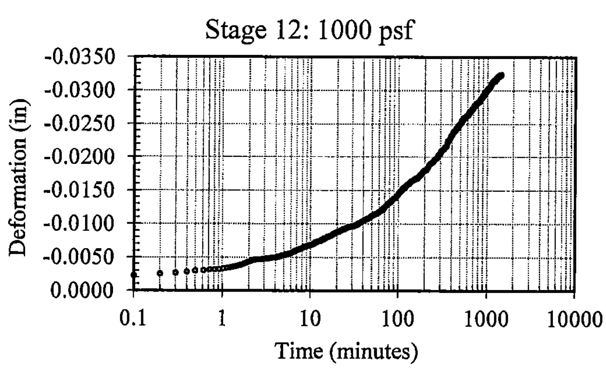

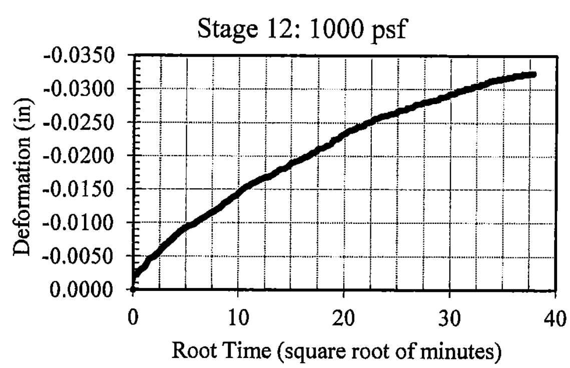

TESTING, RESEARCH, CONSULTING AND FIELD SERVICES

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One-Dimensional Consolidation Properties of Soil (ASTM D2435)

Client: Weaver Consultants Group
Project: 0120-076-11-01 Royal Oaks Landfill
Specimen: WCG-11 (44.5-57)

TRI Log No.: 23-003912.8
Jeffrey A. Kuhn, Ph.D., P.E., 11/11/2023
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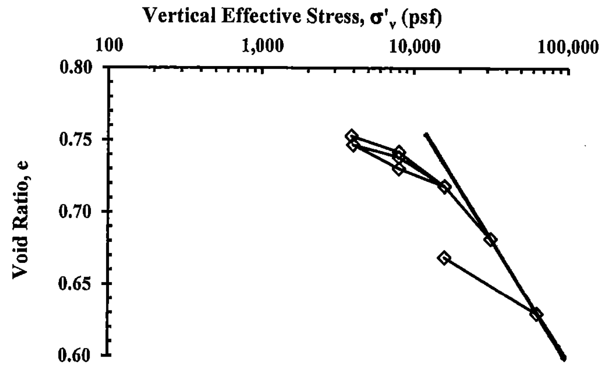
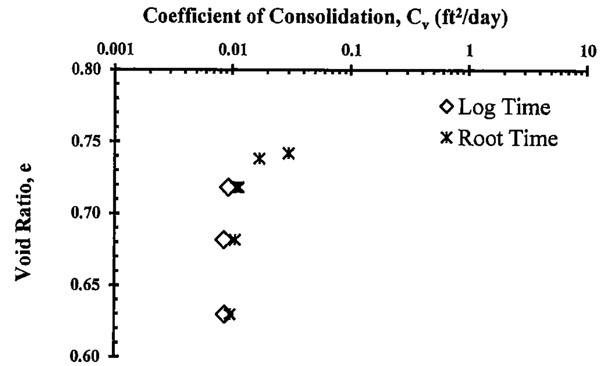
Soil Specimen Properties	Initial	Final
Water Content (%)	25.5	27.7
Diameter (in)	2.50	2.50
Height (in)	1.00	0.95
Dry Unit Weight, γ_o lb_f/ft^3	97.9	102.8
Specific Gravity (Assumed)	2.75	
Void Ratio, e	0.75	0.67
Degree of Saturation (%)	93.2	-

Test Setup		
Specimen Condition	•	Intact / "Undisturbed"
		Compacted/Remolded/Reconstituted
Test Water		Extracted Pore Water
	•	Potable Tap Water
		Demineralized Water
		Saline Water
		Other





Stage	σ'v	log (σ'v)	e	Strain, ε	δe / δlog (σ'v)	C_α	t_{50} (min)		C_v (ft²/day)	
(#)	(psf)	log (psf)	(-)	(%)		(x1,000)	Log Time	Root Time	Log Time	Root Time
1	3,898	3.59	0.753	0.00	-	-	-	-	-	-
2	8,000	3.90	0.742	0.63	-0.036	-	-	16.1	-	3.0E-02
3	16,000	4.20	0.718	1.99	-0.079	-	-	43.3	-	1.1E-02
4	8,000	3.90	0.730	1.30	-0.040	-	-	-	-	-
5	4,000	3.60	0.747	0.35	-0.055	-	-	-	-	-
6	8,000	3.90	0.738	0.85	-0.029	-	-	28.1	-	1.7E-02
7	16,000	4.20	0.718	1.98	-0.066	-	50.3	41.3	9.4E-03	1.1E-02
8	32,000	4.51	0.682	4.06	-0.121	-	52.3	42.3	8.6E-03	1.1E-02
9	64,000	4.81	0.630	7.03	-0.173	-	48.3	43.3	8.8E-03	9.8E-03
10	16,000	4.20	0.669	4.79	-0.065	-	-	-	-	-
11	-	-	-	-	-	-	-	-	-	-
12	-	-	-	-	-	-	-	-	-	-
13	-	-	-	-	-	-	-	-	-	-
14	-	-	-	-	-	-	-	-	-	-
15	-	-	-	-	-	-	-	-	-	-
16	-	-	-	-	-	-	-	-	-	-
17	-	-	-	-	-	-	-	-	-	-
18	-	-	-	-	-	-	-	-	-	-

$$C_c = \frac{.682 - .63}{\log\left(\frac{64}{32}\right)} \approx 0.17$$

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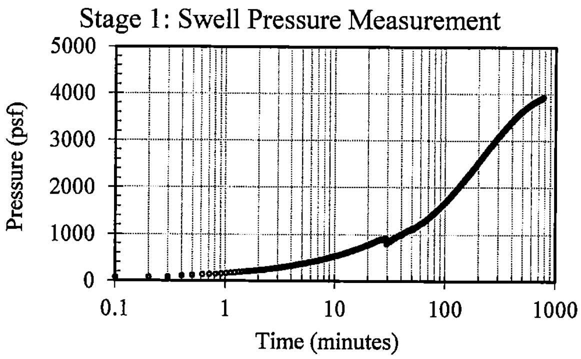

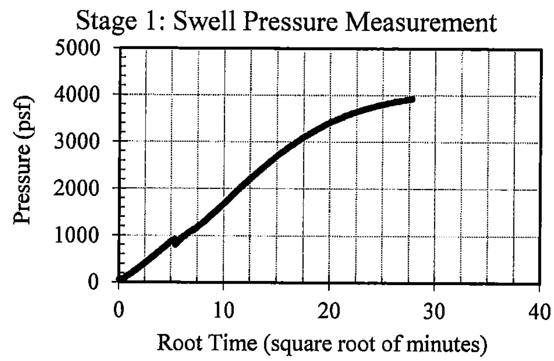

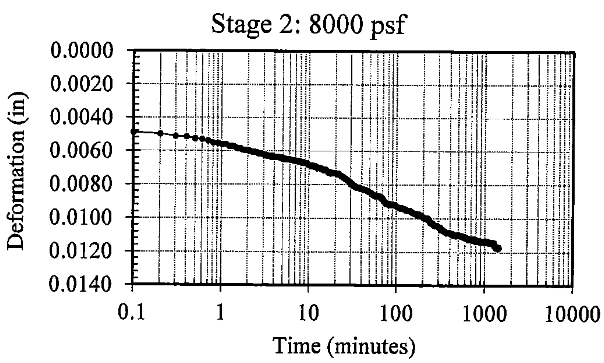

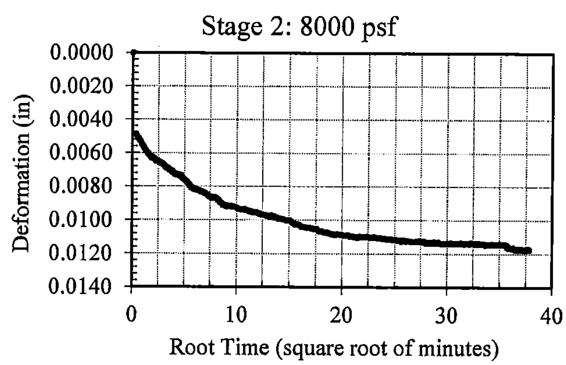

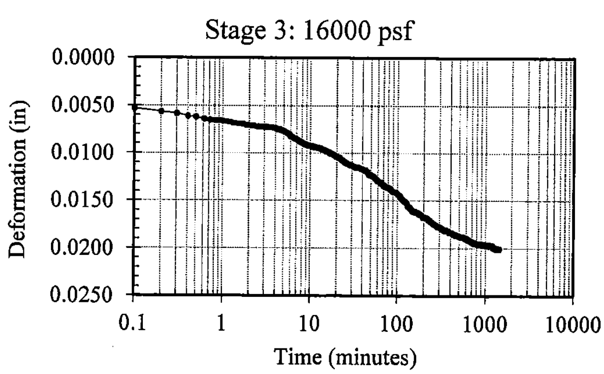

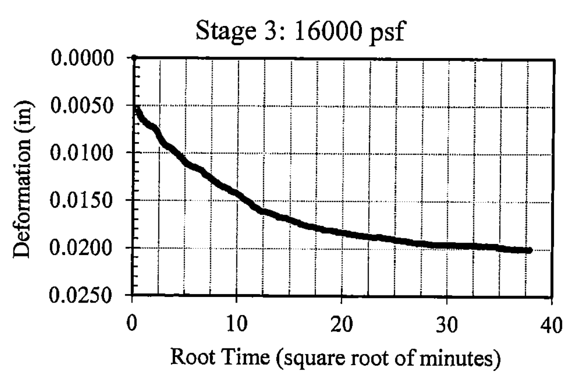

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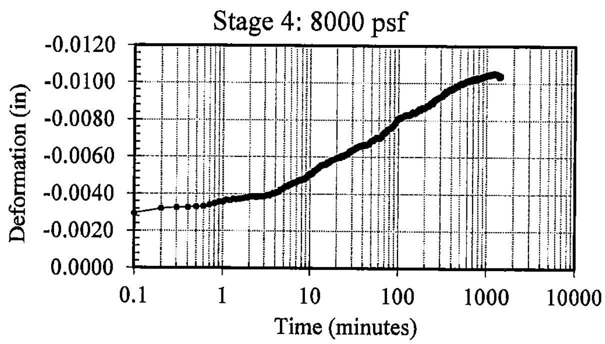

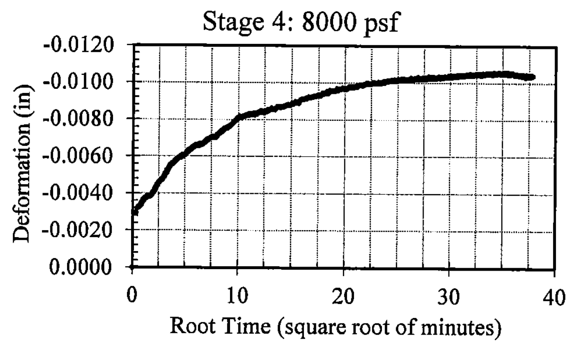

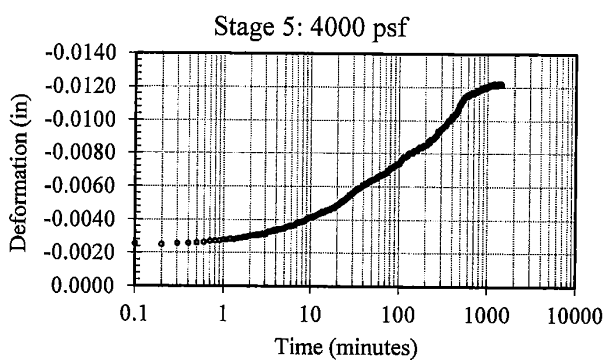

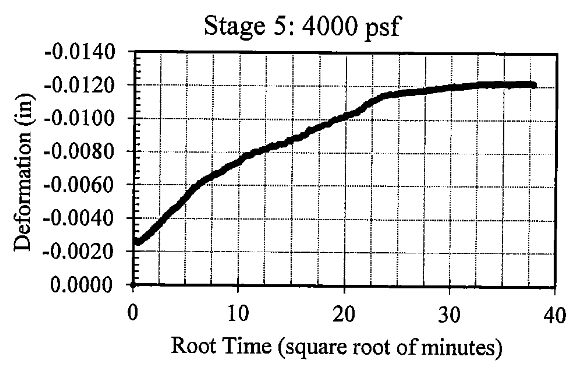

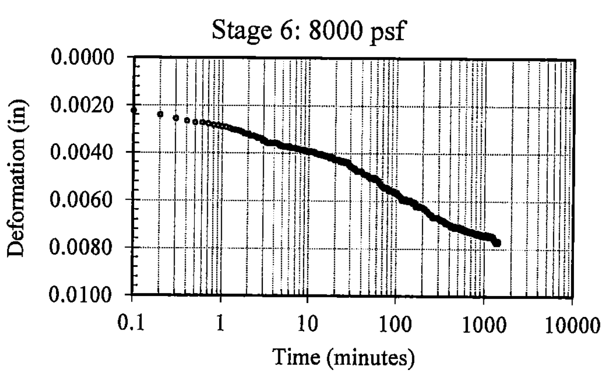

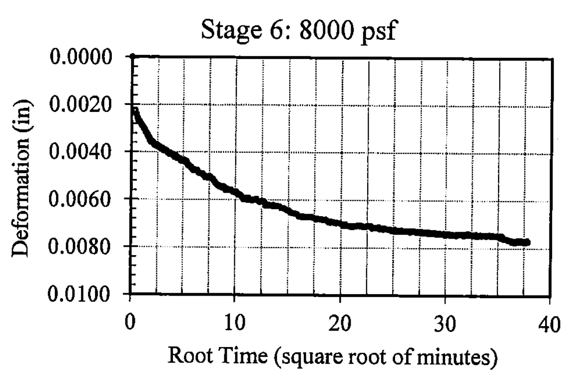

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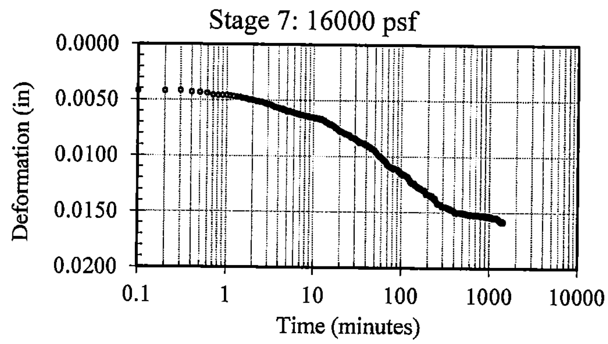

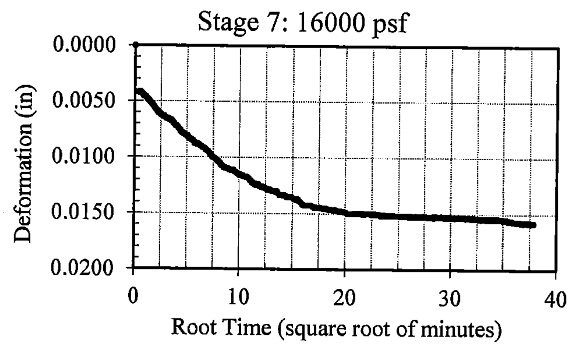

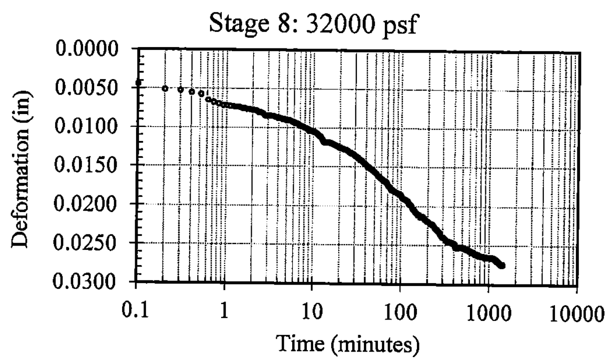

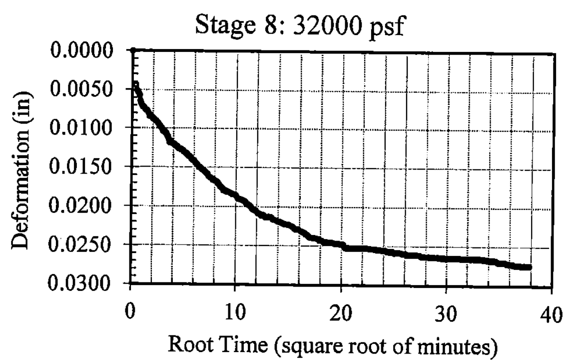

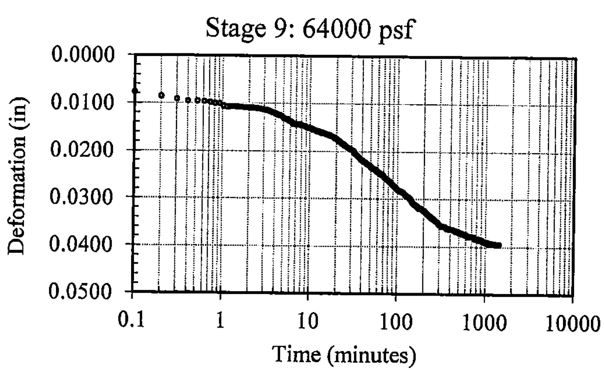

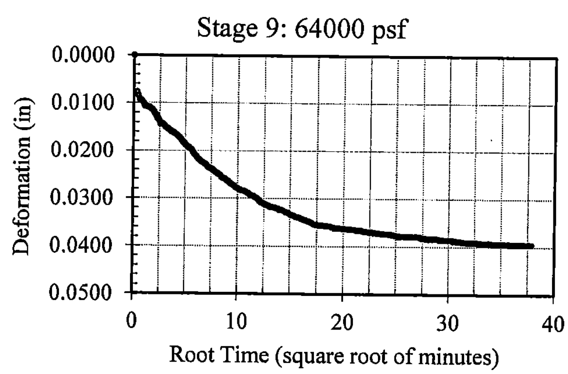

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