PLANT McDONOUGH-ATKINSON
CCR SURFACE IMPOUNDMENT
(CCR UNIT AP-1)
COBB COUNTY, GEORGIA
PART B SECTION 2
ENGINEERING REPORT

FOR

Georgia Power

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Revision 03 – December 2023

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Engineering Design Report

Plant McDonough-Atkinson
CCR Unit AP-1 Closure and Advanced Engineering

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Executive Summary

WSP USA Inc. (formerly Golder Associates Inc.) has compiled supporting calculations for the closure and advanced engineering design of CCR Unit Ash Pond 1 (AP-1) for Plant McDonough-Atkinson (Plant McDonough), owned and operated by Georgia Power Company (Georgia Power). This report provides a narrative of the closure design presented in the Closure Plan Drawings in Part A of this permit application under the following main categories:

- Geotechnical Design
- Final Cover System
- Surface Water Management
- Perimeter Barrier Wall Design (Selected Advanced Engineering Method)

This report and the appended detailed calculations are intended to meet the requirements of the Georgia Solid Waste Management Rules for Coal Combustion Residuals (391-3-4-.10) and to support the presented Closure Plan Drawings.

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1.0 INTRODUCTION

WSP USA Inc. (WSP) (formerly Golder Associates Inc), Southern Company Services (SCS) and Georgia Power Company (Georgia Power) have prepared design calculations to support the design and permitting of CCR Unit AP-1 at Plant McDonough-Atkinson (Plant McDonough or "the site"). Plant McDonough is a power generating facility, owned and operated by Georgia Power, and historically operated as a coal fired facility, utilizing coal combustion residual (CCR) surface impoundments for the storage of CCR material on-site. In 2011, Plant McDonough ceased coal-fired electric generating activities, and subsequently ceased placing CCR in the units, resulting in AP-1 becoming an inactive CCR surface impoundment prior to closure construction activities. In January 2016, closure activities were initiated for the unit, consisting of consolidating and capping, with CCR excavation in limited areas and closure in place as CCR Unit AP-1. At the time of this submittal, installation of the final cover system for AP-1 has been substantially completed and closure construction activities are ongoing, including advanced engineering methods presented in the Closure Plan Drawings in Part A Section 9 of this Permit Application.

Closure activities for AP-1 are being completed following the closure design. The overall closure design objectives consist of the following key aspects:

- A stable containment system under expected final conditions
- Perimeter containment berms that are used to contain the CCR materials once the grades of the closed unit rise above the perimeter berm elevation
- A final cover system to minimize infiltration of surface water into the unit during long term conditions
- A surface water management system used to control runoff from the units and direct it to the designed outlet structures
- A subsurface perimeter barrier wall used to provide subsurface flow inhibition around the closed CCR Unit.

The Closure Plan Drawings provide detailed grading and associated details depicting the closure design that are used as a basis for the design approach. Closure design calculations are included as appendices to this report. This document provides a summary of the various calculations and a brief narrative on the design details for each closure design element. Key design elements include the following:

- Geotechnical Design
- Final Cover System
- Surface Water Management
- Perimeter Barrier Wall (Advanced Engineering) Design

Each design element contains several design calculations, and these are discussed in more detail in this report.

2.0 GEOTECHNICAL DESIGN

2.1 General

A key element of the closure design is associated with the geotechnical stability of the closed unit both during closure construction and during post-closure. There are various elements related to the assessment of the geotechnical stability and performance of the units:

- Geotechnical material properties
- Global slope stability and settlement of the unit under final conditions

This geotechnical design discussion presents WSP's stability evaluation of the containment berms (dikes) surrounding CCR Unit AP-1 at Plant McDonough related to the requirements in the State of Georgia Solid Waste Management Rule 391-3-4-.10 and 40 CFR 257 the US EPA's Final Rule on the Disposal of Coal Combustion Residuals (CCR; EPA Rule).

This report presents the calculated geotechnical stability and settlement of the final closure condition of Unit AP-1, including the barrier wall designed as part of the advanced engineering methods for the Unit. As previously described, CCR materials have been excavated from limited perimeter areas, and the unit is being closed in place. In accordance with the Georgia solid waste permitting requirements, the following conditions were assessed:

- Storage Pool
- Surcharge Pool
- Seismic Loading Conditions
- Post-Seismic Liquefaction Conditions (when liquefaction susceptible materials are present)

Additionally, the integrity of the final cover system has also been evaluated for anchor trench and veneer stability requirements, as further discussed in Section 4.2:

- Veneer Stability Analysis (where applicable, i.e. at the gravel access road locations)
- Anchor Trench Requirements

2.2 Slope Stability Assessment Methodology

Stability safety factors were evaluated for each of the loading scenarios using the computer program SLIDE 7.0 Version 7.031 (2018). As required by the EPA rule, a general limit equilibrium (GLE) method (Morgenstern and Price) was used to calculate factors of safety, and the factor of safety is calculated by dividing the resisting forces by the driving forces along the critical slip surface.

Stability was evaluated along three cross-sections for AP-1 as depicted in Figure 1 of the Slope Stability Analysis Package (Appendix B). Subsurface stratigraphy at each cross-section was developed based on a combination of historic and recent subsurface investigations at the site and with reference to the geologic and hydrogeologic site conditions as presented in the Hydrogeological Assessment Report (Rev. 03 September 2021). Geotechnical material properties were developed for the earthen dams (dikes), foundation soils, and impounded materials from this site-specific data and WSP's greater breadth of experience with similar materials. The Material Properties Calculation Package (Appendix A) provides details on the geotechnical explorations and evaluation of geotechnical data.

The water levels used in stability analyses are reflective of long-term post-closure conditions.

2.2.1 Storage Pool Conditions

WSP modeled the long-term stability for the storage pool conditions (which consist of no permanent storage pool above the post-closure capped condition of AP-1) using the long-term water level at post-closure conditions, per the results of the three-dimensional numerical groundwater modeling with results presented in Appendix B of this Engineering Report. The long-term post-closure groundwater conditions are summarized in Appendix A of the Hydrogeological Assessment Report (HAR Revision 03) presented in Part B Section 1 of this Permit Submittal.

2.2.2 Surcharge Pool Conditions

For the surcharge pool scenario, WSP considered the impact of the 100-year, 24-hour rain event for Atlanta, GA. This event was calculated to cause temporary water flow on top of the pond cap in drainage channels. Water within these channels was calculated to rise to a depth of approximately 0.3 to 0.7 ft. The table below summarizes the water depths for each cross-section analyzed.

Section	Channel Flow Depth (ft) 100 Year 24 Hour Event
A	0.3
B	0.7
C	0.7

For further details related to the routing of surface water refer to the hydraulic and hydrology storm water routing calculations detailed in Section 4 below.

2.2.3 Seismic Loading Conditions

Factors of safety for stability under seismic loading conditions were calculated based on the earthquake hazard corresponding to a probability of exceedance of 2% in 50 years (2,475-year return period). WSP used the Bray and Travasarou displacement-based seismic slope stability screening method (Bray and Travasarou 2009) to evaluate the seismic stability. For this method, a pseudo-static coefficient corresponding to an allowable displacement of six inches (15 cm) is applied as a horizontal force in the static stability model. The pseudo-static coefficient for the above stated criteria was calculated to be 0.029g. Details on the calculation of the pseudo-static coefficient are available in the Seismic Hazard Calculation Package (Appendix C).

2.2.4 Post-Seismic Liquefaction Loading Conditions

The State of Georgia Solid Waste Management Rule, referencing CCR Rule §257.73(e)(iv), the CCR Rule specifies a required factor of safety of 1.2 against liquefaction for pond impoundment structures. WSP completed an evaluation of the liquefaction susceptibility of the site soils which will remain saturated in the long term as presented in the Liquefaction Assessment Calculation Package (Appendix D). The calculated factor of safety against liquefaction is above 1.2 for all materials analyzed except for portions of the impounded ash originally placed sluiced. Thus, post liquefaction stability was evaluated using a reduced post-liquefaction strength for such materials (modeled as a post liquefaction strength ratio of 0.08). For additional details on the liquefaction analysis, please refer to Appendix D.

2.3 Slope Stability Assessment Results

The table below presents the results of the slope stability analyses for the AP-1 dikes. For all cases analyzed, the calculated factors of safety are in excess of those required by the State of Georgia Solid Waste Management Rule, referencing CCR Rule Sections § 257.73(e)(i) to (iv). The detailed stability results are presented in Figures 2 through 4 of Appendix B. These results have been updated to reflect the calculations performed in April 2023.

Long-Term Post-Closure Stability Analysis Results				
Analysis Case	Storage Pool	Surcharge Pool	Seismic	Post Liquefaction
Rule Section	§ 257.73(e)(i)	§ 257.73(e)(ii)	§ 257.73(e)(iii)	§ 257.73(e)(iv)
Target Factor of Safety	1.5	1.4	1.0	1.2
Cross-Sections	**Factor of Safety**			
A-A	1.6	1.6	1.5	1.6
B-B	1.6	1.6	1.3	1.6
C-C	1.5	1.5	1.4	1.5

2.4 Geotechnical Analysis Conclusions

WSP evaluated the slope stability of dikes surrounding AP-1 at Plant McDonough in accordance with the EPA Rule on the Disposal of Coal Combustion Residuals. Specifically, the containment berms (dikes) were evaluated for stability in the following loading scenarios:

- Storage Pool
- Surcharge Pool
- Seismic Loading Conditions
- Post-Seismic Liquefaction Conditions (when liquefaction susceptible materials are present)

For each loading case, the cross section analyzed under this study were found to meet the target factor of safety presented in the governing rules. Additionally, WSP performed veneer stability for the gravel access road on the final cover.

- Settlement Analysis

Long-term settlement potential for AP-1 was calculated and used to evaluate the potential for grade reversals or other settlement induced issues. In general, CCR is much less susceptible to long term settlement than typical municipal solid waste (MSW) and Construction & Demolition (C&D) waste masses and as such liner components and drainage grades are less prone to settlement induced issues in CCR closures. The settlement evaluations for the closed AP-1 conditions consider settlement following closure from dewatering of the CCR and minimal post capping settlement (less than a few inches) across the pond, with a maximum calculated post capping settlement predicted to be 0.43 ft in the northwest area of AP-1, as detailed in Appendix E.

- Veneer Stability Analysis

Long-term and short-term veneer stability analyses were performed for the critical access road conditions applicable to the CCR Unit closures on site (noting the AP-1 closure grades are lower and generally shallower than the AP-3/4 closure grades), including incorporation of equipment acceleration on the roads and were found to meet the required factors of safety as detailed in Appendix G.

- Anchor Trench Analysis

Closure cover liner anchorage was evaluated, and 2 feet deep by 2 feet wide anchor trenches were evaluated to be adequate for the closure as detailed in Appendix H.

3.0 FINAL COVER SYSTEM

3.1 General

The closure of Unit AP-1 was designed with a final cover system that consists of two options for the final cover configuration of the unit. Each cover system option proposed consists of an 18-inch minimum layer of prepared and compacted subgrade material and incorporates a geosynthetic final cap.

Option 1 consists of a ClosureTurf™ geosynthetic cap system utilizing a variety of infill options dependent on the designed closure area. The ClosureTurf™ final cover system consists of:

- 18-inch thick (min.) layer of compacted CCR or earthen subgrade material
- 40-mil minimum Agru linear low-density polyethylene (LLDPE) geomembrane
  - 40-mil MicroSpike® LLDPE geomembrane is utilized for closure areas with final cover surface slopes of less than 10 degrees (10°); or
  - 50-mil Super Gripnet® LLDPE geomembrane (or 50-mil MicroDrain® LLDPE geomembrane) is utilized with spikes down for cover slope areas greater than 10 degrees (10°)
- ClosureTurf™ (combined 8 ounce per square yard (oz/yd²) geotextile and engineered turf layer)
- Turf Infill or Overlying Protective Layer Options
  - Sand infill (0.5-inch minimum) typical design; or
  - Sand infill (0.5-inch minimum) with Armorfill E application; or
  - Hydrobinder® infill (0.75 inch minimum); or
  - Rock or Articulated Concrete Block (ACB) armoring overlying a geosynthetic separation and protection layer.

The Super Gripnet® and MicroSpike® will serve as a flexible membrane liner (FML) barrier to infiltration and are designed such that drainage to convey stormwater off of the FML areas is maintained between the geomembrane and the geotextile of the ClosureTurf™ layer.

Option 2 consists of a closure layer as required for CCR unit closures in §257.102(d)(3)(i) consisting of the following layers:

- 18-inch thick (min.) infiltration layer of compacted material with a minimum hydraulic conductivity of 1 x 10^{-5} centimeters per second (cm/s)
- 6-inch vegetative soil layer with grassy vegetation

The 6-inch vegetative layer of Option 2 is designed to support vegetation over the final cover system. Both final cover system options are designed to overlay the full limits of permanently stored CCR and the interior surfaces of the adjacent containment dike berms. Surface water diversion berms consisting of compacted material are graded into the final cap grading side slopes and are designed to be overlain by the final cover system.

Details of the final cover system options are presented in the Closure Plan Drawings for Unit AP-1 in Part A of this Permit Application.

3.2 Alternative Final Cover Design

As indicated in Section 3.1, the final cover system designed for AP-1 consists of a ClosureTurf™ geosynthetic cap system utilizing a variety of infill options as delineated in the Closure Design Plans. As part of the closure design, WSP completed an evaluation of the percolation potential and liner performance for the final cover system designed for AP-1 in comparison to a CCR unit final cover system (State of Georgia Solid Waste Management Rule, referencing CCR Rule §257.102(d)(3)(i)). The analysis presents estimates and ranges of the anticipated drainage collected from the final cover system as well as percolation estimates through the geomembrane cover. The performance for the designed final cover system, consisting of ClosureTurf™, demonstrates equivalent or superior performance to a traditional soil cover system, as per regulatory requirements (State of Georgia Solid Waste Management Rule, referencing CCR Rule 257.102(d)). Additional detail on the cover equivalency calculations can be found in Appendix F.

3.3 Veneer Stability Analysis

Veneer stability analyses were performed for the final cover system at locations where the final cover system is overlain by another material. For the AP-1 ClosureTurf™ final cover system, these are the locations of access roads where a nominal 6-inch gravel layer is placed on the top of a separation and cushion geosynthetic over the Closure Turf™. Veneer stability factors of safety were calculated using the Koerner and Soong method (Koerner and Soong 1998). The maximum slope percent of the access road is 10 percent. Veneer stability analysis was conducted assuming the height of the slope to be the difference between the highest elevation and the lowest elevation of the access road. It should be noted that most of the slopes at the closed units will be shorter than the maximum slope, and thus will be less critical than accounted for in this analysis.

WSP analyzed that both static and equipment loading scenarios meet the required factors of safety. Details on the calculation of the veneer stability analyses and veneer stability analysis methodology, as well as loading specifications are included in the Veneer Stability Analysis Calculation Package (Appendix G).

3.4 Final Cover Anchor Trench

The ClosureTurf™ final cover system is designed to cover the AP-1 final CCR limits following capping of the CCR material. Appendix H presents the calculated requirements for runout length and anchor trench width and depth for appropriate protection against being compromised by wind and water. An anchor trench with 2 ft depth and 2 ft width is calculated to be adequate for the range of proposed anchorage conditions.

4.0 SURFACE WATER MANAGEMENT

4.1 General

The surface water management system for Unit AP-1 includes several controls for stormwater management at the CCR Unit. The AP-1 system manages the stormwater runoff from the closed landfill surface and select surrounding areas, totaling approximately 41.7 acres of contributing area. Stormwater is routed over the closure system through a system of landfill downslope channels and perimeter channels to two outfall points.

The AP-1 closure system provides attenuation of the inflow design flood as detailed in Section 4.2, which outlines a comprehensive calculation package for the stormwater management system that consists of a series of ditches, and culverts for routing to one of two outfall structures for discharge.

4.2 Surface Water Management Analysis

Appendix I includes a comprehensive surface water management calculation for Unit AP-1. The calculation package estimates run-off for storm events under final development conditions for the unit to the stormwater management system's two stormwater attenuation ponds and corresponding discharge structures – the North Pond and South Pond. Type II rainfall distribution was used for all modeling efforts, and all structures were designed based on the discharge from the 100-year, 24-hour storm event.

Details of the hydrologic analysis are included in the calculation package provided in Appendix I. Watersheds were delineated to route to the North or South attenuation ponds through a series of flat bottom, or trapezoidal, ditches ranging from 2 to 8 ft bottom length conveying flows ranging from 2 to 55 cubic feet per second (cfs); these are designed as either Hydroturf® or riprap lined channels. Four downchutes convey water from the top deck of AP-1 to the perimeter ditches, and ultimately to the North Pond and South Pond. Culverts for road and berm crossings have also been designed and are summarized in Appendix I. The North and South ponds are designed to allow for temporary retention to attenuate the runoff inflow hydrograph and reduce the peak discharge to less than the estimated pre-development discharge, as required by Georgia stormwater guidelines and the requirements for inactive CCR surface impoundments. The stormwater management design includes the following:

- Downchutes for surface water conveyance
- Trapezoidal perimeter channels for surface water conveyance
- Culverts for road and berm crossings at the northwest, southeast, and east perimeter road
- North and South attenuation ponds, for retention of water following storm events and designed to fully drain through piped conveyance systems and outlet structures to provide for controlled gravity stormwater conveyance into rip rap aprons
    - North Pond totals 6.1 acre-feet cumulative storage volume
    - South Pond totals 2.3 acre-feet cumulative storage volume
- Auxiliary overflow spillways at the North and South attenuation ponds

5.0 PERIMETER BARRIER WALL

A subsurface perimeter barrier wall, selected as the advanced engineering method for CCR Unit AP-1, is designed to fully encompass AP-1. The perimeter barrier wall is designed to divert groundwater flow around the CCR material and inhibit groundwater flow in the CCR material. The barrier wall ranges in design depth between

8 and 75 feet, with an average depth of nominally 50 ft, and designed to terminate at the top of partially weathered rock (PWR) as identified in the Closure Plan Drawings.

Design for the barrier wall consists of a soil-bentonite slurry mix to form a continuous, and low permeability barrier to attenuate groundwater flows into and from the unit. Performance requirements of the wall consist of a maximum hydraulic conductivity of 1×10^{-7} cm/sec. Additional performance requirements are presented the CCR Unit AP-1 Construction Quality Assurance (CQA) Plan in Part A Section 5 of the Permit Application.

In 2023, WSP completed a supplemental bench scale testing program ((Phases 2 & 3) that consisted of mixing site soils with different percentages of bentonite, Portland cement, Ground Granulated Blast Furnace Slag (GGBFS) and water (W) to evaluate potential mixes to achieve the Performance Requirements presented in the CQA Plan. The latest phase of supplemental mixes (Phase 3) included the addition of GGBFS to the mixes to further reduce the permeability of the mixes. The findings from this bench scale testing program are presented in the Phase 2 & 3 Laboratory Bench Scale Testing for Plant McDonough CCR Unit AP-1 AEM Perimeter Subsurface Barrier Wall Technical Memorandum, included as Appendix K to this Engineering Report. The testing summarized in the bench scale testing memorandum demonstrates that the proposed barrier wall can be constructed by mixing specific percentages of site soils with additives to produce an engineered mixture consisting of water, bentonite, and common construction industry binders (i.e., Portland cement and GGBFS) to produce a hydraulic barrier wall capable of meeting the design Performance Requirements established in the CQA Plan.

The long-term slope stability of the proposed dikes presented in Section 2 of this Engineering Report and supported by the calculations presented in Appendix B were performed including the barrier wall, and for each loading case, the proposed dikes with barrier wall were calculated to meet or exceed the target factors of safety.

6.0 CLOSING

This engineering design report provides a summary of key calculations for the design of the final closure for Plant McDonough's CCR Unit AP-1. Appendices to this report include calculations as discussed herein.

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WSP, 2023. Hydrogeologic Assessment Report for Plant McDonough-Atkinson CCR Unit AP-1

APPENDIX A

Material Properties Calculations Package

GOLDER	**SUBJECT:** **Geotechnical Material Property Package** **Project Number:** **1777449** **Project Name:** **Plant McDonough Surface Impoundment Units AP-1, AP-2, and AP-3/4 Closure** **Prepared by:** JGM **Checked by:** LJ / LS **Date:** Jul 2018 **Reviewed by:** GLH

1.0 OBJECTIVE

Estimate strength parameters for coal combustion residuals (CCR) in-situ soils and soil fill at Ash Ponds 1, 2, 3, and 4 (AP-1, AP-2, AP-3 and AP-4) located at Georgia Power Company's (GPC) Plant McDonough-Atkinson (Plant McDonough) in Cobb County, GA.

Materials types considered include the following:
- Sluiced CCR
- Stacked / Compacted CCR
- Fill Soils
- Upper Residuum
- Lower Residuum

2.0 METHOD

Material parameters used in analyses were estimated based on a combination of the following:

- Information collected during Golder's geotechnical investigation in October and November 2015, and Golder's supplemental January 2016 investigation. The field investigation included: cone penetration testing (CPT), standard penetration testing (SPT), fixed piston tube sampling, vane shear testing (VST), and groundwater monitoring data;
- Correlations of strength parameters from CPT data (Lunne et al, Conetec, Robertson, Mayne)
- Geotechnical laboratory testing (Direct Shear, Triaxial, Plasticity, Proctor Compaction, Particle Size, Permeability, etc.);
- Correlations of strength parameters from SPT N-values, plasticity indices, and published values (Mesri and Shahien Plasticity correlations, Peck et al. and Meyerhof, etc.);
- Empirical relationships and/or typical ranges of values for the applicable materials; and,
- Golder's professional experience.

Golder also analyzed data provided in AMEC's 2010 report titled "Report of Dam Safety Assessment of Coal Combustion Surface Impoundments, Plant McDonough, Smyrna, GA." Golder found this data to be consistent with data collected during Golder's various site investigations. Interpretation and analysis of data for each soil type is summarized in the subsequent sections.

2.1 Abbreviations / Symbols:

ϕ = friction angle
ϕ' = effective friction angle
ϕ_r = residual friction angle
c = cohesion
c' = effective cohesion
S_u = undrained shear strength
γ = unit weight
γ_{sat} = saturated unit weight
CCR = coal combustion residuals
SCS = Southern Company Services
GPC = Georgia Power Company
deg = degrees
psi = pounds per square inch
psf = pounds per square foot
pcf = pounds per cubic foot
tsf = tons per square foot
SPT = Standard Penetration Test
CPT = cone penetration test
SCPT = seismic cone penetration test
VST = vane shear test
ft-msl = feet above mean sea level (elevation)
ft-bgs = feet below ground surface (depth)
AP = ash pond

3.0 CCR MATERIAL GEOTECHNICAL DESIGN PROPERTIES (PONDS 1, 3, & 4)

Objective

Develop geotechnical design parameters for the soils and Coal Combustion Residual (CCR) materials at AP-1 and AP-3 and AP-4 at GPC's Plant McDonough.

Lab Testing

Soil samples were collected via standard penetration testing (SPT) from October 26 through 29, 2015 and sent to Golder's geotechnical laboratory for analysis. Additional samples collected from SPT and fixed piston tube methods were collected during the January 2016 supplemental CCR investigations. Borings adjacent to the following CPTs were completed within the AP-3/4 area: CPT-18, -19, -30, and -36, PZ-02, and partial depth borings (CPT-28, 32, 33, and 39) in Dry Stack Investigation Area #1, and two partial depth borings (CPT-41 and 42) within Dry Stack Investigation Area #2 on Ash Pond #4.

Laboratory properties of CCR samples tested are summarized in the table below. Further laboratory information can be found in the attached documents.

Summary of Geotechnical Testing Data - Material Properties CCR (AP-3 & AP-4)					
Property	***No. of Data Points***	**Min**	**Max**	**Avg**	**Med**
Water Content (%)	12	21.4	82.8	43.0	42.9
Gravel (%)		0.0	3.2	0.4	0.0
Sand (%)		1.4	52.2	17.0	15.1
Fines (%)		44.6	98.6	83.0	84.9
Clay-Sized Particles (%)	8	8.0	30.0	19.1	16.8
Liquid Limit (LL) (%)	2 (10 NP)	33.5	35.6	34.5	--
Plastic Limit (PL) (%)		30.0	32.0	31.0	--
Plasticity Index (PI)		3.5	3.6	3.5	--
Max Dry Density (pcf)	2	85.0	87.4	86.2	--
Optimum Moisture (%)		23.8	26.6	25.2	--

Calculated & Measured Unit Weight

Saturated unit weight was calculated based on in-situ moisture content and specific gravity for six samples of sluiced CCR collected in borehole PZ-02 and was directly measured in two undisturbed Shelby tube samples collected in boreholes CPT-18 and CPT-19.

The water content of CCR samples taken below the water table and assumed saturated conditions were used to calculate saturated unit weights. From laboratory testing and Golder's extensive experience with CCR, a specific gravity between 2.15 was assumed for the calculations of samples for which specific gravity had not been directly measured. The formula below was used to calculate the saturated unit weights.

$$\gamma_{sat}\ (pcf) = [G_s * \left(\frac{1}{1+G_S * w}\right)] * [62.4 * (1 + w)]$$

Where:

w = water content of sample

Gs = Specific gravity of ash solids (2.15 to 2.45)

62.4 pcf = unit weight of water

Using calculated units weights, unit weights measured in undisturbed samples, and Golder's experience with CCR material, a total unit weight of 90 pcf was selected for sluiced CCR. A total unit weight of 110 pcf was selected for stacked CCR based on proctor test results and Golder's experience. A summary of the calculated and measured saturated unit weight ranges and a summary of selected unit weight values are listed below.

Summary of Calculated & measured Unit Weight Sluiced Ash					
Property	***No. of Data Points***	**Min**	**Max**	**Avg**	**Med**
Calc & Meas Sat Unit Weight (pcf)	8	88.2	108.0	98.0	98.1

Selected Representative Unit Weights (pcf)	
Uncompacted, Saturated	90
Stacked / Compacted, Saturated	110

Strength Data

Strength parameters for CCR were evaluated based on Golder's in-situ investigation (CPT and BHs) and are summarized in the table below. Strength parameters were selected for AP-1, AP-3 and AP-4 for Sluiced and Stacked conditions. In AP-3 and AP-4, sluiced ash is identified as material deposited below an elevation of 840 ft-msl, or 6 feet below the dam crest elevation.

Unit weights and CPT strength data in the sluiced CCR show an increasing trend with regard to increased stress/depth below elevation 840. Strength trends in the sluiced ash do not appear to be affected by the weight of stacked ash on top of the sluiced ash. This independent behavior suggests a structure formed in the sluiced ash preventing stacked ash operations from consolidating the sluiced ash. That is, stresses imposed by the stacked ash were not large enough to affect the strength of the structured sluiced ash. Therefore, vane shear tests in the sluiced ash were normalized by vertical effective stresses calculated by neglecting the stacked ash material (material above elevation 840).

Drained friction angles of 24 and 30 degrees (Figures 1, 2 and 3) were selected for the Sluiced and Stacked CCR, respectively. The selected compacted CCR friction angle is based on the average correlated friction angle from CPT (33.5° and 35.2°, respectively for AP-3 and AP-4) and the lab test results from direct shear and triaxial testing (29-30 degrees). Peak strengths from lab testing indicate appropriate conservatism of the selected friction angle for stability analyses. For the drained condition and the vertical stress range tested, the CCR is best modeled without a cohesion parameter, according to the laboratory results, noting that apparent cohesion will exist due to capillarity in partially saturated samples.

An undrained strength represented with a friction angle of 12.4 degrees and cohesion of 0.05 tsf was selected for the sluiced CCR based on the lowest total strength envelope from CU test, correlated CPT values (Figure 4 and 5), and vane shear results. Frictional parameters were selected for the stacked CCR based on fitting the lower bound correlated undrained shear strength (from CPT) with depth (Figures 4 and 6). Friction angle of 24^{o} and cohesion value of 0.18 tsf was selected.

In some cases, CCR is susceptible to liquefaction. For analyses requiring a post-liquefied or post-earthquake strength, a stress ratio (S_u/σ'_v) of 0.08 with a minimum undrained shear strength of 0.05 tsf was selected based on Golder's experience.

Summary of Geotechnical Strength Data

AP-3 CCR- Stacked (Above Elev. 840 ft-msl)

Property		No. of Data Points	Min	Max	Avg	Median
Drilling	*SPT N (bpf)*	-	-	-	-	-
	φ' (°) (Meyerhof)	-	-	-	-	-
	φ' (°) (Peck et al.)	-	-	-	-	-
CPT Interpreted	*Peak φ' (°)*	1536	22.4	>40	33.5	32.4
	Su (tsf)		0.12	>4	3.32	1.73
	SPT N_{60} (bpf)		1	>100	20	12
	Norm. CPT Tip (Qtn)		3.3	385.3	50.6	25.0

AP-3 CCR - Sluiced (Below Elev. 840 ft-msl)

Property		No. of Data Points	Min	Max	Avg	Median
Drilling	*SPT N (bpf)*	-	-	-	-	-
	φ' (°) (Meyerhof)	-	-	-	-	-
	φ' (°) (Peck et al.)	-	-	-	-	-
CPT Interpreted	*Peak φ' (°)*	881	15.6	>40	28.5	28.2
	Su (tsf)		0.06	>4	2.41	1.29
	SPT N_{60} (bpf)		1	82	15	11
	Norm. CPT Tip (Qtn)		2.4	137.3	25.2	15.1

AP-4 CCR- Stacked (Above Elev. 840 ft-msl) - Before Closure (2016)

Property		No. of Data Points (Borings)	Min	Max	Avg	Median
Drilling	*SPT N (bpf)*	11 (6)	2	12	4.5	4
	φ' (°) (Meyerhof)	-	28.3	35.5	30.0	30.0
	φ' (°) (Peck et al.)	-	27.3	30.6	28.0	28.0
	Peak Su/σ'v - VST	2 (2)	0.87	0.89	0.88	0.88
	Residual Su/σ'v- VST	2 (2)	0.30	0.72	0.51	0.51
CPT Interpreted	*Peak φ' (°)*	1899	24.1	>40	35.2	35.1
	Su (tsf)		0.21	>4	2.80	1.91
	SPT N_{60} (bpf)		2	>100	18	13
	Norm. CPT Tip (Qtn)		4.8	754.5	58.7	38.9

AP-4 CCR - Sluiced (Below Elev. 840 ft-msl) - Before Closure (2016)						
Property		***No. of Data Points (Borings)***	***Min***	***Max***	***Avg***	***Median***
Drilling	*SPT N (bpf)*	10 (3)	0	2	1	0
	φ' (°) (Meyerhof)	-	27.5	28.3	27.5	27.5
	φ' (°) (Peck et al.)	-	27	27.3	27.0	27.0
	*Peak Su/σ'v - VST**	28 (4)	0.23	4.33	0.98	0.73
	*Residual Su/σ'v- VST**	28 (4)	0.03	0.50	0.21	0.20
CPT Interpreted	*Peak φ' (°)*	7471	13.2	>40	25.5	24.9
	Su (tsf)		0.03	>4	0.87	0.54
	SPT N_{60} (bpf)		1	>100	7	6
	Norm. CPT Tip (Qtn)		1.4	268.0	11.8	7.6

* Vertical effective stress measured from elevation 840.

AP-4 CCR - Sluiced (Below Elev. 840 ft-msl) - During Closure (2017-2018)						
Property		***No. of Data Points (Borings)***	***Min***	***Max***	***Avg***	***Median***
CPT Interpreted	*Peak φ' (°)*	7869	7.8	>40	26.8	24.9
	Su (tsf)		0.01	>4	0.67	0.37
	SPT N_{60} (bpf)		0	41	7	6
	Norm. CPT Tip (Qtn)		0.1	108.5	9.0	6.4

Stacked / Compacted CCR			
Lab Test	***Strength Type***	***ϕ (deg)***	***c (tsf)***
Direct Shear CPT-32-AP4 5-10 ft	*Peak Effective*	29.1	0
	Post-Peak Effective	29.1	0
Direct Shear CPT-39-AP4 9-10.5 ft	*Peak Effective*	30.4	0
	Post-Peak Effective	30.0	0

Sluiced CCR			
Lab Test	***Strength Type***	***ϕ (deg)***	***c (tsf)***
CU Triaxial BH-CPT-18-AP4 35-37 ft	*Peak Effective*	28.8	0.00
	Peak Total	10.8	0.22
CU Triaxial BH-CPT-19-AP4 35-37 ft	*Peak Effective*	28.4	0.00
	Peak Total	19.9	0.31

Summary of Selected Strength Parameters for CCR Materials				
CCR Material	Drained		Undrained	
	ϕ' (deg)	c' (psf)	ϕ (deg)	Su (tsf) (or c')
Sluiced CCR	24	0	12.4	0.05
Post Liquefied CCR	---	---	Su/σ'vo = 0.08 (min 100 psf)	
Stacked CCR	30	0	24	0.18
Compacted CCR	30	0	24	0.18

4.0 FILL SOILS GEOTECHNICAL DESIGN PROPERTIES (PONDS 1, 2, 3, & 4)

Objective

Develop strength parameters for existing embankment fill materials in the vicinity of AP-1, AP-3 and AP-4.

Lab Testing

Soil samples were collected via standard penetration testing (SPT) from October 26 through 29, 2015 and sent to Golder's geotechnical laboratory for analysis. Two borings (CPT-46 and 49) were drilled within the embankment of AP-4 to depths of 50 and 45 ft-bgs, respectively.

Laboratory properties of fill soil samples collected during SPT are summarized in the table below.

Summary of Geotechnical Testing Data Basic Properties Fill					
Property	***No. of Data Points***	**Min**	**Max**	**Avg**	**Med**
Water Content (%)	4	16.1	20.9	18.9	18.7
Gravel (%)		1.9	7.7	3.5	4.1
Sand (%)		37.9	46.0	43.9	43.0
Fines (%)		49.5	58.4	51.9	52.9
Liquid Limit (LL) (%)		33.1	45.0	39.1	39.2
Plastic Limit (PL) (%)		25.0	29.0	28.0	29.0
Plasticity Index (PI)		4.1	16.0	11.1	12.2
Max Dry Density (pcf)	2	110.6	113.2	111.9	--
Optimum Moisture (%)		14.4	15.1	14.75	--

Strength Correlations for Fine-Grained Material

Strength Data

Strength parameters for the fill soils were evaluated based on in-situ and laboratory testing, summarized in the table below.

The drained strength appears to decrease with depth to approximately 820 feet, where the trend becomes less prevalent (Figure 7). The correlated effective fraction angle varies from approximately 45 to 30° with an average value of 34.1°. A lower bound drained effective friction angle of 30° and cohesion of 50 psf were selected based on laboratory and in-situ testing. These values are based on CPT correlation (Figure 7), laboratory testing, and plasticity correlations. Undrained strengths (Su) vary less with depth than drained strengths (Figure 8). An undrained strength of 1.0 tsf was selected. The CPT correlation is not valid for Su > 4 tsf; these values are excluded from Figure 8.

Summary of Geotechnical Strength Data
Fill Soils

Property		No. of Data Points	Min	Max	Avg	Median
Fill Soils						
Drilling	*SPT N (bpf)*	14	7	28	12	--
	φ' (°) (Meyerhof)		32.5	39.5	35.5	--
	φ' (°) (Peck et al.)		29.0	35.4	30.6	--
CPT Interpreted	*Peak φ' (°)*	2130	17.2	>40	34.1	34.5
	Su (tsf)		0.2	>4	3.7	3.4
	SPT N_{60} (bpf)		4	>100	35	33
	Norm. CPT Tip (Qtn)		0.9	455	49.7	32.9

Correlations from Terzaghi et al. (1996) can be used to estimate friction angles of cohesive soils using laboratory data of plasticity index (PI). NAVFAC Design Manual 7.02 also gives estimated correlations for effective friction angle for various fine-grained material, as referenced in the table below.

For PI < 100: $\phi' = 0.0013(PI)^2 - 0.2717(PI) + 35.876$ $R^2 = 0.9972$ (Terzaghi et al., 1996)

Calculated Strength Based on Plasticity (mean PI = 11)		
Terzaghi et al. Correlations	***Cohesive Soil Peak Friction Angle***	
	Friction Angle (deg), $(\phi'_{fs})_{tan}$	33.0

Other relations can also be used to estimate the fully-softened strength of fine-grained materials, such as that presented by Mesri and Shahien (2003), using plasticity index (see attached Figure 9).

Calculated Strength Based on Plasticity (mean PI = 11)		
Mesri and Shahien Correlations	***Fully Softened***	
	Cohesion, c' (psf)	104.0
	Friction Angle (deg), $(\phi'_{fs})_{tan}$	29.5

Summary of Selected Strength Parameters for Fill Soils			
Material	**Drained**		**Undrained**
	φ' (deg)	c' (psf)	Su (tsf)
Fill Soils	30	100	1.0

Selected Total Unit Weight (pcf)	
Fill Soils	125

5.0 RESIDUAL SOILS GEOTECHNICAL DESIGN PROPERTIES (AP-1, AP-2, AP-3, and AP-4)

Objective

Develop strength parameters for fill material in the vicinity of AP-1, AP-2, AP-3, and AP-4.

All Ash Ponds

Basic properties for the Residuum were evaluated based on laboratory testing and CPT correlations and are summarized in the table below.

For stability and settlement analyses, a unit weight of 125 pcf was selected for the residuum. This selection is based on proctor tests, CPT correlations, and Golder's past experience with residual soils.

Other laboratory tests used to determine strength properties are described below.

Summary of Geotechnical Testing Data
Basic Properties Residuum

Property		No. Tests	Min	Max	Avg	Med
Primary Laboratory Tests						
Depth Range (ft)		-	84.2	114.5	99.3	99.3
Water Content (%)		2	7	28	17	17
Gravel (> 4.75 mm) (%)		1	2	2	2	2
Sand (%)		1	39	39	39	39
Fines (< 0.075 mm) (%)		1	59	59	59	59
Liquid Limit (LL) (%)		1	43	2	2	2
Plastic Limit (PL) (%)		1	28	28	28	28
Plasticity Index (PI)		1	15	15	15	15
Non Plastic Results		0	0 of 1			
Unit Weight (pcf)	CPT interpreted	2130	97	140	125	126

CPT Interpreted Data

Strength parameters for the residuum were evaluated based on in-situ and laboratory testing, summarized in the table below. The residual soils were broken into upper and lower residuum.

A drained friction angle of 30° with a cohesion of 50 psf was selected for the residuum material, both upper and lower. These values are based on in-situ testing (CPT and SPT correlation) and plasticity correlation from laboratory tests. Strength correlations are plotted in Figure 10.

Undrained shear strengths of 0.5 tsf and 1.5 tsf were selected for upper and lower residuum, respectively (Figure 11). These values are based on correlated CPT data and Golder's extensive experience with residual soils.

GOLDER

SUBJECT:	Geotechnical Material Property Package
Project Number:	1777449
Project Name:	Plant McDonough Surface Impoundment Units AP-1, AP-2, and AP-3/4 Closure
Prepared by:	JGM
Checked by:	LJ / LS
Date:	Jul 2018
Reviewed by:	GLH

Summary of Geotechnical Strength Data
Upper Residuum

Property		*No. of Data Points (Borings)*	*Min*	*Max*	*Avg*	*Median*
Residual Soils						
Drilling	*SPT N (bpf)*	7	5	26	12	10
	φ' (°) (Meyerhof)		30.8	39.0	35.5	35.0
	φ' (°) (Peck et al.)		28.3	34.8	30.6	30.0
CPT Interpreted	*Peak φ' (°)*	2172	16.0	>40	30.1	30.1
	Su (tsf)		0.1	>4	2.5	2.2
	SPT N60 (bpf)		3	>100	22	21
	Qtn		0.7	177.3	19.6	13.6

Summary of Geotechnical Strength Data
Lower Residuum

Property		*No. of Data Points (Borings)*	*Min*	*Max*	*Avg*	*Median*
Residual Soils						
Drilling	*SPT N (bpf)*	7	5	26	12	10
	φ' (°) (Meyerhof)		30.8	39.0	35.5	35.0
	φ' (°) (Peck et al.)		28.3	34.8	30.6	30.0
CPT Interpreted	*Peak φ' (°)*	1975	19.0	>40	35.4	36.1
	Su (tsf)		0.2	>4	>4	>4
	SPT N60 (bpf)		2.9	>100	43	33.7
	Qtn		1.3	281.6	43.5	31.7

Summary of Selected Strength Parameters for Residual Soils

Material	**Drained**		**Undrained**
	ϕ' (deg)	c' (psf)	
Upper Residuum	30	50	$Su/\sigma'_{vo} = 0.65$
Lower Residuum	30	100	Su = 1.5 tsf

Selected Total Unit Weight (pcf)

Residuum	125

6.0 ALLUVIUM (POND 1)

Based on borehole data provided in AMEC's 2010 report, alluvial soils exist along the southern portion of the AP-1 dikes. This material was categorized as a low plasticity clay with trace organics and some sandy pockets in drilling logs provided in AMEC's report. Blow counts in this soil were generally found to be around five blows per foot, and the consistency was noted as medium stiff. Golder modeled this soil with the strength parameters shown in the table below base on local experience with similar materials.

Summary of Selected Strength Parameters for Alluvial Soils				
Material	**Unit Weight (pcf)**	**Drained**		**Undrained**
		ϕ' (deg)	c' (psf)	Su (tsf)
Alluvial Soil	115	28	50	0.5

7.0 SUMMARY OF ESTIMATED PROPERTIES

Representative material properties, as shown in table below, have been selected for use in slope stability analysis of temporary (during construction), final (long-term, steady state), and post-liquefaction conditions.

As stated in Section 2.0, strength parameters are based on a combination of CPT-based correlations for peak effective friction angle, borehole blow count data, vane shear data, laboratory shear strength test results, plasticity correlations for fully-softened shear strength, and Golder's experience.

Selected Strength Parameters								
Material		**Total Unit Weight (pcf)**	**Drained Strength**		**Undrained Strength**		**Post-Earthquake Strength**	
			ϕ' (deg)	**c' (psf)**	**ϕ (deg)**	**c (tsf)**	**ϕ (deg)**	**c (tsf)**
Sluiced CCR	**Above GW**	90	24	0	12	0.05	24	0
	Below GW						Su/σ'_{vo} = 0.08 (min 100 psf)	
Stacked / Compacted CCR		110	30	0	24	0.18	30	0
Fill Soils		125	30	100	Su = 1.0 tsf		30	100
Upper Residuum		125	30	50	Su/σ'_{vo} = 0.65		30	50
Lower Residuum		125	30	100	Su = 1.5 tsf		30	100
Alluvium		115	28	50	Su = 0.5 tsf		28	50

8.0 ATTACHMENTS

1 - Lab Data Summary
2 - CPT Data Summary From Field Investigation
3 - Boring Logs From Field Investigation

9.0 REFERENCES

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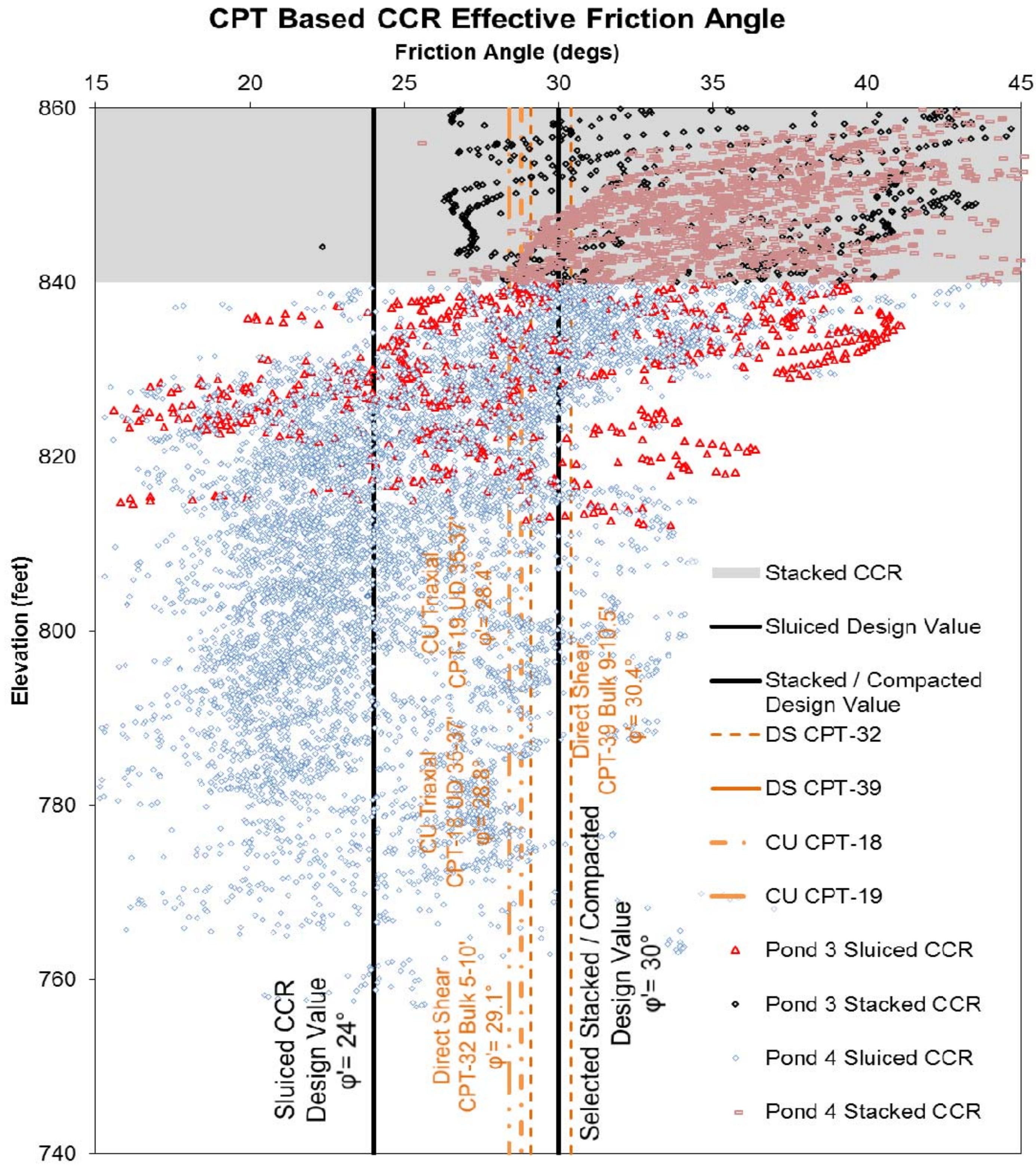
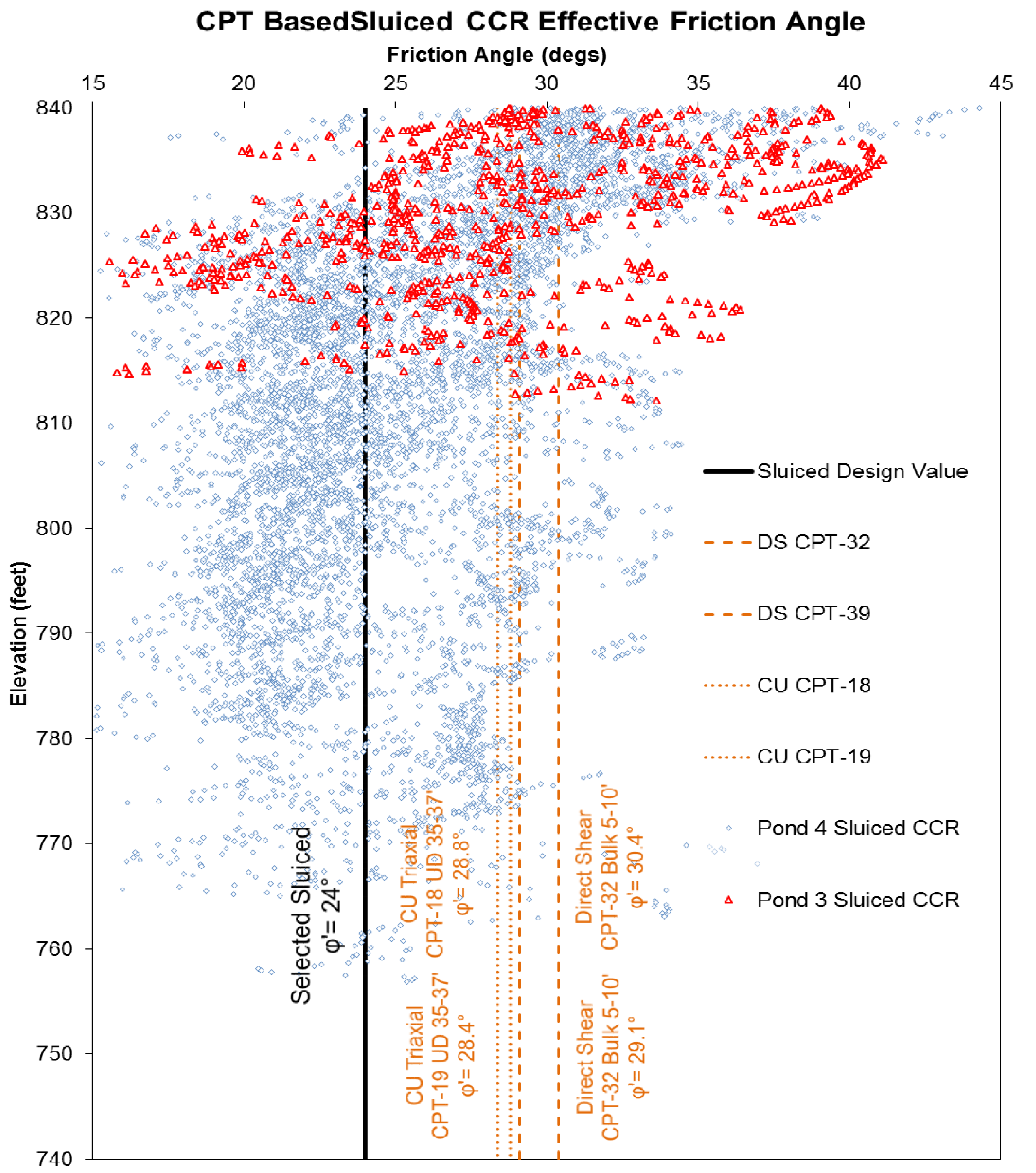


GOLDER Golder Associates Inc.	SCALE	AS SHOWN	**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear & Triaxial)** **Correlation Developed By Mayne And Kulhawy 1996**
	DATE	Apr 2016	
	DESIGN		
	DRAWN	WRP	
Figure 1	CHECK	TPC	
PROJECT No. 1539180 REV. 1	REVIEW	GLH	



GOLDER Golder Associates Inc.	SCALE	AS SHOWN	**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear)** **Correlation Developed By Mayne And Kulhawy 1996**	
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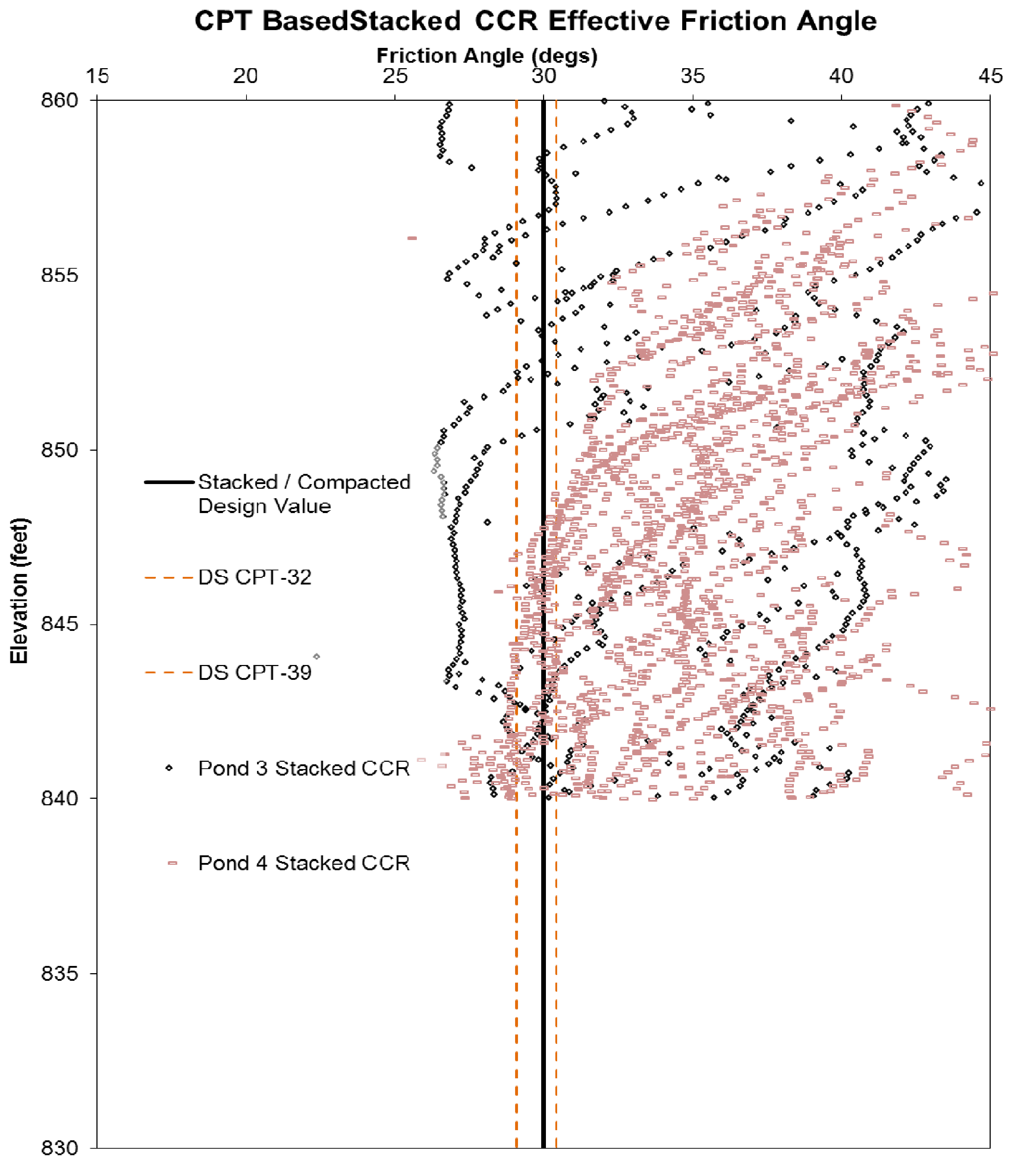
	DESIGN			
	DRAWN	WRP		
Figure 2	CHECK	TPC		
PROJECT No. 1539180	REV. 0	REVIEW	GLH	



GOLDER Golder Associates Inc.		SCALE	AS SHOWN	**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear)** **Correlation Developed By Mayne And Kulhawy 1996**
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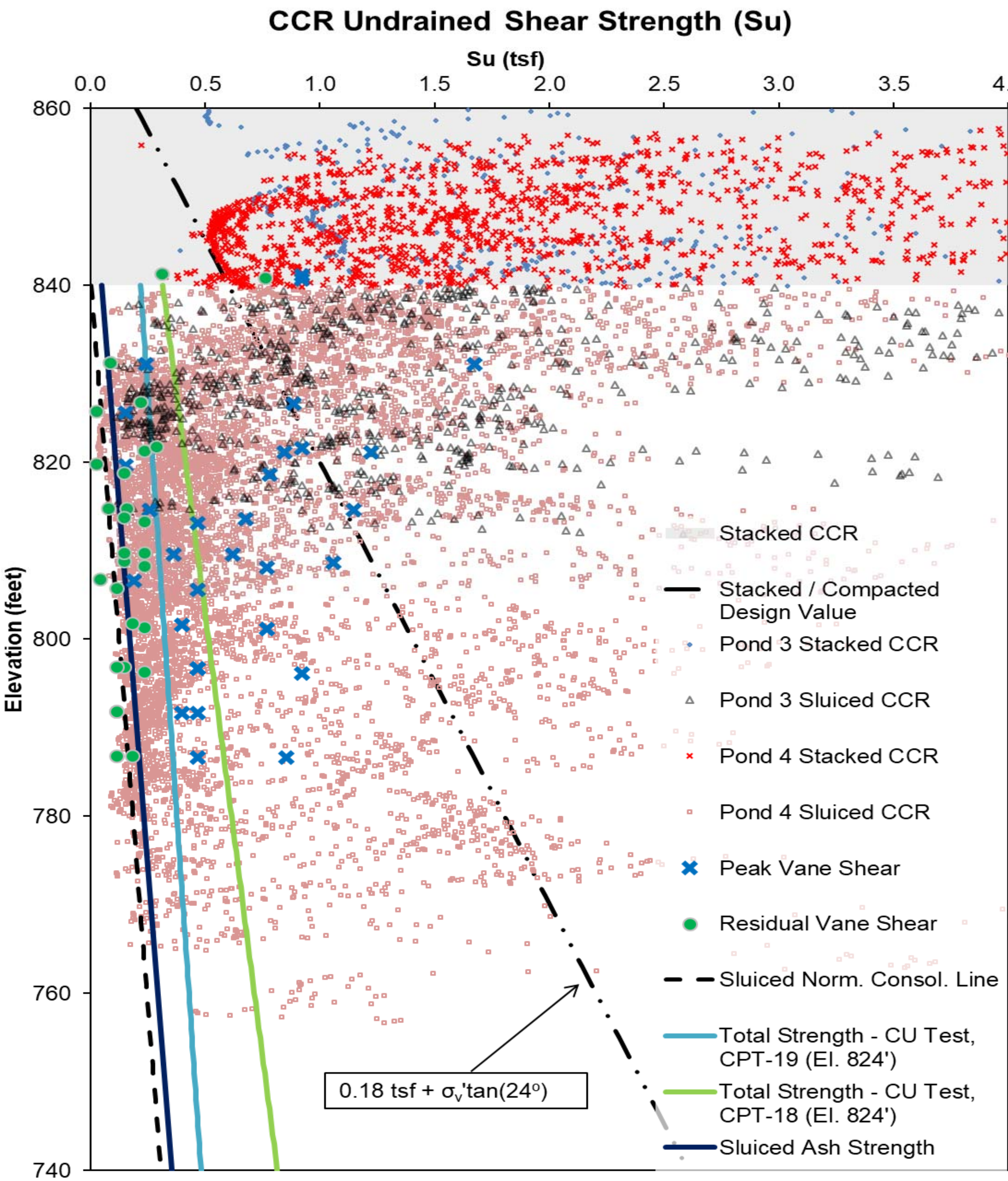
		DRAWN	WRP	
Figure 3		CHECK	TPC	
PROJECT No. 1539180	REV. 1	REVIEW	GLH	



GOLDER

Golder Associates Inc.

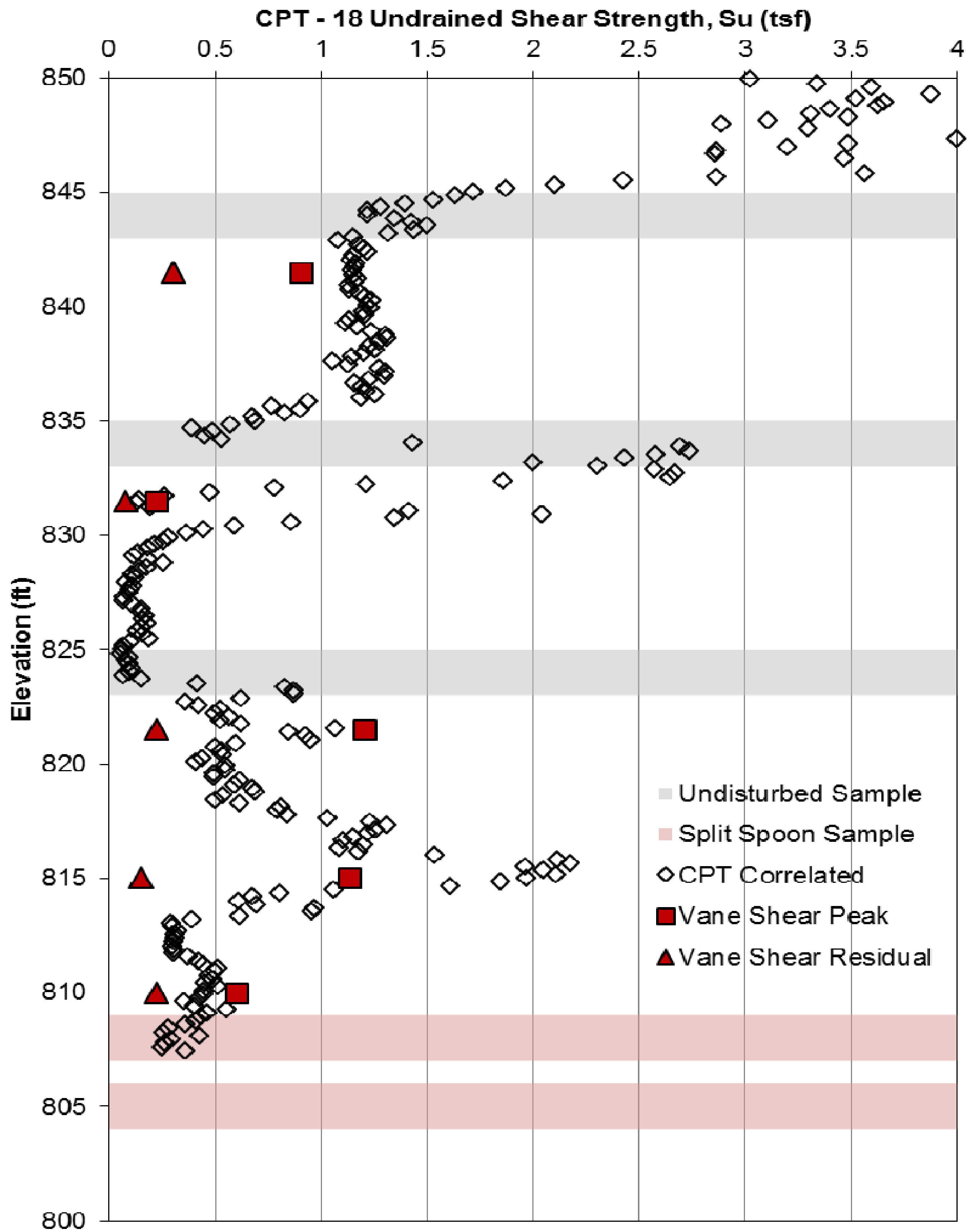
SCALE	AS SHOWN
DATE	Jul 2018
DESIGN	-
DRAWN	WRP
CHECK	JGM
REVIEW	GLH

Measured and Correlated Undrained Shear Strengths

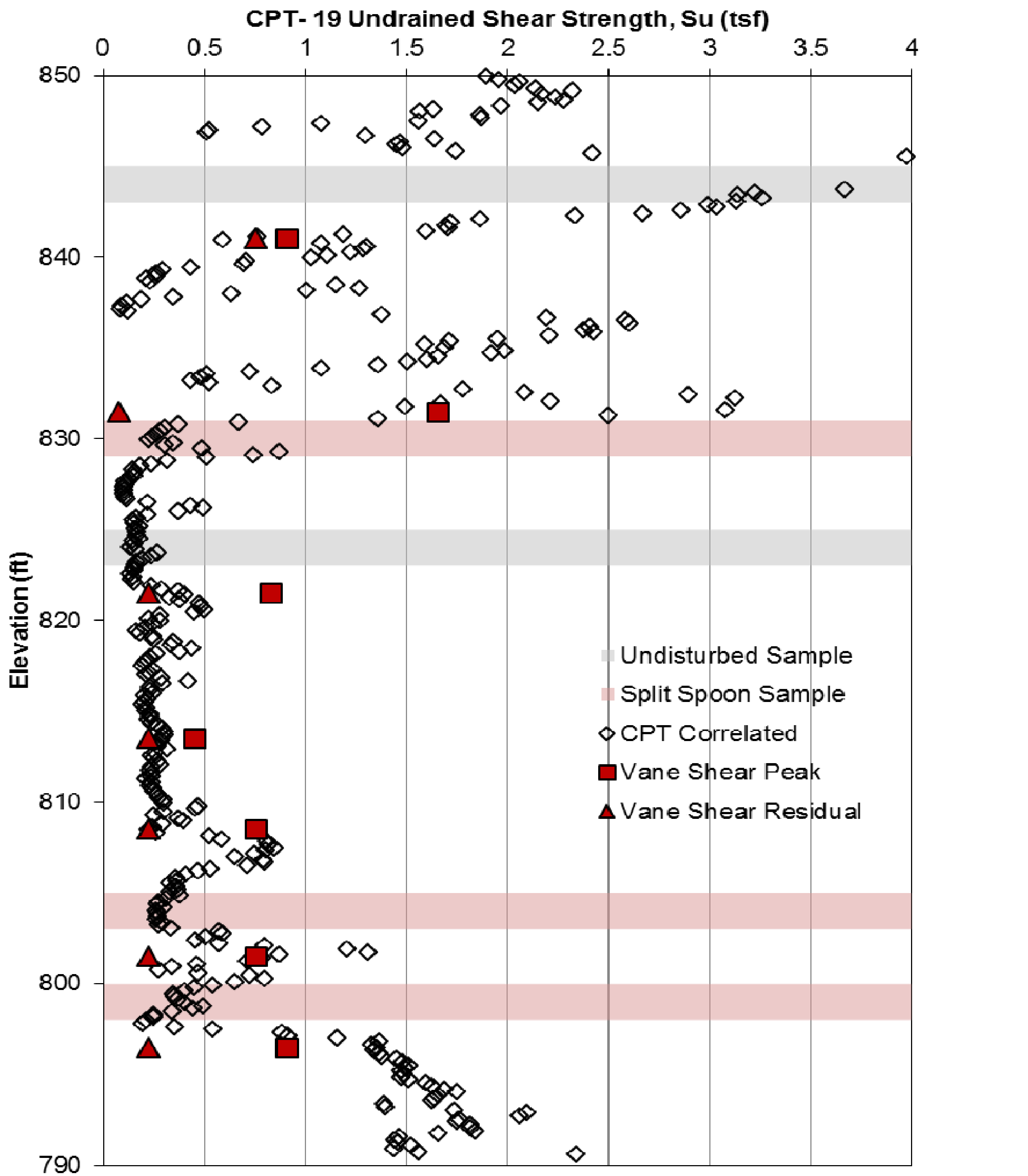
CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996

Figure 4a

PROJECT No. 1539180 REV. 1



GOLDER Golder Associates Inc.	SCALE	AS SHOWN	**Measured and Correlated Undrained Shear Strengths at CPT-18** **CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
	DATE	Apr 2016	
	DESIGN	-	
	DRAWN	WRP	
Figure 4b	CHECK	JGM	
PROJECT No. 1539180 REV. 1	REVIEW	GLH	



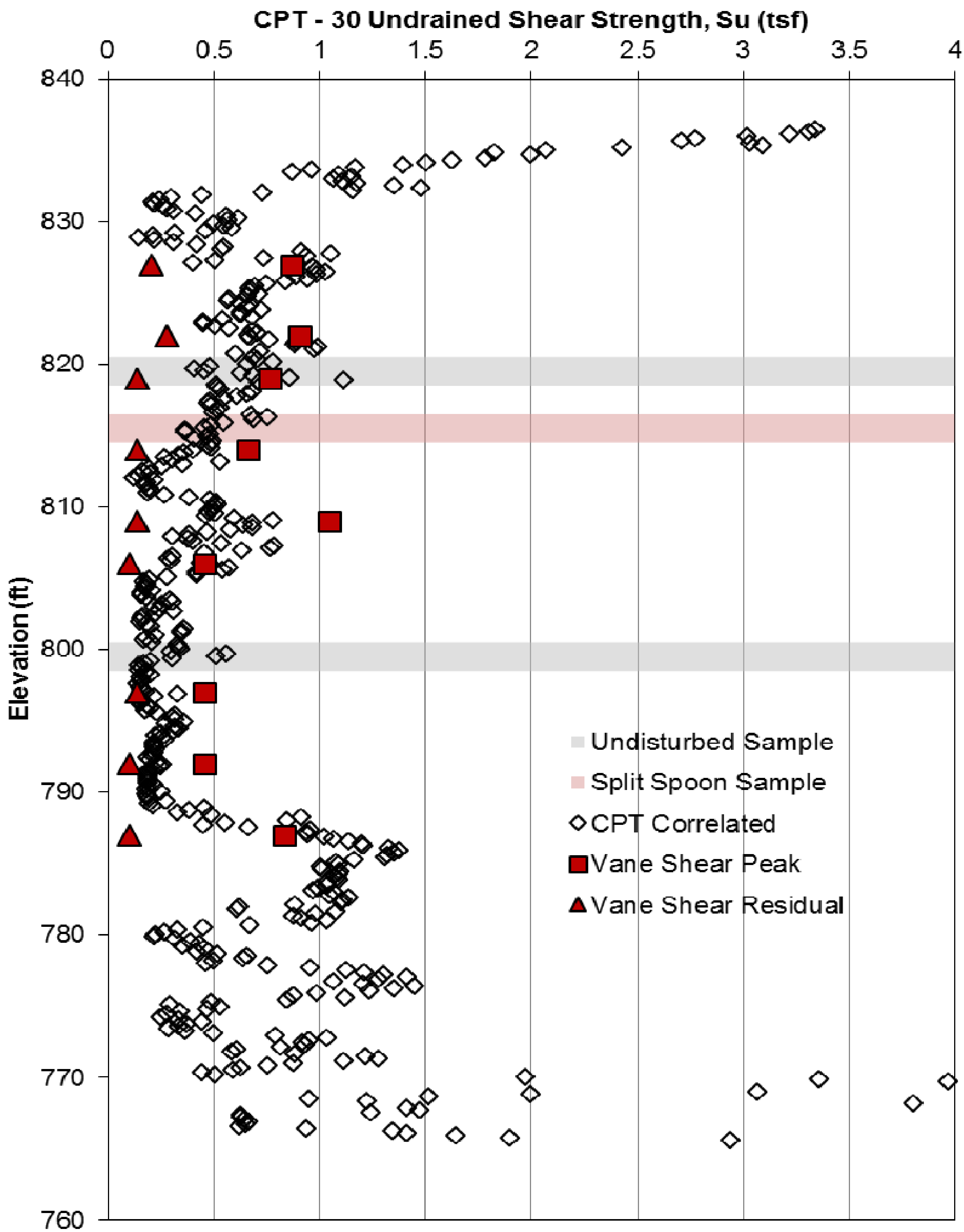
GOLDER

Golder Associates Inc.

Figure 4c		SCALE	AS SHOWN	
		DATE	Apr 2016	
		DESIGN	-	
		DRAWN	WRP	
		CHECK	JGM	
PROJECT No. 1539180	REV. 1	REVIEW	GLH	

Measured and Correlated Undrained Shear Strengths at CPT-19

CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996



GOLDER Golder Associates Inc.	SCALE AS SHOWN		**Measured and Correlated Undrained Shear Strengths at CPT-30** **CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
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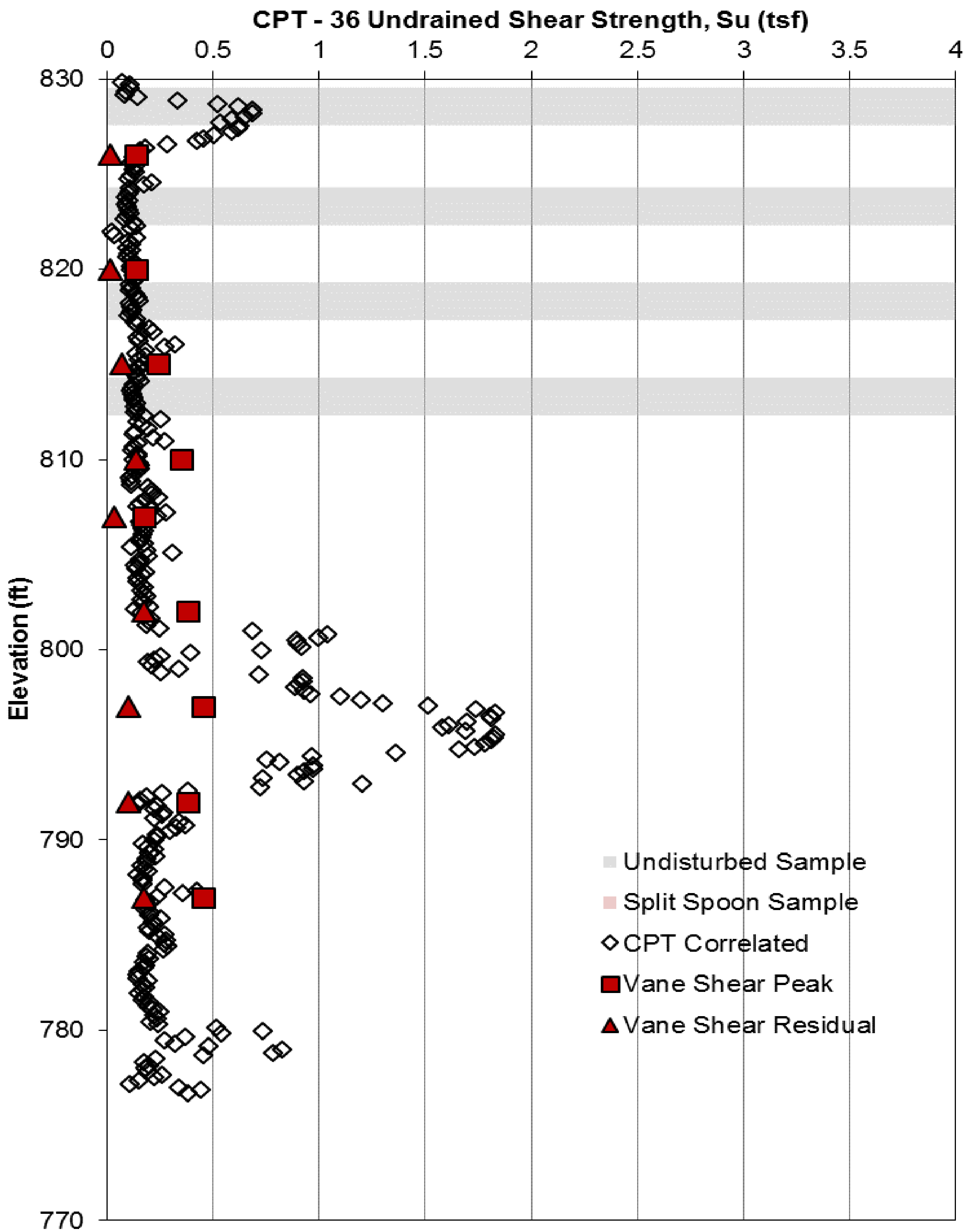
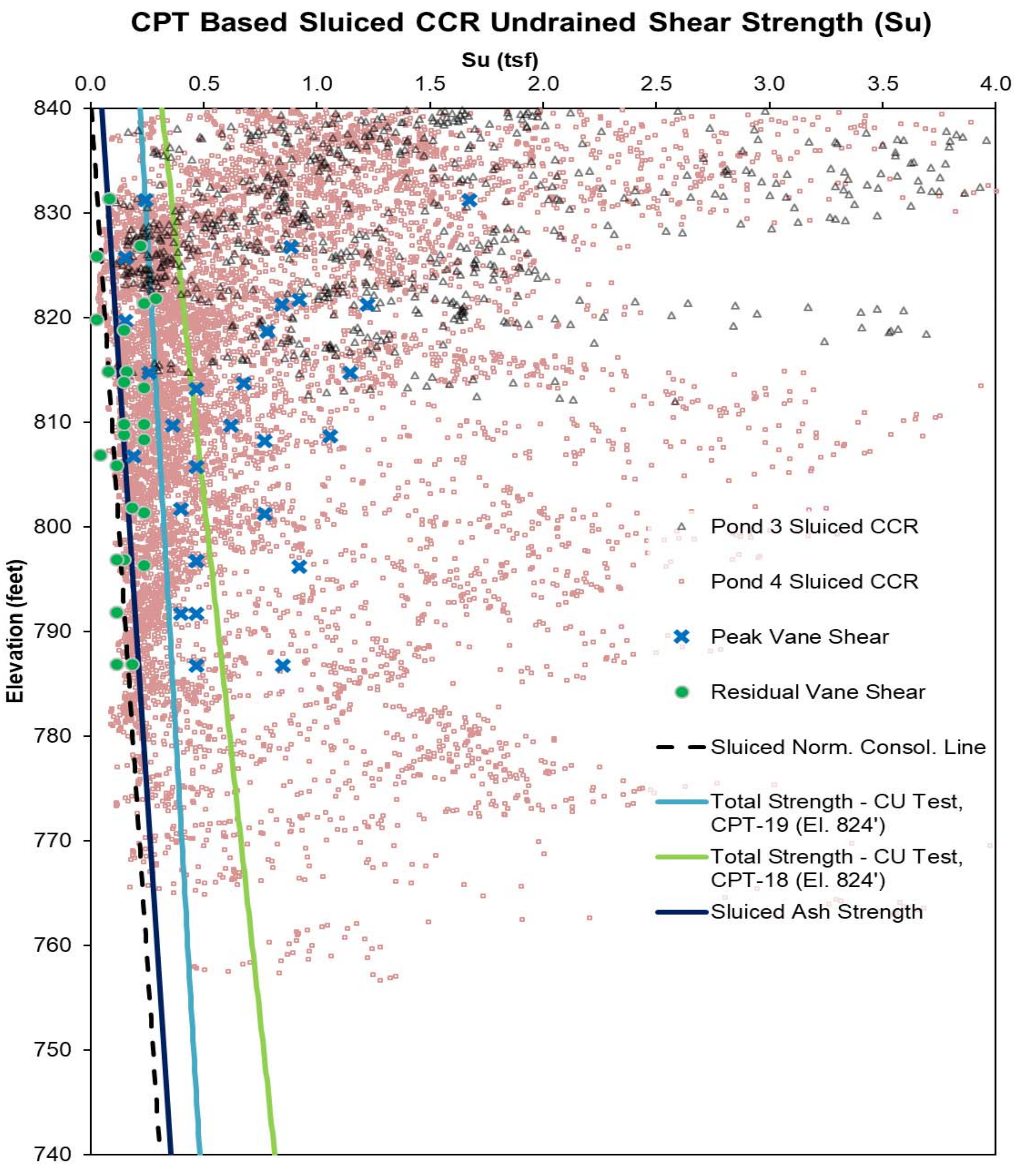
	DESIGN -		
	DRAWN WRP		
Figure 4d	CHECK JGM		
PROJECT No. 1539180	REV. 1	REVIEW GLH	



GOLDER Golder Associates Inc.		SCALE	AS SHOWN
		DATE	Apr 2016
		DESIGN	-
		DRAWN	WRP
Figure 4e		CHECK	JGM
PROJECT No. 1539180	REV. 1	REVIEW	GLH

Measured and Correlated Undrained Shear Strengths at CPT-36

CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996



GOLDER Golder Associates Inc.	SCALE	AS SHOWN	**Undrained Shear Strength Based On CPT Correlation** **Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
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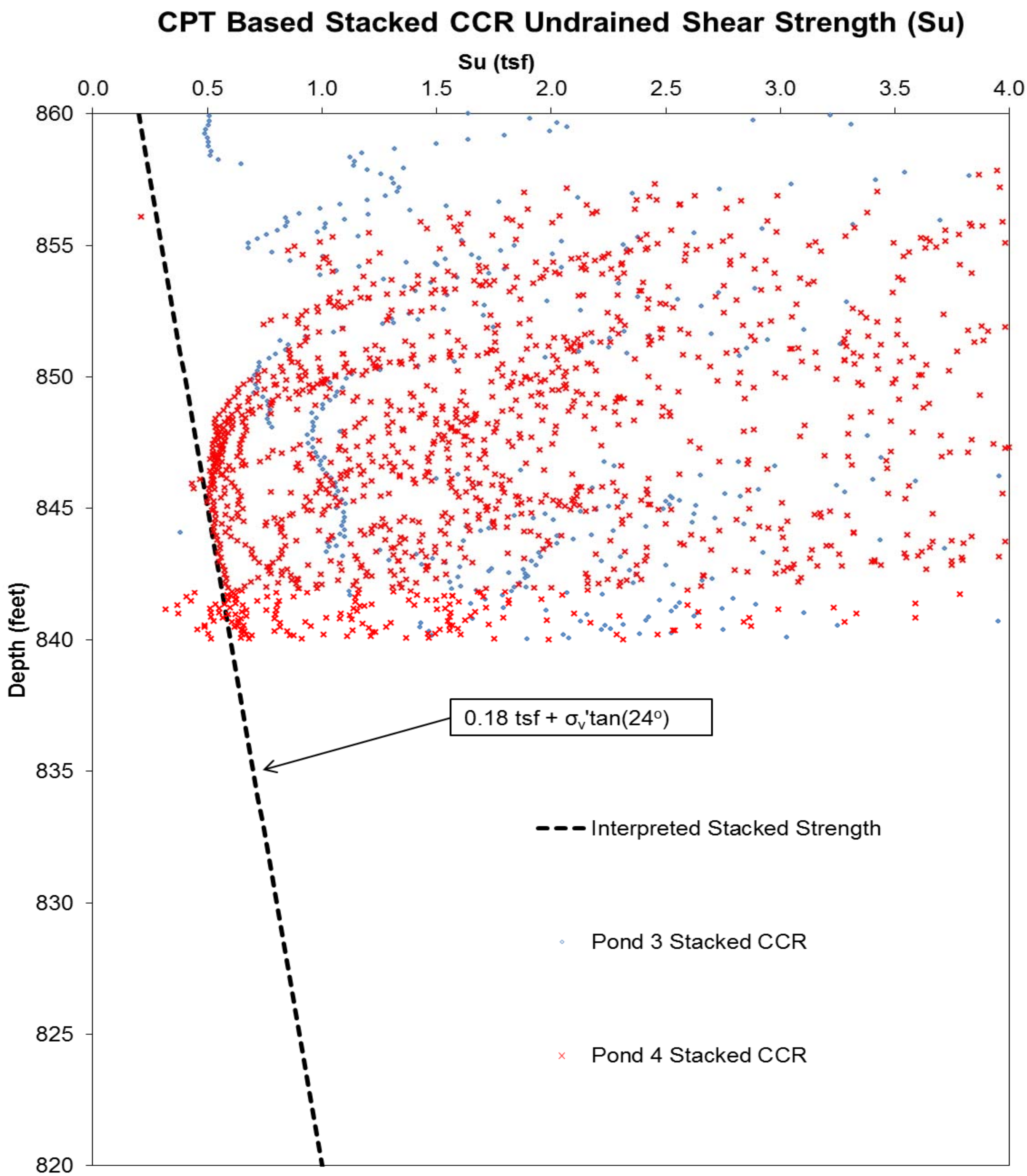
	DESIGN		
	DRAWN	WRP	
Figure 5	CHECK	TPC	
PROJECT No. 1539180 REV. 1	REVIEW	GLH	



GOLDER Golder Associates Inc.		SCALE AS SHOWN	**Undrained Shear Strength Based On CPT Correlation** **Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
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		DESIGN	
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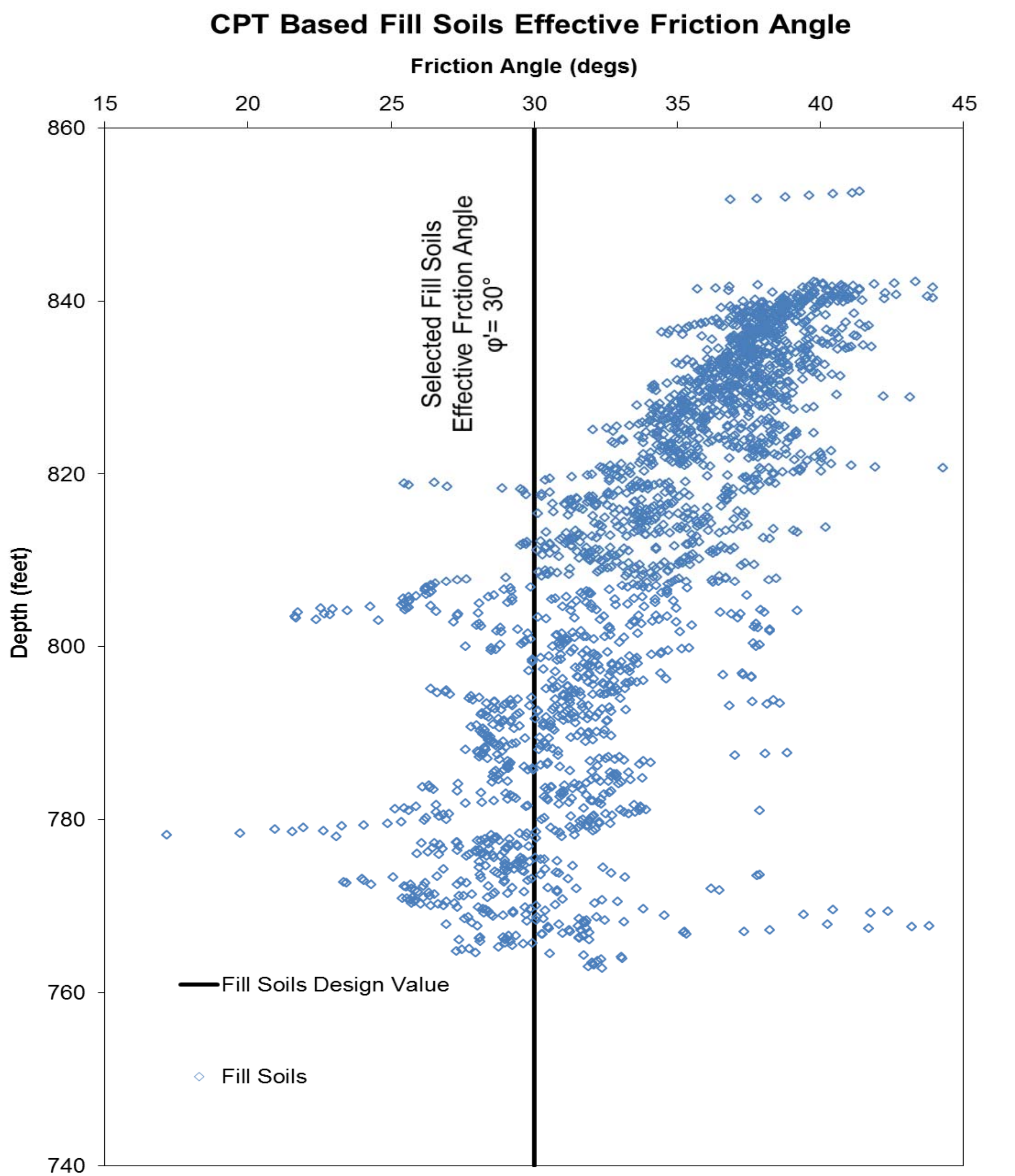
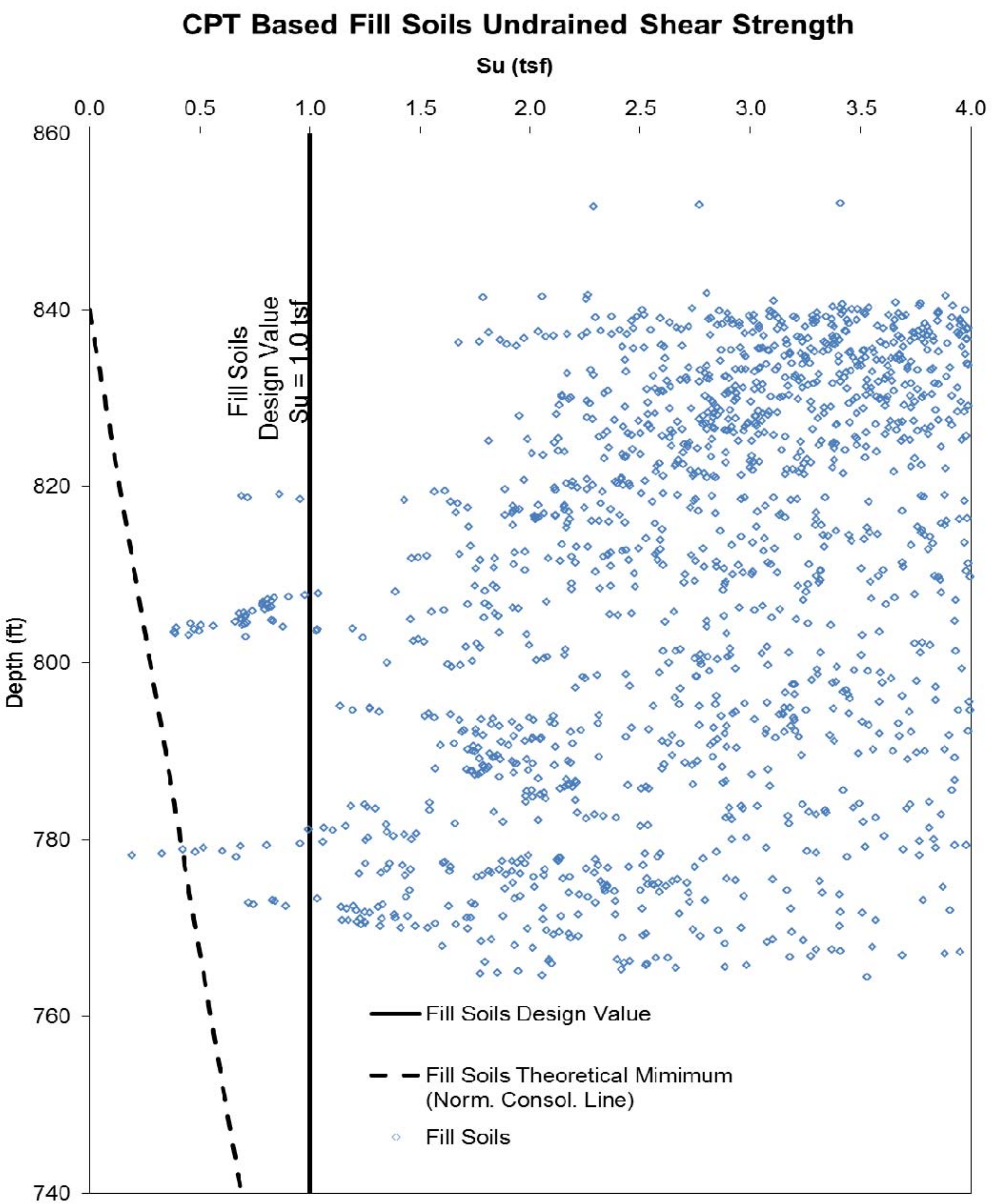
Figure 6		CHECK TPC	
PROJECT No. 1539180	REV. 0	REVIEW GLH	



GOLDER Golder Associates Inc.		SCALE AS SHOWN DATE Dec 2015 DESIGN DRAWN WRP	**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear)** **Correlation Developed By Mayne And Kulhawy 1996**
Figure 7		CHECK TPC	
PROJECT No. 1539180	REV. 0	REVIEW GLH	



GOLDER

Golder Associates Inc.

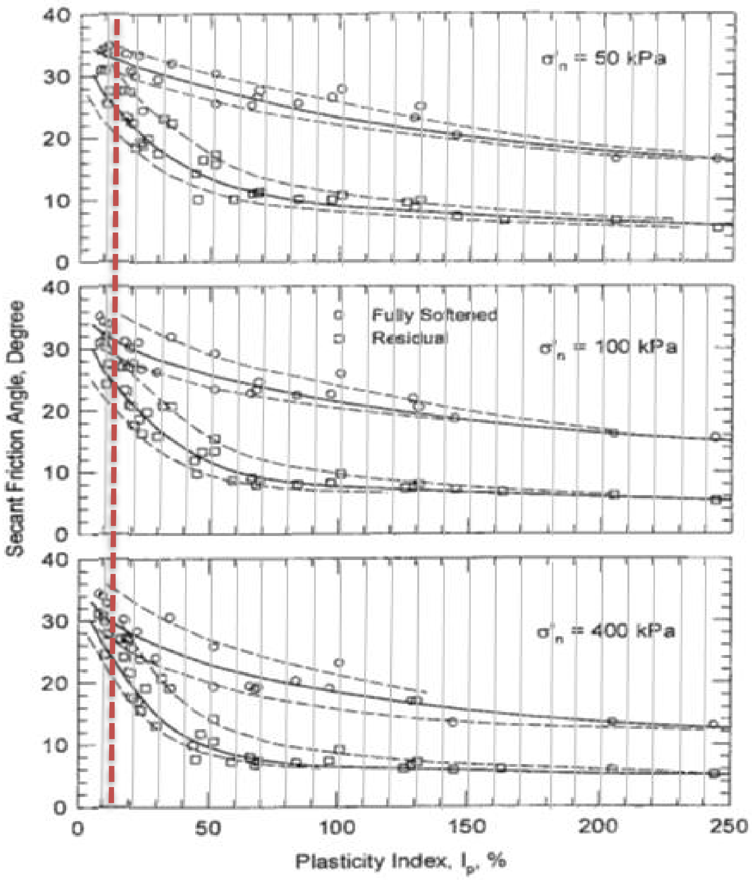
SCALE	AS SHOWN		
DATE	Dec 2015		
DESIGN			
DRAWN	WRP		
CHECK	TPC		
REVIEW	GLH		

Figure 8

PROJECT No.	1539180	REV.	0

Undrained Shear Strength Based On CPT Correlation

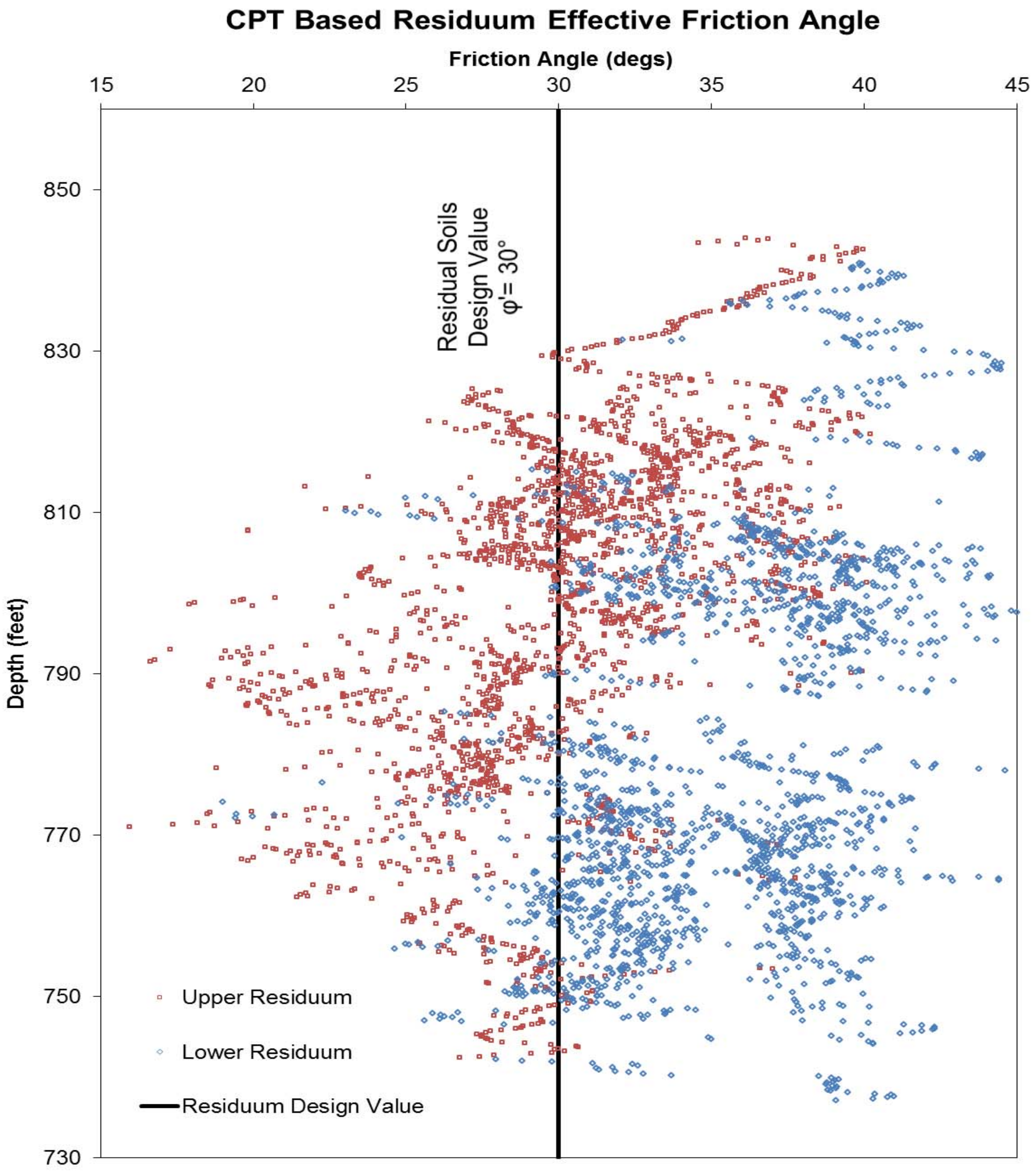
Correlation Based On Robertson et al. 1986 and Lunne et al. 1996



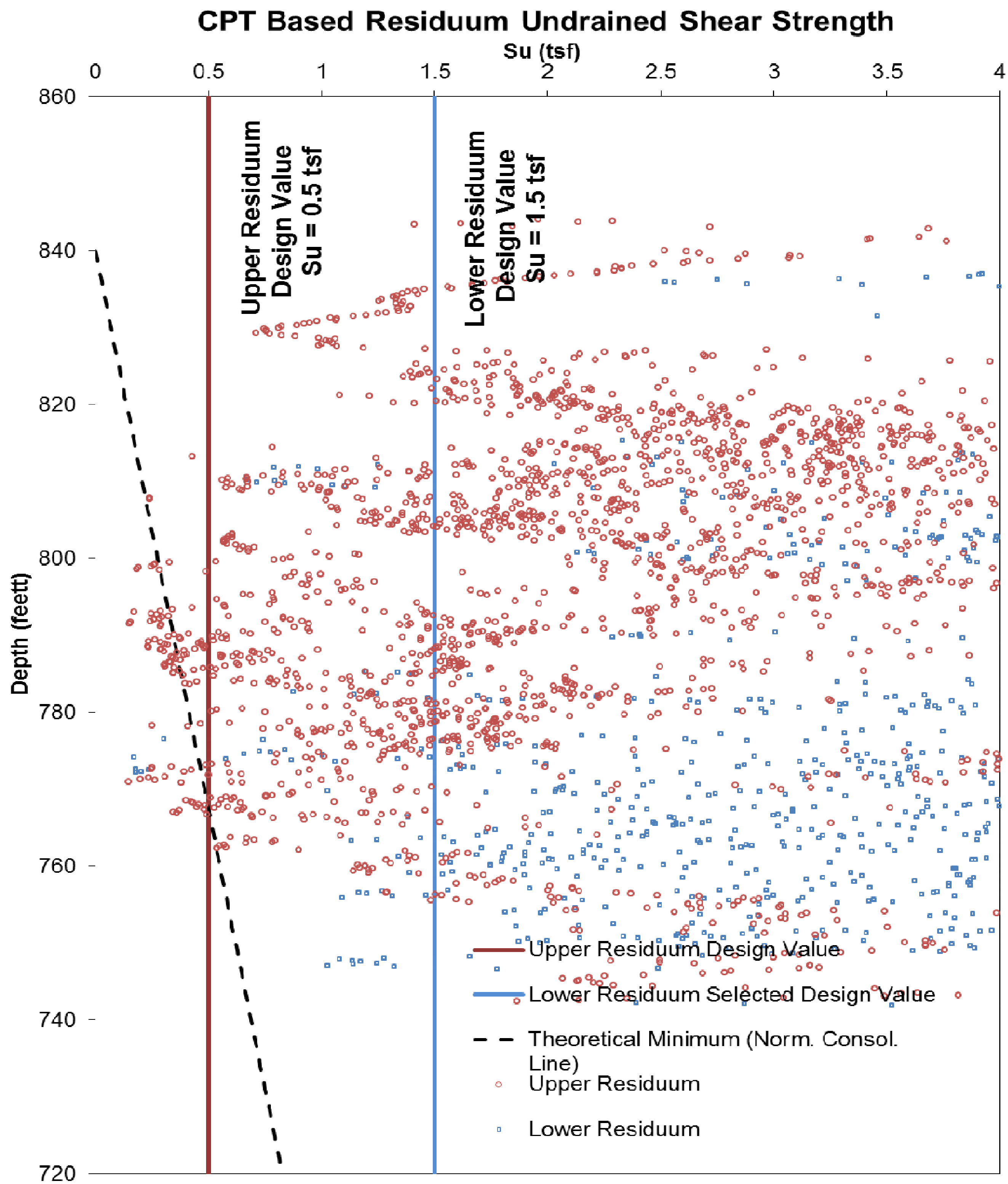
Note - Vertical Lines on Figure are Approximate and Not on Original

ENTER SECANT FRICTION ANGLES OBTAINED FROM MESRI & SHAHIEN 2003 FIG. 2		
OBTAIN RESULTS FOR M-C PARAMETERS		
Effective Normal Stress (kPa)	**Fully Softened Secant Friction Angle (º)**	**Calculated Shear Stress (kPa)**
σ'_n	$(\phi'_{fs})_{sec}$	τ_{fs}
50 kPa	33.0	32 kPa
100 kPa	32.0	62 kPa
400 kPa	30.0	231 kPa
Mohr-Coulomb Tangent ϕ' and c' Data		
Fully Softened	c'	5.0 kPa
	$(\phi'_{fs})_{tan}$	29.5 º

GOLDER Golder Associates Inc.	SCALE AS SHOWN DATE Dec 2015 MADE BY TPC CAD - CHECK WRP REVIEW GLH	TITLE **Plant McDonough Material Property Analysis** **Correlation Between Fully-Softened Friction Angle and Plasticity Index by Mesri and Shahien (2003) (data from Stark and Eid 1994; Eid 1996; and Stark and Eid 1997)**
FILE PROJECT No. 1539180 REV. 0		**Southern Company Services** FIGURE 9



GOLDER Golder Associates Inc.		SCALE	AS SHOWN	**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear)** **Correlation Developed By Mayne And Kulhawy 1996**
		DATE	Dec 2015	
		DESIGN		
		DRAWN	WRP	
Figure 10		CHECK		
PROJECT No. 1539180	REV. 0	REVIEW		



GOLDER Golder Associates Inc.	SCALE	AS SHOWN	**Undrained Shear Strength Based On CPT Correlation** **Correlation Based On Robertson et al. 1986 and Lunne et al. 1997**
	DATE	Dec 2015	
	DESIGN		
	DRAWN	WRP	
Figure 11	CHECK	JGM	
PROJECT No. 1539180 REV. 0	REVIEW	GLH	

1.0 Typical Values and Terminology

Undrained shear strength and effective friction angle correlations based on consistency and density from Peck, Hanson, and Thornburn (1974) are shown in Tables 1 and 2 below.

Table 1 - Fine Grained Soils

Consistency	Field Identification	Undrained Shear Strength (kPa)	
Very Soft	Extrudes between fingers when squeezed	0	12
Soft	Molded by light finger pressure	12	25
Firm	Molded by strong finger pressure	25	50
Stiff	Indented by thumb	50	100
Very Stiff	Indented by thumbnail	100	200
Hard	Difficult to indent with thumbnail	> 200	

Table 2 - Coarse Grained Soils

Density	Field Identification	Dr (%)	ϕ' (Deg)
Very Loose	Easily penetrated with shovel handle	<20	< 29
Loose	Easily penetrated with 1/2 inch rebar pushed by hand. Easily excavated with hand shovel.	20 - 40	29 - 30
Compact	Easily penetrated with 1/2 inch rebar driven by 5 lb. hammer. Difficult to excavate with hand shovel.	40 - 60	30 - 36
Dense	Penetrated 1 foot with driven rebar. Must be loosened with pick to hand excavate.	60 - 80	36 - 41
Very Dense	Penetrated only a few inches with driven rebar. Very difficult to excavate even with pick.	> 80	> 41

D_r (%) = Relative Density = $(e_{max} - e) / (e_{max} - e_{min})$ * 100%.

ϕ' (Deg) = Effective Friction Angle

Effective friction angle correlations based on SPT N-values from Peck et al. and Meyerhof are shown in Table 3 and Figure 1 below.

Table 3: Estimation of Granular Material Effective Friction Angle Based on SPT N-Value (EPRI, 1990)

N-Value(blows/ft)	Approximate ϕ' (deg)	
	Peck et al.	Meyerhof
0 to 4	< 28	< 30
4 to 10	28 to 30	30 to 35
10 to 30	30 to 36	35 to 40
30 to 50	36 to 41	40 to 45
> 50	> 41	> 45

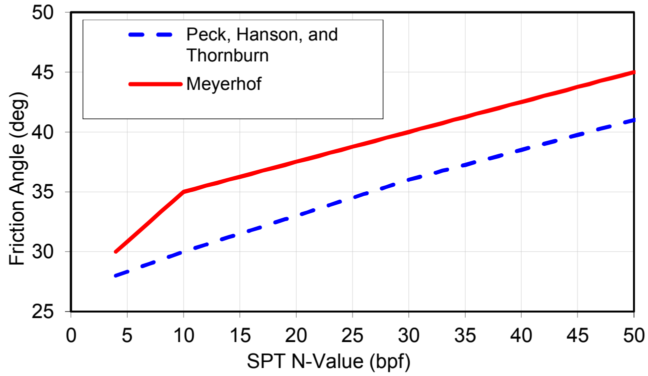


Figure 1: Graphical Representation of Table 1

Correlations from Terzaghi et al. (1996) can be used to estimate friction angles of cohesive soils using laboratory data of plasticity index (PI). NAVFAC Design Manual 7.02 also gives estimated correlations for effective friction angle for various fine-grained material, as referenced in the table below.

For PI < 100: $\phi' = 0.0013(PI)^2 - 0.2717(PI) + 35.876$ $R^2 = 0.9972$ (Terzaghi et al., 1996)

Table 4: Estimation of effective friction angle based on USCS (for Compacted Fine-Grained Material) (NAVFAC, 1986)		
USCS	Soil Type	Effective Friction Angle (deg)
ML	Inorganic silts and clayey silts	32
ML-CL	Mixture of inorganic silt and clay	32
CL	Inorganic clays of low to medium plasticity	28
MH	Inorganic clayey silts, elastic silts	25
CH	Inorganic clays of high plasticity	19

2.0 Cone Penetration Testing

The CPT soundings in this study were completed with a 10 cm^2 area (3.57 cm diameter) piezocone using operating procedures in accordance with ASTM Standard D-5778. Pore pressure filter elements, made of porous plastic, were saturated under a vacuum using silicone oil as the saturating fluid. The pore pressure element was six millimeters (mm) thick and was located immediately behind the tip (the u_2 location) for all soundings. The cone was advanced using a WWC-707 drill rig mounted on tracks owned and operated by CONETEC.

Raw CPT data measurements of the following parameters were recorded at a rate of 1 measurement every 1 mm of penetration:

- tip stress (qc)
- sleeve friction (fs)
- pore pressure (u2)
- Dual Axis Inclination (Ix & Iy)
- Temperature (T)
- rate of penetration (v).

Golder used the CPT data processing software CPT-It by Geologismiki to provide initial processing of the raw data into engineering units of the standard CPT presentation parameters:

- corrected cone resistance (qt)
- friction ratio (Rf)
- pore pressure (u2)
- soil behavior type index (Ic SBT)
- soil behavior type (SBT) based on the Robertson (2010) soil classification scheme
- normalized cone tip resistance (Qtn)
- normalized friction ratio (Fr)
- normalized pore pressure ratio (Bq)
- normalized soil behavior type index (Ic)
- normalized soil behavior type (SBTn) based on the Robertson (1990) soil classification scheme

Equations (1) and (2) present the relationships for qt and Rf:
qt = qc + u2 (1-a) (1)
Rf = fs / qt x 100% (2)

The parameter ‘a’ in equation (1) is known as the end area ratio of the cone penetrometer device. This parameter represents the ratio of the cross-sectional area of the tip load cell element along the shaft to that of the projected cone area. It corrects the measured cone tip stress (qc) to account for the effects of water pressure acting unequally on the geometry of the cone tip. For the 10 cm2 cone, the value of ‘a’ is 0.8, as provided by the manufacturer’s calibration.

Equation (3) presents the un-normalized relationship used to determine Ic SBT presented in Attachment 2,
Ic SBT = [(3.47 – log (qt / pa))2 + (log Rf + 1.22)2]0.5 (3)

Equations (4) to (6) present the normalized relationships used to generate the SBTn and Ic values presented in Attachment 2,
Qtn = ((qt – σv0) / pa) x (pa /σv0′)n (4)
Fr = fs / (qt - σv0) x 100% (5)
Bq = (u2 – u0) / (qt - σv0) (6)

where:
n = 0.381 x Ic + 0.05 x ((σv0′) / pa) – 0.15
Ic = [(3.47 – log (Qtn))2 + (log Fr + 1.22)2]0.5

The parameters [σv0 and σv0′] in the above equations represent the total and effective vertical stress at a given measurement location, respectively. The parameters [u2 and u0] in the above equations are the dynamic pore pressure measured during CPT penetration and the static equilibrium pore pressure at a given measurement location, respectively. The parameter pa in the above equations is the atmospheric pressure, i.e ~101 kPa = 1.06 tsf.

Attachment 2 to this report provides plots of the above-described standard and normalized parameters for each of the completed CPT soundings providing a near continuous profile of the encountered ground conditions.

Prior to performing each CPT, the piezocone tip and sleeve were removed from the piezocone housing, cleaned, lubricated and reassembled with a new pore pressure filter element. Each pore pressure filter element was pre-saturated (free of air). A latex membrane was placed on the piezocone tip after piezocone cleaning and lubrication to avoid de-saturation of the pore pressure element while waiting for the start of each test and was removed prior to performing the test. The potentiometer and piezocone instruments were connected to a data control box where the measurements were saved for post-processing and also viewed real-time on a ruggedized field computer

References

FHWA (1998), Training Course in Geotechnical and Foundation Engineering - Rock Slopes, Publication No. FHWA HI-99-007

Mesri, G. and Shahien, M. (2003) "Residual Shear Strength Mobilized in First-Time Slope Failures,", JGGE, 129, 1, 12-31.

Gregg Drilling, "Guide to Cone Penetration Testing", 6th Edition 2015.

Kulhawy, F. H. and Mayne, P.W. (1990). Manual on Estimating Soil Properties for Foundation Design, EL-6800, Electric Power Research Institute (EPRI).

Terzaghi, K., Peck, R.B., Mesri, G. (1996). Soil Mechanics in Engineering Practice, 3rd Edition, John Wiley & Sons, New York. Figure 19.7

Naval Facilities Engineering Command (NAVFAC) (1986). Design Manual 7.02 Foundations and Earth Structures.

APPENDIX B

Stability Analysis Figures for AP-1

LEGEND

EXISTING CONTOURS

SOUTHERN COMPANY BOREHOLES

REFERENCES

1. THE EXISTING TOPOGRAPHY SHOWN EVERYWHERE ELSE WAS PROVIDED BY SOUTHERN COMPANY SERVICES AS AN INTERIM CONSTRUCTION PROGRESS SURVEY. FLOWN ON 04-15-17 USING LiDAR.

2. SOUTHERN COMPANY BOREHOLES COMPLETED IN JANUARY 2009.

0 100 200
1" = 100' FEET

CLIENT
GEORGIA POWER COMPANY

Georgia Power

PROJECT
PLANT MCDONOUGH
SAFETY FACTOR ASSESSMENT

TITLE
CCR UNIT AP-1 - STABILITY SECTIONS PLAN

CONSULTANT

YYYY-MM-DD	2023 APRIL
DESIGNED	LJ
PREPARED	RMS
REVIEWED	JGM/ HJ / LS
APPROVED	GLH

PROJECT NO. 1777449 REV. SHEET 1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D

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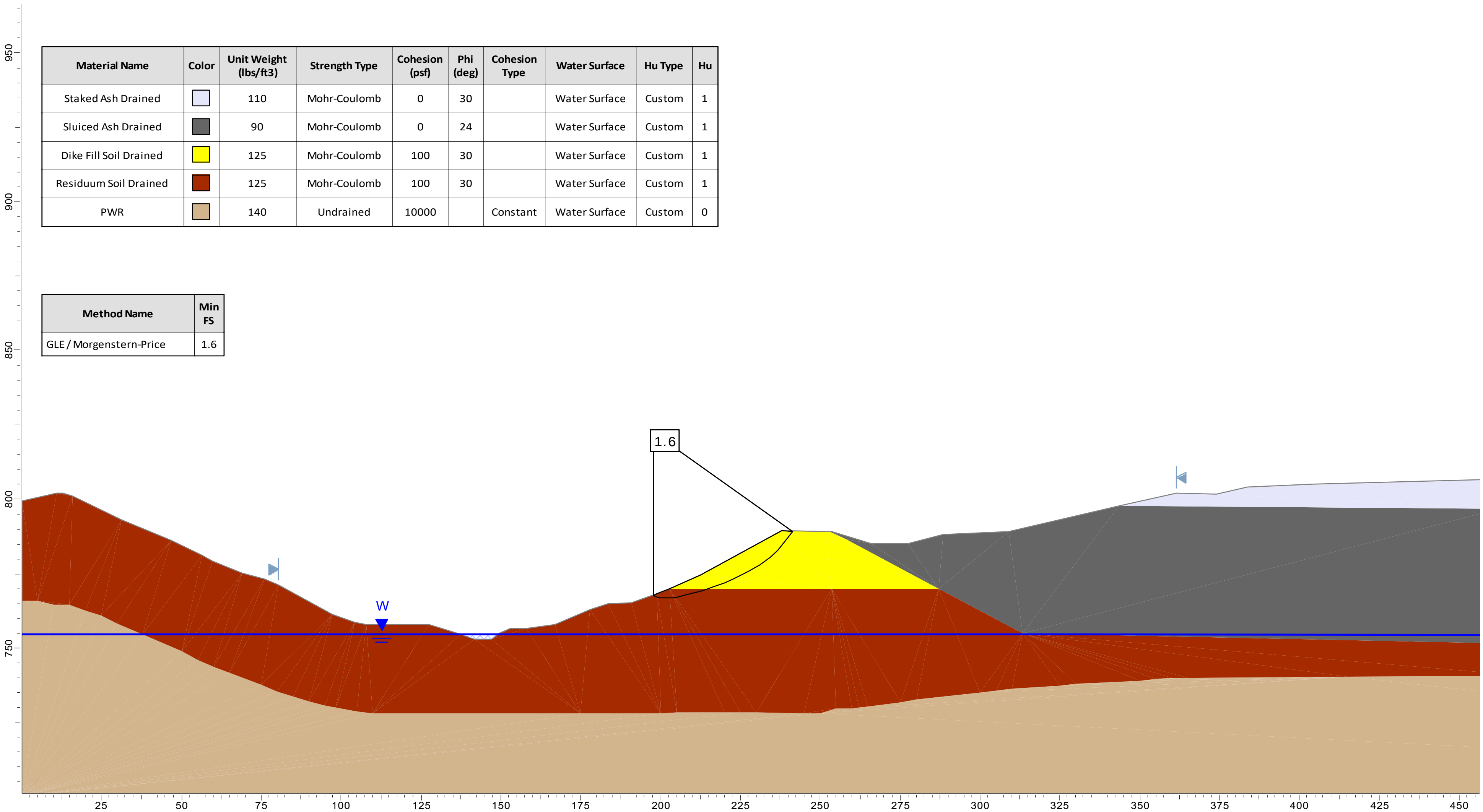
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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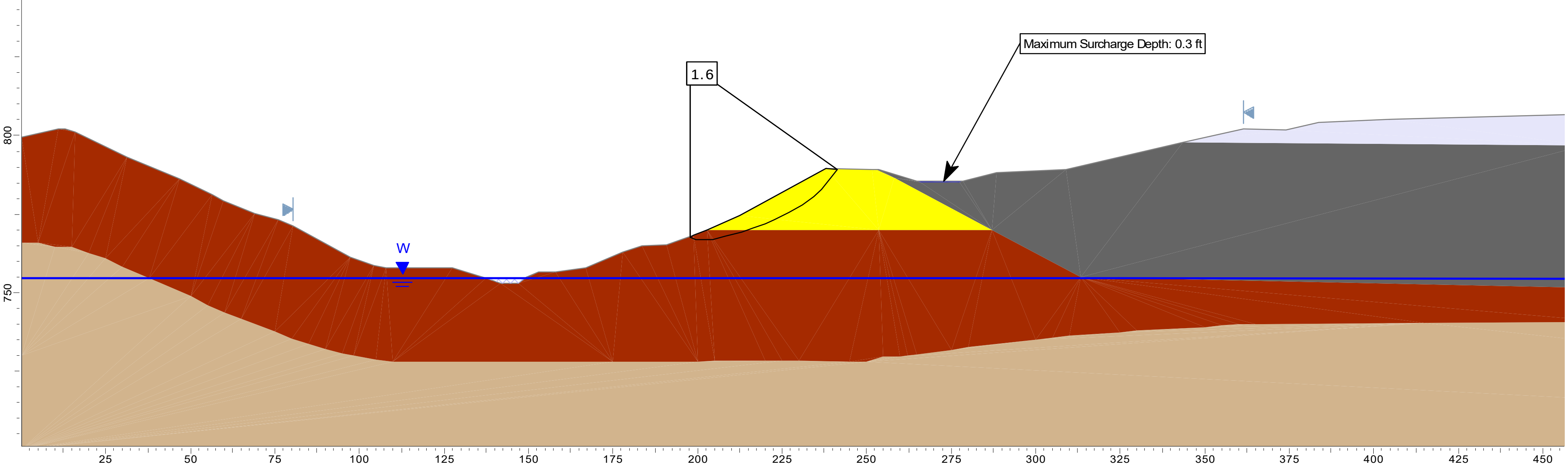
DATE	April 2023	TITLE	Section A-A Long Term, Storage Pool
MADE BY	LJ		
CAD	-		
CHECK	HJ	CLIENT	Georgia Power Company
REVIEW	GLH	FIGURE	2(a)

FILE STABILITY

PROJECT No. 1777449 REV. 0

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0
Water		62.4	No strength				Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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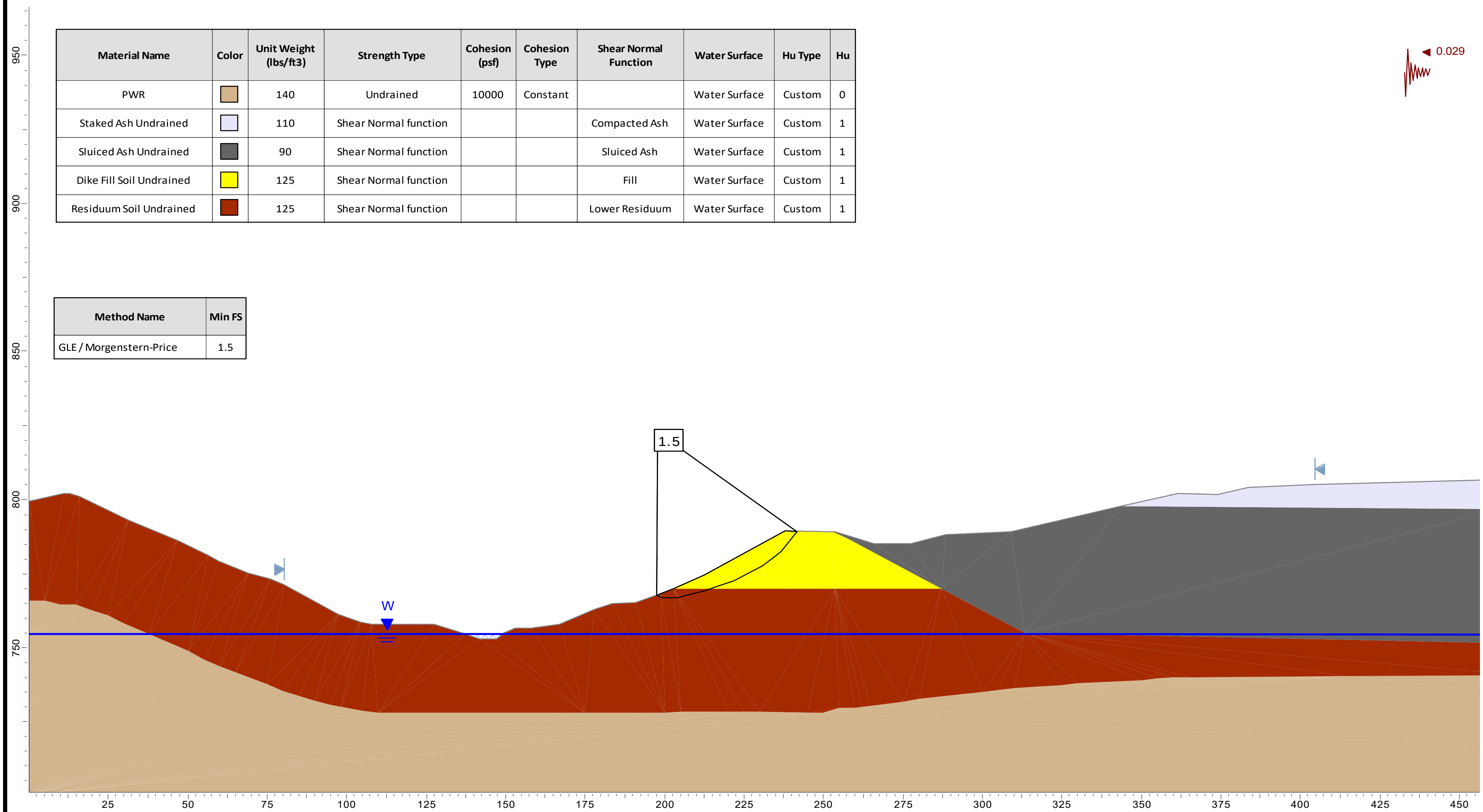
	DATE April 2023	TITLE	Section A-A Surchage Pool
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK HJ	CLIENT Georgia Power Company	FIGURE 2(b)
PROJECT No. 1777449 REV. 0	REVIEW GLH		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Cohesion Type	Shear Normal Function	Water Surface	Hu Type	Hu
PWR		140	Undrained	10000	Constant		Water Surface	Custom	0
Staked Ash Undrained		110	Shear Normal function			Compacted Ash	Water Surface	Custom	1
Sluiced Ash Undrained		90	Shear Normal function			Sluiced Ash	Water Surface	Custom	1
Dike Fill Soil Undrained		125	Shear Normal function			Fill	Water Surface	Custom	1
Residuum Soil Undrained		125	Shear Normal function			Lower Residuum	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.5



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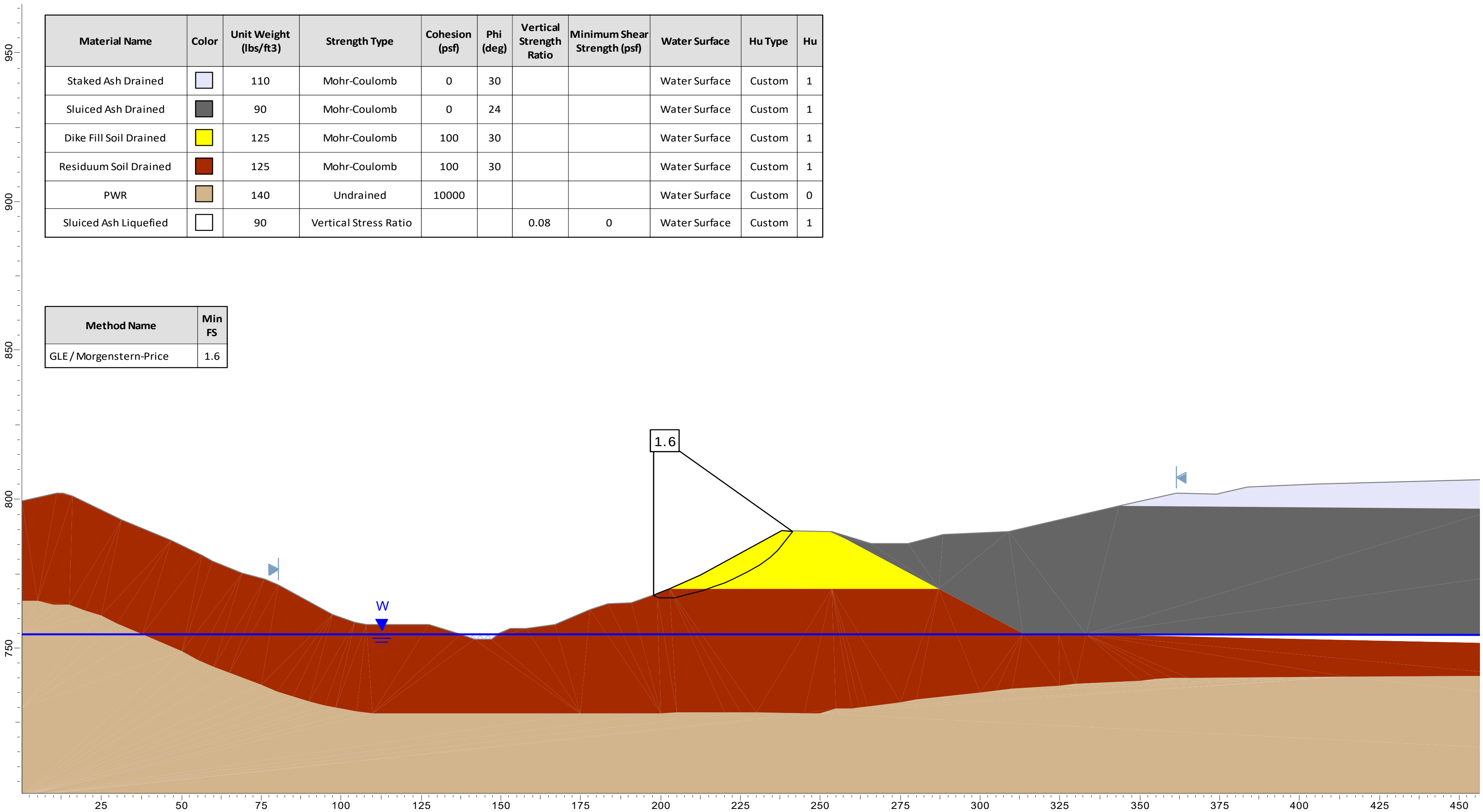
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	2(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30			Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24			Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30			Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30			Water Surface	Custom	1
PWR		140	Undrained	10000				Water Surface	Custom	0
Sluiced Ash Liquefied		90	Vertical Stress Ratio			0.08	0	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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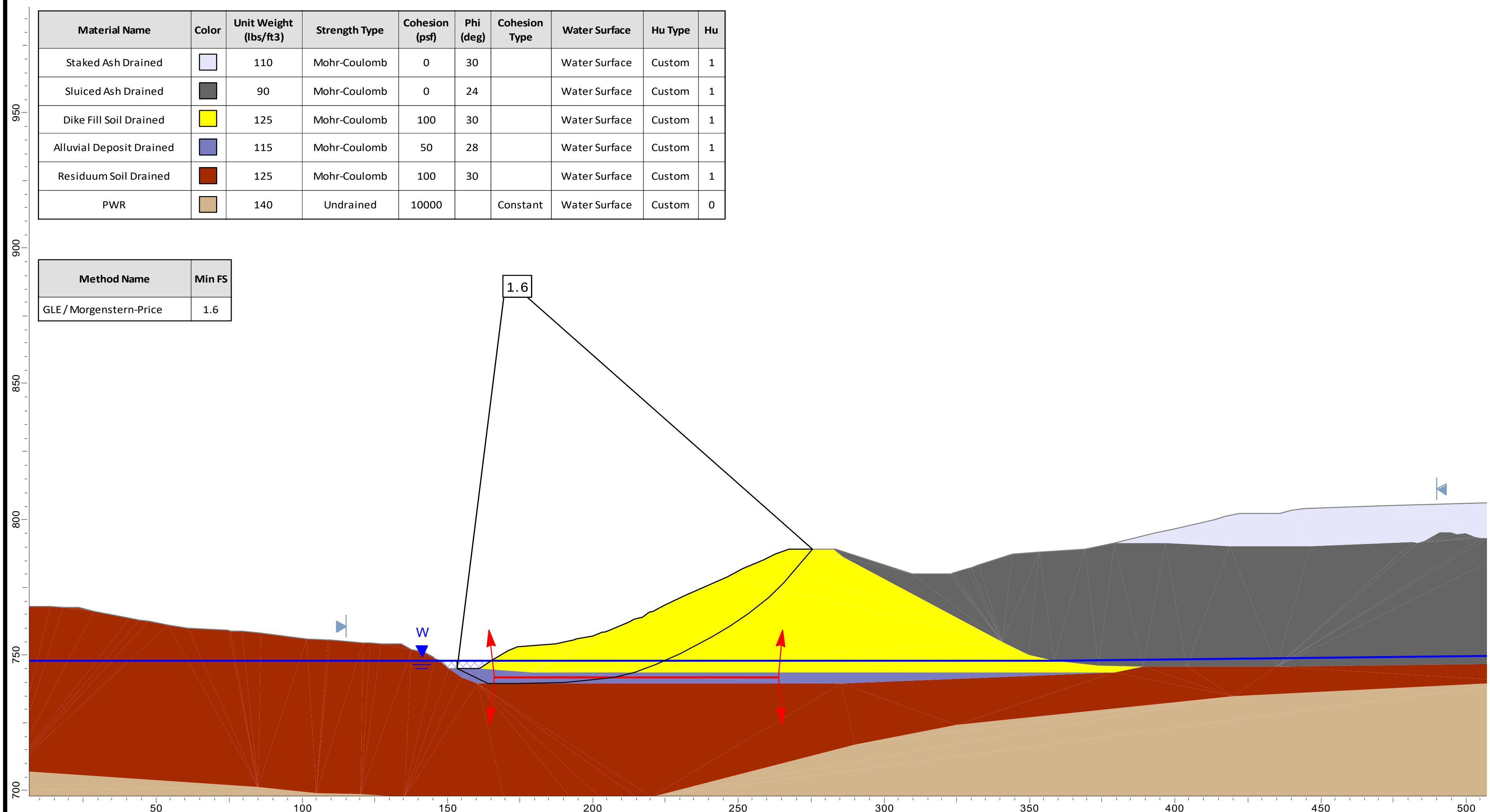
MADE BY	LJ		
CAD	-		
FILE: STABILITY	CHECK: HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW: GLH	FIGURE	2(d)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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	DATE April 2023	TITLE	Section B-B Long Term, Storage Pool
	MADE BY LJ		
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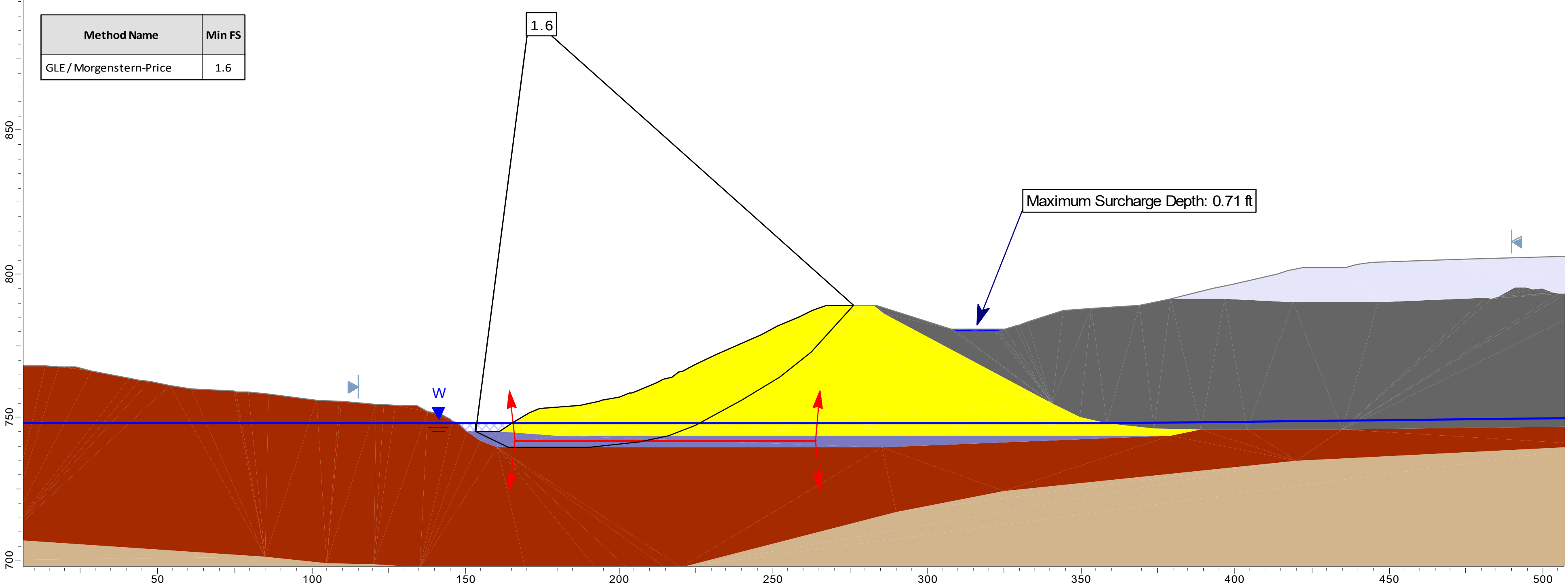
FILE STABILITY	CHECK HJ	CLIENT	Georgia Power Company — FIGURE 3(a)
PROJECT No. 1777449 REV. 0	REVIEW GLH		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0
Water		62.4	No strength				Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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DATE	April 2023
MADE BY	LJ
CAD	-
CHECK	HJ
REVIEW	GLH

FILE STABILITY

PROJECT No. 1777449 REV. 0

PROJECT **State CCR Permitting Services - MCD Pond 1**

TITLE

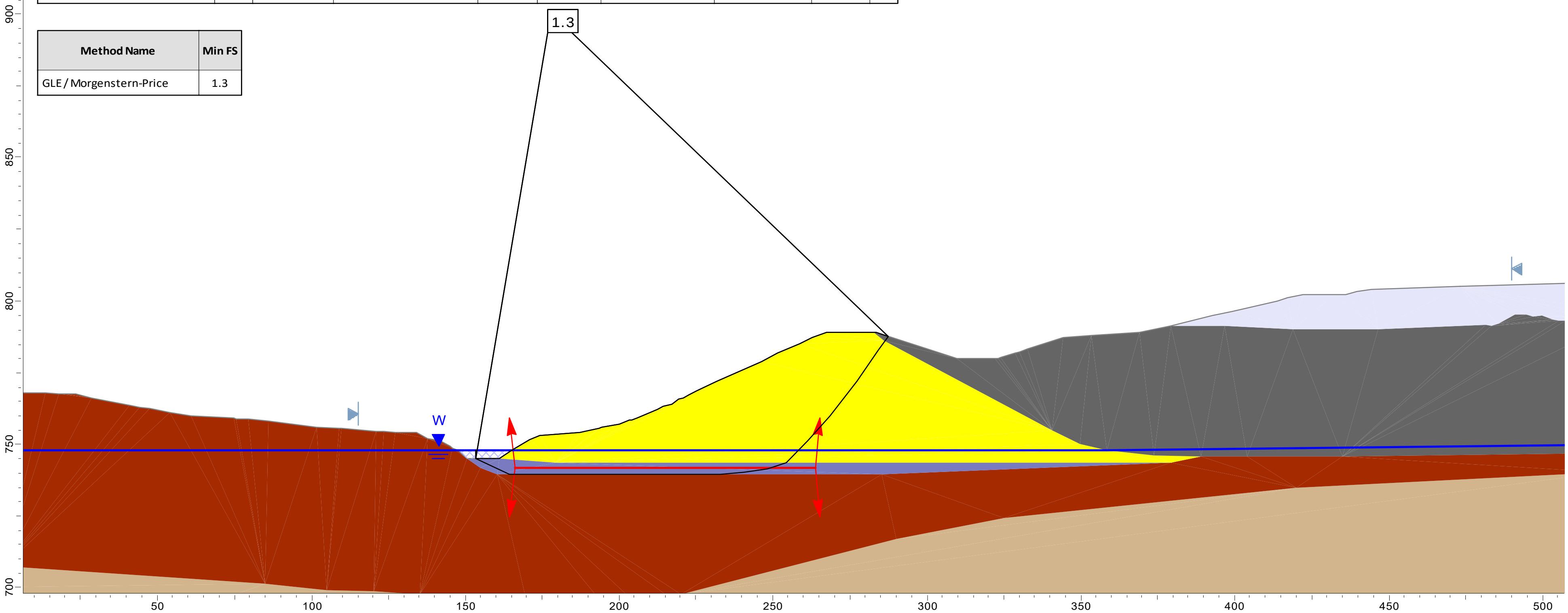
Section B-B
Surchage Pool

CLIENT **Georgia Power Company**

FIGURE **3(b)**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Cohesion Type	Shear Normal Function	Water Surface	Hu Type	Hu
PWR		140	Undrained	10000	Constant		Water Surface	Custom	0
Staked Ash Undrained		110	Shear Normal function			Compacted Ash	Water Surface	Custom	1
Sluiced Ash Undrained		90	Shear Normal function			Sluiced Ash	Water Surface	Custom	1
Dike Fill Soil Undrained		125	Shear Normal function			Fill	Water Surface	Custom	1
Alluvial Deposit Undrained		115	Shear Normal function			Alluvium	Water Surface	Custom	1
Residuum Soil Undrained		125	Shear Normal function			Lower Residuum	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.3



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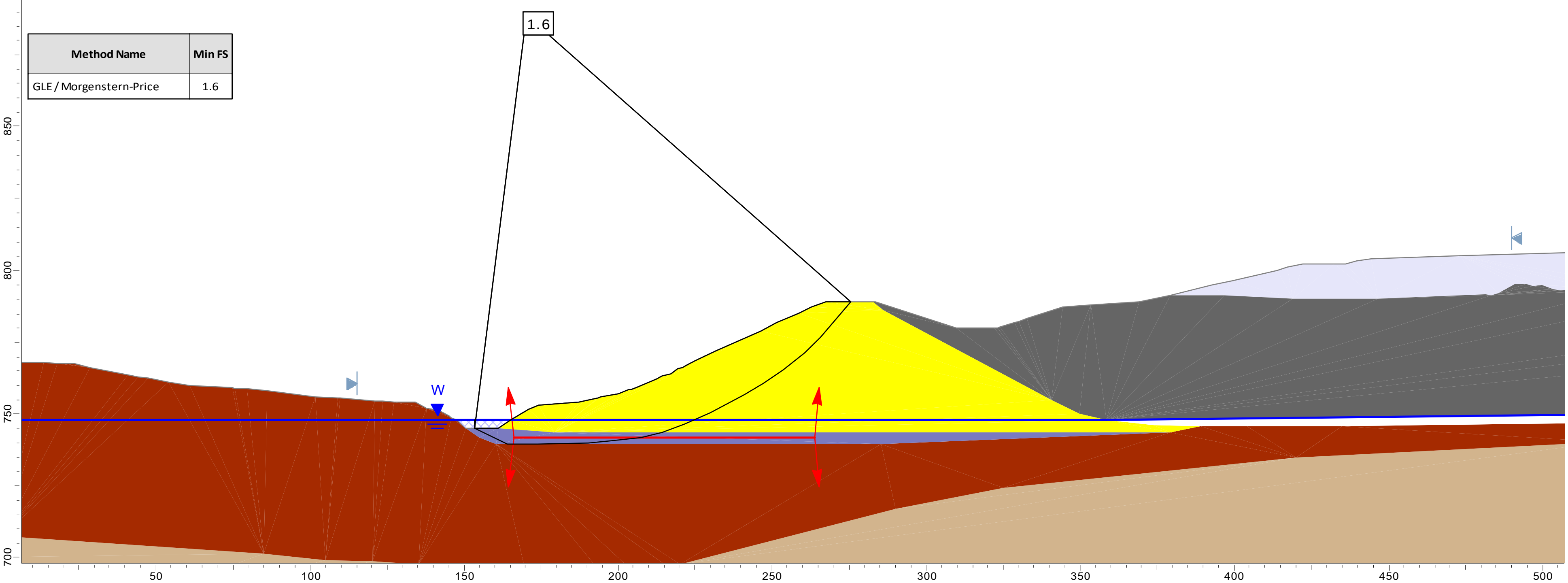
DATE	April 2023	TITLE	Section B-B Seismic Screening
MADE BY	LJ		
CAD	-		
FILE: STABILITY	CHECK: HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW: GLH	FIGURE	3(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30				Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24				Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30				Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28				Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30				Water Surface	Custom	1
PWR		140	Undrained	10000		Constant			Water Surface	Custom	0
Sluiced Ash Liquefied		90	Vertical Stress Ratio				0.08	0	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.6



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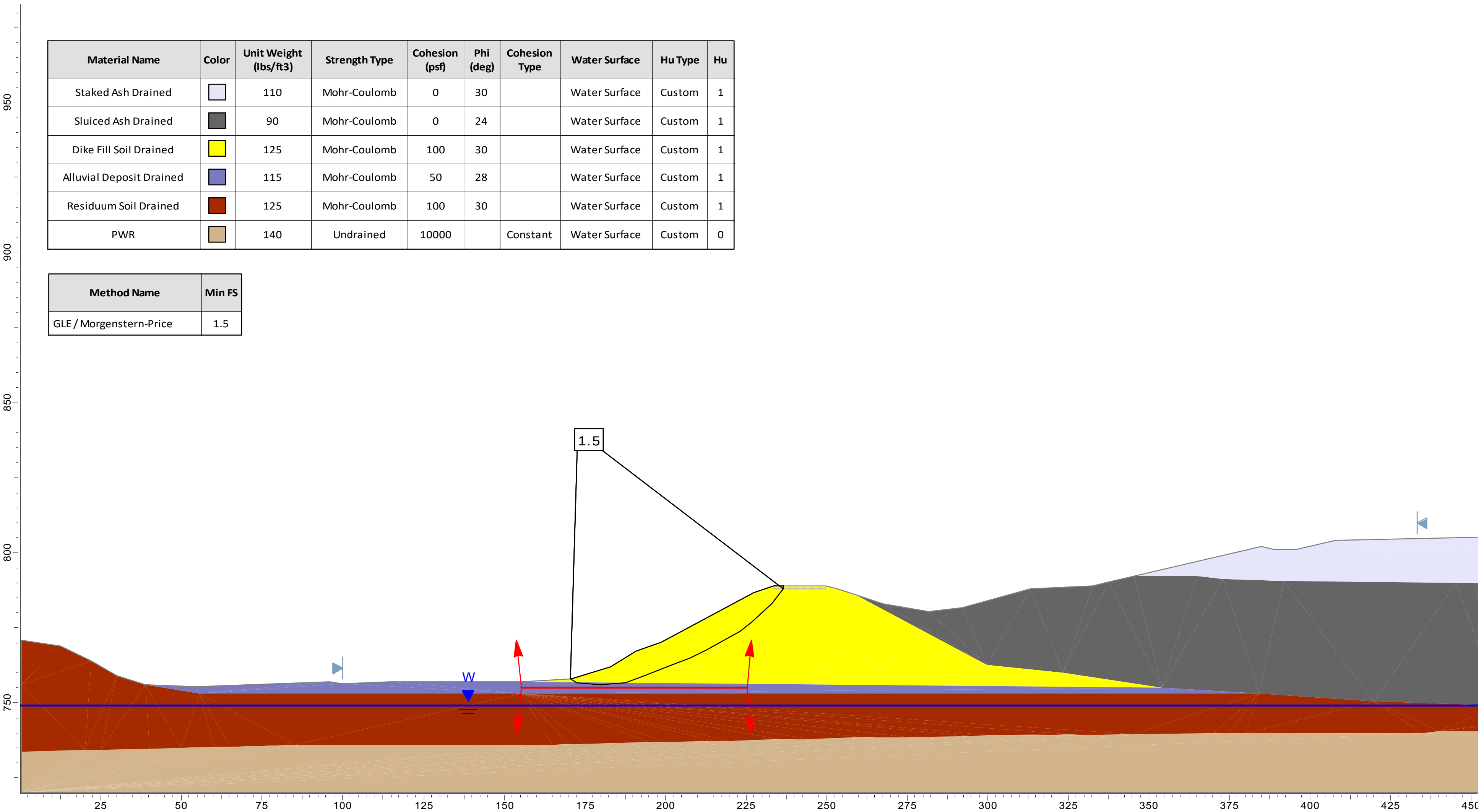
	DATE April 2023	TITLE	**Section B-B Post Liquefaction**
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK HJ	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	3(d)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.5



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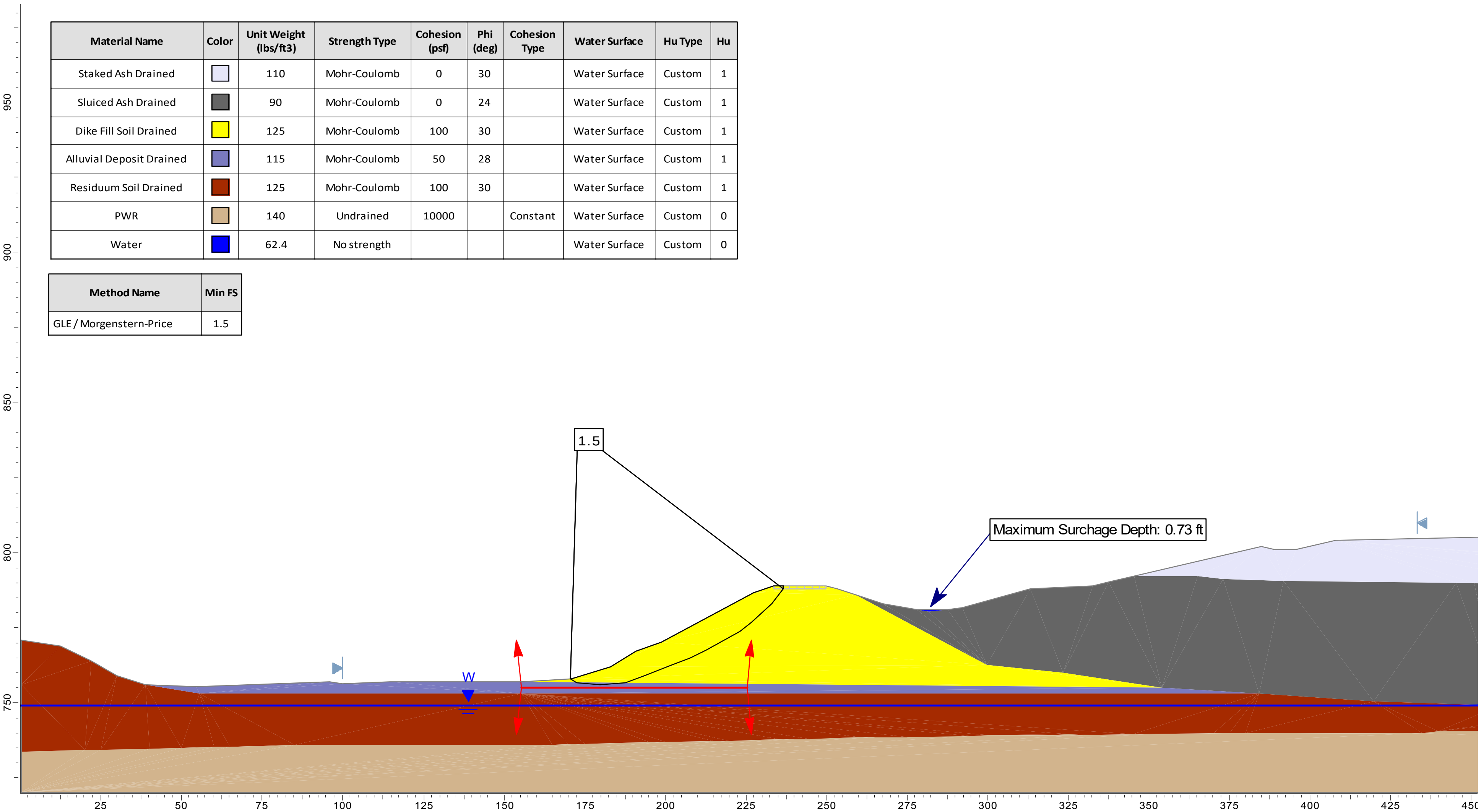
	DATE April 2023	TITLE	Section C-C Long Term, Storage Pool
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	4(a)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0
Water		62.4	No strength				Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.5



FILE	STABILITY		
PROJECT No.	1777449	REV.	0

SCALE	AS SHOWN
DATE	April 2023
MADE BY	LJ
CAD	-
CHECK	HJ
REVIEW	GLH

PROJECT **State CCR Permitting Services - MCD Pond 1**

TITLE

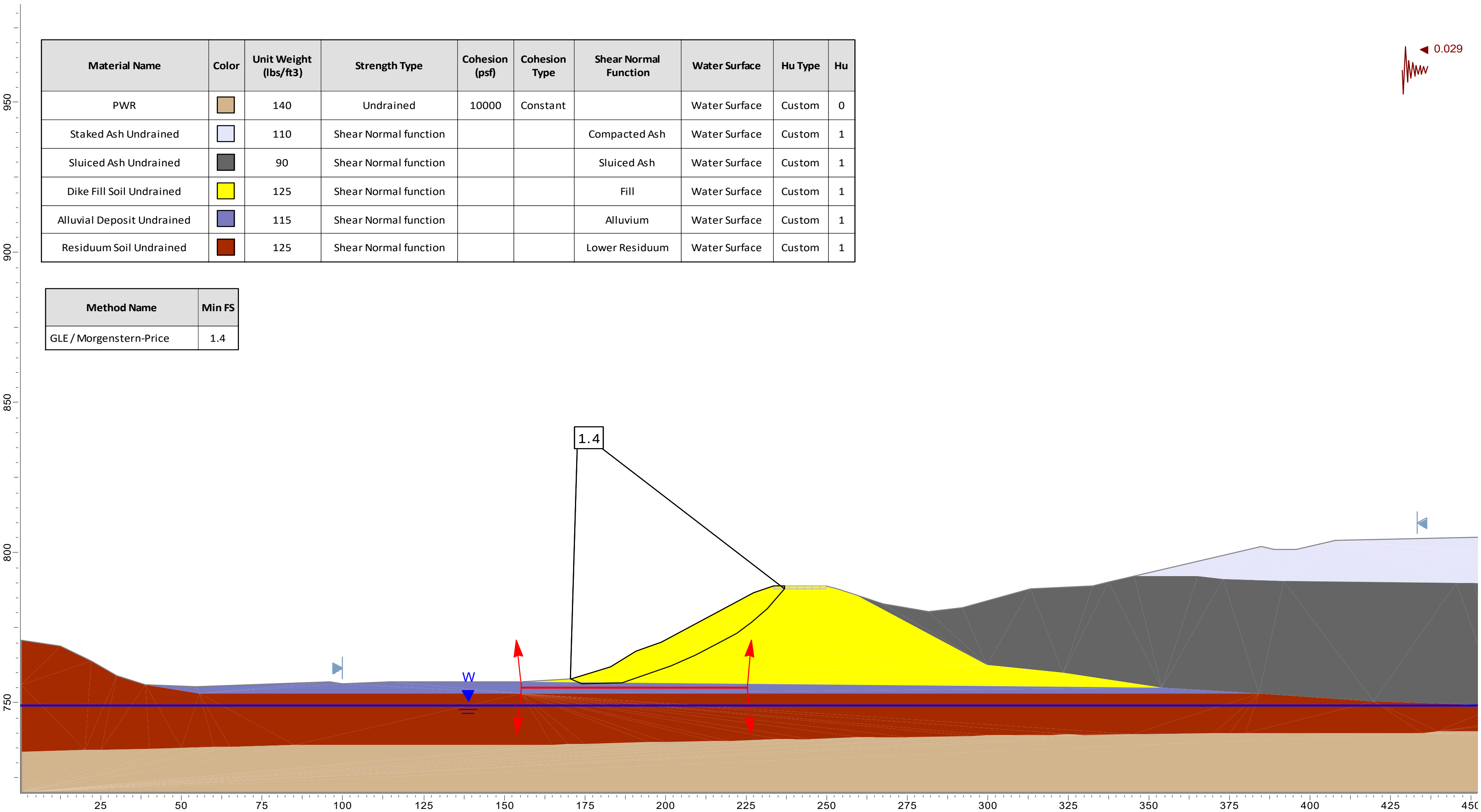
Section C-C
Surchage Pool

CLIENT **Georgia Power Company**

FIGURE **4(b)**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Cohesion Type	Shear Normal Function	Water Surface	Hu Type	Hu
PWR		140	Undrained	10000	Constant		Water Surface	Custom	0
Staked Ash Undrained		110	Shear Normal function			Compacted Ash	Water Surface	Custom	1
Sluiced Ash Undrained		90	Shear Normal function			Sluiced Ash	Water Surface	Custom	1
Dike Fill Soil Undrained		125	Shear Normal function			Fill	Water Surface	Custom	1
Alluvial Deposit Undrained		115	Shear Normal function			Alluvium	Water Surface	Custom	1
Residuum Soil Undrained		125	Shear Normal function			Lower Residuum	Water Surface	Custom	1

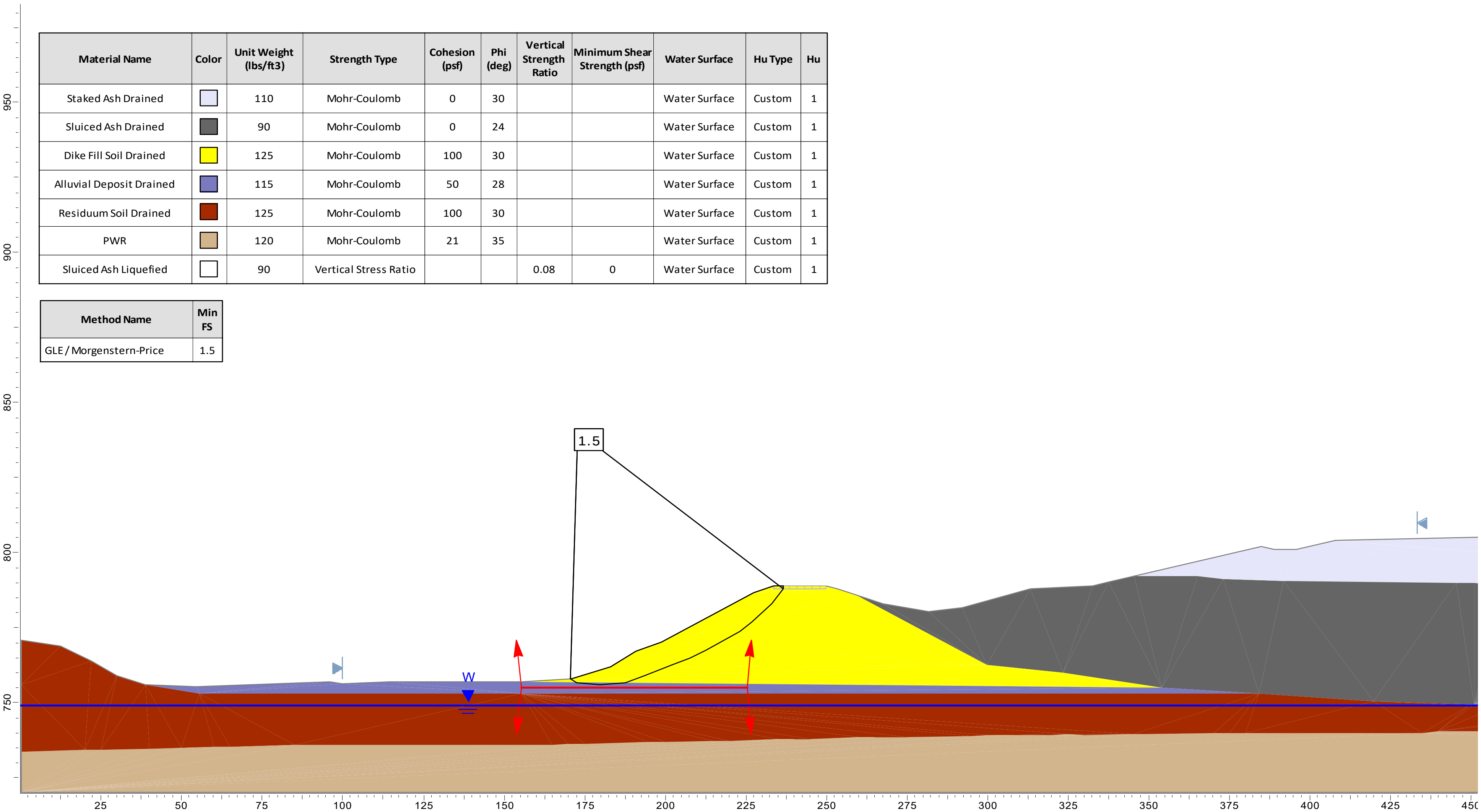
Method Name	Min FS
GLE / Morgenstern-Price	1.4



	SCALE AS SHOWN	PROJECT	State CCR Permitting Services - MCD Pond 1
	DATE April 2023	TITLE	Section C-C Seismic Screening
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	4(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Staked Ash Drained		110	Mohr-Coulomb	0	30			Water Surface	Custom	1
Sluiced Ash Drained		90	Mohr-Coulomb	0	24			Water Surface	Custom	1
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30			Water Surface	Custom	1
Alluvial Deposit Drained		115	Mohr-Coulomb	50	28			Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30			Water Surface	Custom	1
PWR		120	Mohr-Coulomb	21	35			Water Surface	Custom	1
Sluiced Ash Liquefied		90	Vertical Stress Ratio			0.08	0	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.5



wsp	SCALE: AS SHOWN	PROJECT	State CCR Permitting Services - MCD Pond 1
	DATE: April 2023	TITLE	Section C-C Post Liquefaction
	MADE BY: LJ		
	CAD: -		
FILE: STABILITY	CHECK: HJ	CLIENT	Georgia Power Company
PROJECT No. 1777449 / REV. 0	REVIEW: GLH	FIGURE	4(d)

APPENDIX C

Seismic Hazard Calculation Package

GOLDER

CALCULATIONS

Date:	February 20, 2018	**Made by:**	LJ
Project No.:	1777449	**Checked by:**	JGM
Subject:	Seismic Hazard Calculation	**Reviewed by:**	GLH

PROJECT: PLANT MCDONOUGH INACTIVE CCR SURFACE IMPOUNDMENTS

1.0 OBJECTIVE

This calculation package identifies and summarizes the seismic hazard for the closed conditions of the inactive CCR surface impoundments AP-1, AP-2, and combined unit AP-3/4 at Plant McDonough-Atkinson (Plant McDonough), located at 84.476˚W and 33.829˚N. The seismic hazard is necessary for geotechnical design evaluations of stability under earthquake loading and liquefaction susceptibility.

2.0 SEISMIC HAZARD SUMMARY

The United States Environmental Protection Agency's (EPA) "Disposal of Coal Combustion Residuals from Electric Utilities" Final Rule (40 C.F.R. Part 257 and Part 261) (CCR Rule) specifies seismic analyses be completed for a seismic event with a 2% probability of exceedance in 50 years (2% / 50yr), equivalent to a return period of approximately 2,500 years, based on the United States Geological Survey (USGS) seismic hazard maps. The USGS has provided online tools associated with this hazard for its 2014 seismic hazard model. The sections below detail the use of these tools to obtain seismic hazard data for use in analyses.

3.0 PEAK GROUND AND SPECTRAL ACCELERATION

The peak ground acceleration (PGA) and spectral ground accelerations (Sa) corresponding to a range of spectral periods are necessary for many engineering analyses including slope stability and liquefaction analyses. For a 2% probability of exceedance (PE) in 50 years, The USGS provides a reference PGA and spectral accelerations corresponding to a reference site on the border between the National Earthquake Reductions Hazard Program (NEHRP) site classes B and C with an average shear wave velocity in the upper 30 m (Vs30) of 760 m/s. These reference accelerations are often referenced with a BC subscript (e.g. PGA_{BC}) and are scaled as appropriate to match site conditions and analysis input requirements. Figure 1 below shows the project site on the 2014 seismic hazard map for PGA_{BC}, and Figure 2 displays the uniform hazard response spectrum curve, which plots the reference spectral acceleration, or ground motion, for various spectral periods. The uniform hazard response spectrum curve for the site is presented in tabular form in Table 1.

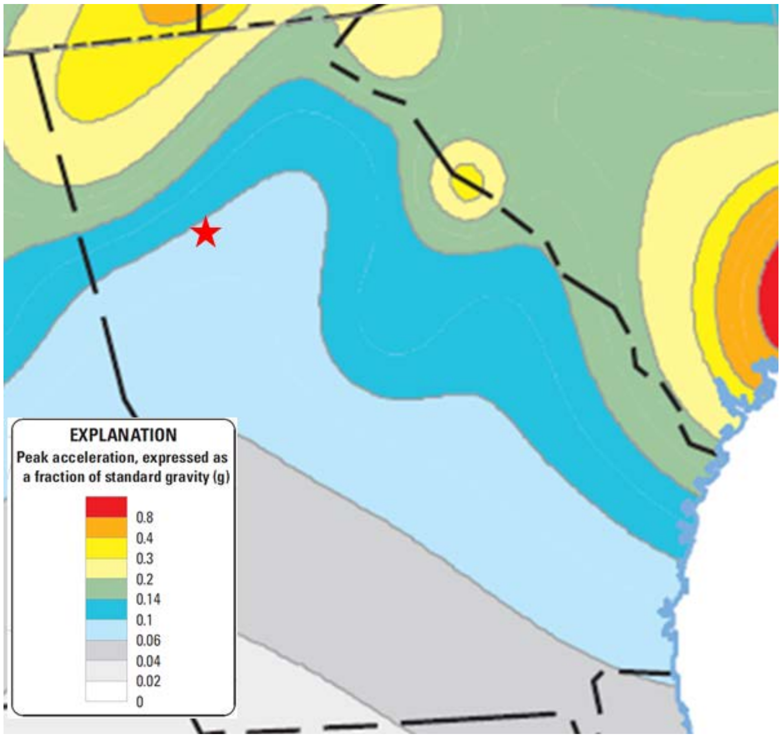


Figure 1: PGA(BC) for the 2% PE in 50 years at the project site (red star). (USGS 2014).

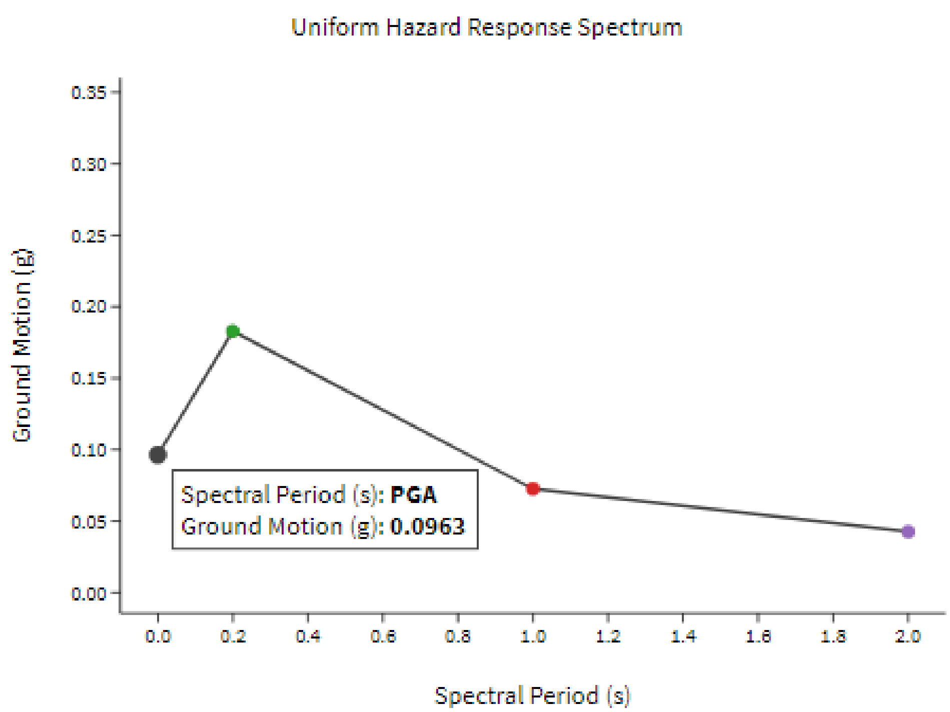


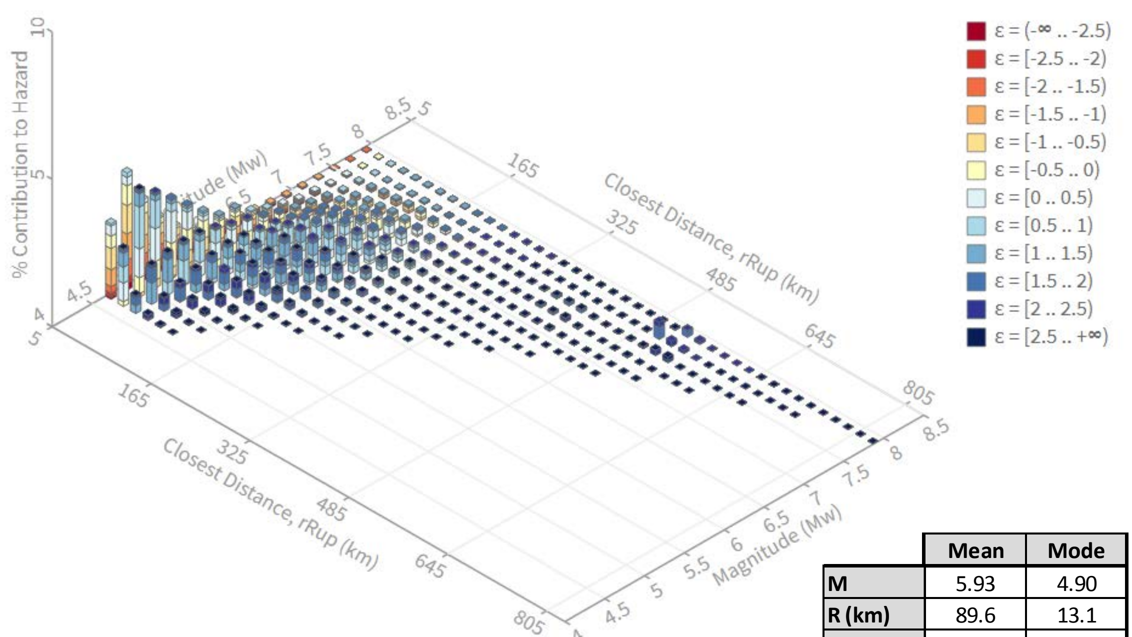
Figure 2: Uniform Hazard Response Spectrum for the 2% PE in 50 years Seismic Hazard at the Project Site (USGS 2014).

Table 1: Reference Site (BC) PGA and Spectral Acceleration for The 2% PE in 50 Year Seismic Hazard at the Project Site (USGS 2014).

Spectral Period (s)	Acceleration, BC (g)
0 (PGA)	0.0963
0.2	0.1829
1.0	0.0725
2.0	0.0426

3.1 Seismic Hazard Deaggregation

The seismic hazard is compiled from multiple predictive models which consider many seismic sources of varying combinations of earthquake magnitude and distance from the project site. For each magnitude and distance pair, models predict the resulting accelerations and activity rates for the project site. The results of these predictive models are aggregated to produce the seismic hazard model for specified return periods. The seismic hazard model can be deaggregated to obtain the contribution to hazard percentage of magnitude and distance combinations. This information is necessary for analyzes requiring earthquake magnitude (e.g. liquefaction susceptibility) or distance. Figure 3 below displays a deaggregation plot of the PGA_{BC} at the project site for a 2% PE in 50 years with descriptive statistics available through the USGS online tools.



	Mean	Mode
M	5.93	4.90
R (km)	89.6	13.1
ε_0	0.39	-1.01

Figure 3: Deaggregation Plot of the PGA_{BC} at the Project Site for a 2% PE in 50 Years

3.2 Design Earthquake Magnitude

Some seismic analysis methods require a design earthquake magnitude as an input. One such analysis is the liquefaction screening method. While the probabilistic seismic hazard tool provided by the USGS (discussed above) gives a design PGA and deaggregated magnitude and distance pairs for all sources contributing to the earthquake hazard, a design magnitude is not explicitly provided by the tool.

The selection of either the mean or modal magnitude produces inconsistent results for some analyses. Specifically, liquefaction assessments based on a design earthquake magnitude and ground acceleration are particularly sensitive to this selection because the relationship between duration (represented by magnitude) and liquefaction potential is non-linear. Kramer (2008) suggests that the best way to handle this issue is to perform liquefaction calculations for a series of realistic site magnitudes and to weight the results according to the relative contribution of each magnitude to the probabilistic seismic hazard (provided in the USGS tools).

Golder implemented this approach in the liquefaction analysis. Recognizing that the Magnitude Scaling Factor (MSF) is the only magnitude-dependent term in the simplified NCEER approach (Youd et al. 2001), Golder calculated a weighted-average MSF (weighted by the relative contribution of each magnitude), and then calculated the magnitude corresponding to that MSF.

Golder calculated the design earthquake magnitude to be 5.75 and was used in all seismic analyses requiring a design magnitude for consistency. As is typical, the design earthquake magnitude (5.75) fell between the mean magnitude (5.93) and modal magnitude (4.90) provided in Figure 3.

4.0 DETERMINATION OF SITE-SPECIFIC PEAK GROUND ACCELERATION

For liquefaction analysis, the site-specific peak ground acceleration at the surface, a_{max}, was calculated from the site reference peak ground acceleration (PGA_{BC}). The PGA_{BC} value was multiplied by an amplification factor calculated from the average shear wave velocity in the upper 30 meters (Vs30) to obtain a representative a_{max}. The shear wave velocity was directly measured every two meters in CPT-10-AP3, and a representative shear wave velocity was derived from these measurements. Figure 4 shows the measured shear wave velocities and the representative shear wave velocity profile. The Vs30 (listed in Table 2) was calculated from the representative profile to be 621 ft/s.

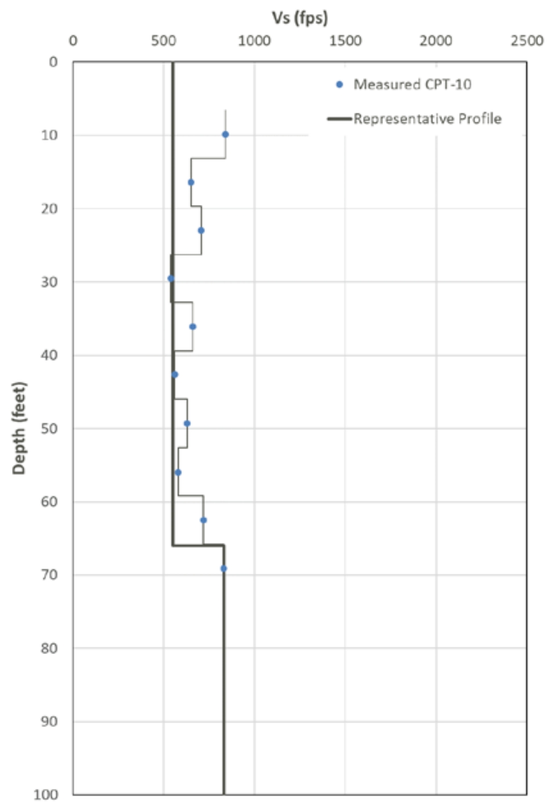


Figure 4. Representative shear wave velocity profile for Plant McDonough CCR Surface Impoundments

Table 2: Representative Shear Wave Velocity in the Upper 30 m (Vs30)

Pond ID	Vs30 (ft/s)	Vs30 (m/s)
AP-1, AP-2 &AP-3/4	621	189

4.1 Determination of site amplification factor coefficient F_a

An amplification factor was determined from two sources:

a) Atkinson and Boore's 2006 publication on earthquake ground-motion prediction equations for Eastern North America

b) International Building Code (IBC, 2012)

Atkinson and Boore's publication provides a site response term which is used to amplify the PGA_{BC}, and the IBC provides a site coefficient F_a (amplification factor) as well. While the IBC factor was originally developed for buildings, the IBC amplification factor was calculated as a check on the Atkinson and Boore method. Amplification factors from these two sources were averaged to obtain a representative amplification factor.

Table 3: Site coefficient F_a

Pond ID	Atkinson and Boore (2006)	IBC (2012)	Selected for Analysis
AP-1, AP-2, & AP-3/4	1.71	1.6	1.66

4.2 Site-specific peak ground acceleration a_{max}

$$a_{max} = PGA_{BC} * F_a = 0.0963g * 1.66 = 0.16\ g \tag{1}$$

With a proposed site coefficient F_a of 1.66, Golder calculated the amplified site-specific peak ground acceleration a_{max} to be 0.16 g.

Table 4: a_{max} at AP-1, AP-2, & AP-3/4

Pond ID	Site Specific Amplified PGA a_{max}
AP-1, AP-2, & AP-3/4	0.16 g

5.0 PSEUDOSTATIC COEFFICIENT – SEISMIC SLOPE STABILITY ANALYSIS

For slope stability analyses, Golder used the Bray and Travasarou (2009) screening method which models seismic loading using a pseudostatic coefficient (k). This section details the calculation of the pseudostatic coefficient for the project site. Details on the slope stability analysis are available in the Safety Factor Assessment package for the facility units.

Stability under seismic conditions is calculated using the pseudostatic method to model horizontal seismic forces as the product of a seismic coefficient (k) and the weight of the sliding mass. Bray and Travasarou (2009) proposed screening methodology to determine the seismic coefficient k based on the degraded period of the sliding mass and an allowable seismic displacement threshold. The screening method includes an equation to calculate the pseudostatic coefficient for periods of 0.2 and 0.5 seconds, which encompasses the range of typical slope periods. A period of 0.2 s produces a more conservative coefficient, so for this analysis, Golder used the equation associated with a period of 0.2 s and an allowable seismic displacement of 15 cm:

$$k_{15\ cm} = (0.036M_w - 0.004)S_a - 0.030 > 0.0,\ for\ S_a = S_a(T = 0.2\ s) < 2.0\ \text{g} \tag{2}$$

Where, k_{15cm} = pseudostatic coefficient

Mw = Design Earthquake Magnitude

Sa = Spectral acceleration at the base of the sliding mass

As noted in Section 3.0, the BC spectral acceleration at a period of 0.2 s is 0.1829 g. This value is multiplied by an amplification factor to obtain the acceleration at the base of the sliding mass. Golder used an amplification factor of 1.6 as prescribed by the international building code (IBC 2012) for a site class D. The project site was classified as D according to the representative shear wave velocity in the upper 30 meters or 100 feet (Vs30). Thus, the spectral acceleration S_a used in the equation is 0.293 g (0.1829g x 1.6). The pseudostatic coefficient was calculated to be 0.029 g as shown in Table 5.

Table 5: $k_{15\ cm}$ at AP-1, AP-2, & AP-3/4

Pond ID	$k_{15\ cm}$
AP-1, AP-2, & AP-3/4	0.029 g

6.0 REFERENCE

Atkinson, G.M. and D.M. Boore (2006) “Earthquake Ground-Motion Prediction Equations for Eastern North America,” Bulletin of the Seismological Society of America, Vol. 96, No. 6, pp. 2181-2205.

Bray, J.D., and Travasarou, T. (2009). Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 9: pp. 1336-1340.

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International Code Council, Inc. (2012), “2012 Insertional Building Code”, Section 1613.3

United States Geologic Survey (2018), Unified Hazard Tool. Accessed January 9, 2018. https://earthquake.usgs.gov/hazards/interactive/.

Youd, T.L. et al. (2001). “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils”, Journal of Geotechnical and Geoenvironmental Engineering, vol. 127, No. 4, April 2001.

APPENDIX D

Liquefaction Assessment Calculation Package

CALCULATIONS

Date:	March 2018	**Made by:**	LJ
Project No.:	1777449	**Checked by:**	JGM
Subject:	AP-1 Liquefaction Assessment	**Reviewed by:**	GLH

PROJECT: PLANT MCDONOUGH-ATKINSON CCR UNIT AP-1

1.0 OBJECTIVE

The objective of this calculation is to assess the liquefaction potential of the foundation and dike soils at CCR Unit AP-1 at Georgia Power Company's Plant McDonough in Cobb County, Georgia. This assessment is a screening-level assessment performed according to the simplified procedure as described in Youd et al. (2001). Borehole records and standard penetration test (SPT) data were used to estimate the soil properties for this assessment.

Data for the dike and foundation soils was available from the following six SPT borings, and factors of safety against liquefaction were calculated for these borings:

- AP1-1
- AP1-3
- AP1-4
- AP1-10
- B-11
- B-12

2.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

2.1 Site Geology

The site is located in the Piedmont geologic region, characterized by igneous and metamorphic bedrock. In general, underlying rock at the project site consists of schist and gneiss. The depth to rock surface varies across the site, but rock is generally encountered 20 to 60 feet below ground surface (ft-bgs). This depth corresponds to a top of bedrock elevation ranging from about 730 to 750 feet above mean sea level (ft-msl). Additional details of the site geology are available in the Site Hydrogeologic Report (Golder, 2018).

2.2 Near Surface Conditions

The subsurface conditions near the perimeter berms of AP-1 consists of the following major layers:

- Compacted coal combustion residual (CCR) fill
- Sluiced CCR fill
- Dike fill (reworked site residual and alluvial soils)
- Alluvial layers
- Residuum (Silty Sands and Sandy Silts)
- Lower saprolite and partially weathered rock (PWR)
- Bedrock (Schist and Gneiss)

The alluvial deposit is observed locally in the areas of current or historic stream channels with a thickness of up to 15 ft nominally over the piedmont residuum at the site. The piedmont residuum presents as sandy silts near the ground surface and transitions with depth to a silty sand. These soils are derived from the underlying rock which generally consists of schist and gneiss. The subsurface materials and their properties are summarized in Appendix B and were developed based on subsurface explorations by Golder at the site in 2015, 2016, and 2017 and historic records from Georgia Power, Southern Company, and other consultants that have worked on the site previously (AMEC, Law, etc.).

3.0 LIQUEFACTION ASSESSMENT METHODOLOGY

Seismically-induced liquefaction susceptibility was evaluated using the National Center for Earthquake Engineering Research (NCEER) simplified procedure with SPT data (Youd et al., 2001). The simplified procedure is an empirical method to calculate the factor of safety against liquefaction. The factor of safety is defined as a ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). CRR is a measure of a given soil's resistance to liquefaction, and Golder calculated the CRR using SPT data. The CSR is a measure of the seismic demand on the soil and was estimated using seismic hazard assessment resources provided by the United States Geologic Survey (USGS).

3.1 CSR Calculation

The CSR is defined as:

$$CSR = \frac{\tau_{ave}}{\sigma'_v} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d \qquad (1)$$

Where a_{max} is the peak horizontal acceleration at the ground surface, g is the acceleration due to gravity, σ_v is the total vertical overburden stress, σ_v' is the effective vertical overburden stress, and r_d is a depth-dependent stress reduction factor defined as:

$$r_d = 1.0 - 0.00765z \quad for\ z \leq 9.15\ m \qquad (2)$$

$$r_d = 1.174 - 0.0267z \quad for\ 9.15\ m < z \leq 23\ m \tag{3}$$

$$r_d = 0.744 - 0.008z \quad for\ 23\ m < z \leq 30\ m \tag{4}$$

$$r_d = 0.50 \quad for\ z > 30\ m \tag{5}$$

Where z is the depth in meters (m).

3.1.1 Site-Specific Peak Ground Acceleration (a_{max})

The site-specific peak ground acceleration (a_{max}) is based on the reference site peak ground acceleration (PGA_{BC}). For this analysis, the PGA_{BC} is 0.096g. Details on the PGA_{BC} are available in Appendix C: Seismic Hazard Calculation Package. The PGA_{BC} was scaled to the site-specific peak ground acceleration according to the amplification factor (F_a) provided in the international building code (IBC 2012). This amplification factor depends on the site class, which can be calculated with shear wave velocity (Vs), SPT blow counts (N), or undrained shear strength (Su). Table 1 presents the various IBC site classes and their associated amplification factors.

Table 1. Site Class Types (IBC 2012)

Site Class ID	Site Class Name	$V_{S(30)}$ (ft/s)	$\overline{N}$	$\overline{s_u}$ (psf)	$F_a(T \leq 0.25\ s)$
E	Soft Soil	< 600	<15	< 1,000	2.5
D	Stiff Soil	600 to 1,200	15 to 50	1,000 to 2,000	1.6
C	Very Dense Soil and Soft Rock	1,200 to 2,500	>50	> 2,000	1.2
B	Rock	2,500 to 5,000	N. A.	N. A.	1.0
A	Hard Rock	> 5,000	N. A.	N. A.	0.8

Golder used the blow count method ($\overline{N}$) to estimate the site class. $\overline{N}$ is a representative blow count for materials in the upper 100 feet and is defined as:

$$\overline{N} = \frac{\sum_{i=0}^{n} d_i}{\sum_{i=0}^{n} \frac{d_i}{N_i}} \tag{6}$$

Where d_i = the thickness of any layer (between 0 and 100 ft) and N_i = the representative SPT N values at each layer. For each borehole analyzed, Golder calculated the representative SPT blow count and obtained amplification factors for each borehole according the IBC site class (see Table 2.). At each borehole, a_{max} was calculated according to the following equation. The resulting a_{max} is also listed in Table 2.

Table 2. Amplification factor at boring locations

BH ID	Predominant Soil Type	$\bar{N}$	Site Class	Fa	a_{max} (T<0.25s)
AP1-1	Fill	22	D	1.6	0.15 g
AP1-3	Fill	16	D	1.6	0.15 g
AP1-10	Fill + Residuum	25	D	1.6	0.15 g
AP1-4	Sluiced Ash	4	E	2.5	0.24 g
B-11	Alluvium	16	D	1.6	0.15 g
B-12	Residuum	33	D	1.6	0.15 g

3.2 CRR Determination

The second major step in assessing the liquefaction susceptibility using the simplified approach is to estimate the CRR. The CRR is calculated from the normalized SPT blow counts $(N_1)_{60cs}$ as a function of the "clean sand" blow count normalized to an overburden pressure of approximately 100 kPa (1 tsf) and a hammer energy ratio or hammer efficiency of 60%. The CRR is based on an earthquake magnitude of 7.5 and a magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5.

The $CRR_{7.5}$ can be approximated as (Youd et al. 2001),

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10 \cdot (N_1)_{60cs} + 45]^2} - \frac{1}{200} \quad (7)$$

3.2.1 Clean Sand Equivalent Cone Penetration Resistance $(N_1)_{60cs}$

According to the NCEER approach, the presence of fines affects the liquefaction resistance of soils. Youd et al. 2001 provides the fine correction of normalized $(N_1)_{60}$ to an equivalent clean sand value $(N_1)_{60cs}$ based on fines content (FC). Golder estimated the fines content from the soil descriptions and lab test results included on each borehole log for all soils except CCR. Golder modeled CCR with 20% fines content because CCR has been measured to behave like a soil with 20% fines content from a drainage and liquefaction potential even though the measured fines content of CCR is typically greater than 80%. The use of a lower fines content adjustment is conservative and results in a greater potential to calculate liquefaction may occur in CCR materials.

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60} \quad (8)$$

Where α and β are coefficients determined from the following relationships,

$$\alpha = 0, \beta = 1.0, \qquad for\ FC \leq 5\% \tag{9}$$

$$\alpha = \exp[1.76 - (190/FC^2)], \beta = \left[0.99 + \left(\frac{FC^2}{1000}\right)\right], \qquad for\ 5\% < FC < 35\% \tag{10}$$

$$\alpha = 5.0, \beta = 1.2, \qquad for\ FC > 35\% \tag{11}$$

Youd et al. (2001) also presents correction factors to normalized SPT results for other influences,

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_s \tag{12}$$

Where N_m is the measured standard pentration resistance; C_N normalizes N_m to a common reference effective overburden stress; C_E corrects for hammer energy ratio (ER); C_R corrects for rod length; and C_s corrects for samplers with or without liners. The detailed equations for each correction factor are available in Youd et al. (2001, Table 2). It should be noted that in Youd et al. (2001), for cases with $(N_1)_{60} > 30$, the soil is regarded as too dense to liquefy. For the purpose of illustration, Golder capped the value of $(N_1)_{60} = 30$.

3.2.2 Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5 (Youd et al. 2001). A number of different MSF values are discussed in the NCEER approach. The MSF values used in this assessment are the revised Idriss values (which are considered a lower bound set of values), and are calculated as:

$$MSF = \frac{10^{2.24}}{M^{2.56}} \tag{13}$$

A probabilistic seismic hazard analysis was used to estimate the ground acceleration, and while such an analysis includes the aggregate contributions from a large number of possible combinations of magnitude and distance from a wide range of known potential sources, a design earthquake magnitude is not specified in the probabilistic tools provided by the USGS. The simplified approach requires the selection of a single earthquake magnitude. Since liquefaction is sensitive to ground motion duration, which is correlated to earthquake magnitude, this selection is an important issue in liquefaction assessments.

The selection of either the mean or modal magnitude produces inconsistent risks of liquefaction because the relationship between duration (represented by magnitude) and liquefaction potential is non-linear. Kramer (2008) suggests that the best way to handle this issue is to perform liquefaction calculations for all magnitudes and to weight the results according to the relative contribution of each magnitude.

Golder has implemented this approach by recognizing that the MSF is the only term in the simplified approach that is affected by the magnitude selection. Golder calculated a weighted-average MSF (weighted by the relative contribution of each magnitude) and then calculated the magnitude corresponding to that MSF.

Golder calculated the earthquake magnitude to be 5.75. This value is less than the mean magnitude (5.93), and is greater than the modal magnitude (4.90).

3.3 Factor of Safety Against Liquefaction

The factor of safety was calculated as:

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF \qquad (14)$$

The factor of safety was calculated for each SPT reading (every recorded SPT depth reading).

4.0 RESULTS AND CONCLUSIONS

The USEPA specifies a minimum factor of safety against liquefaction of 1.2 in the CCR Rule (EPA 2015). Figures 1 and 2 (below) present the calculated factors of safety and supporting data for each SPT reading in each of the six boreholes. From Figure 1(b), the calculations show that the dike fill, residuum, and alluvial deposit meets the target FS and as such are not calculated to liquefy during the design seismic event.

However, the results also indicate that the sluiced CCR may liquefy if saturated. The long-term CCR liquefaction risk is largely mitigated with dewatering, but the effect of saturated ash remaining in the long-term was analyzed through post-liquefaction slope stability analyses. Results of these “post liquefaction” stability analyses are fully presented in the stability analysis section, with it being noted here that calculated factors of safety are in excess of the reference criteria for all cases.

5.0 REFERENCES

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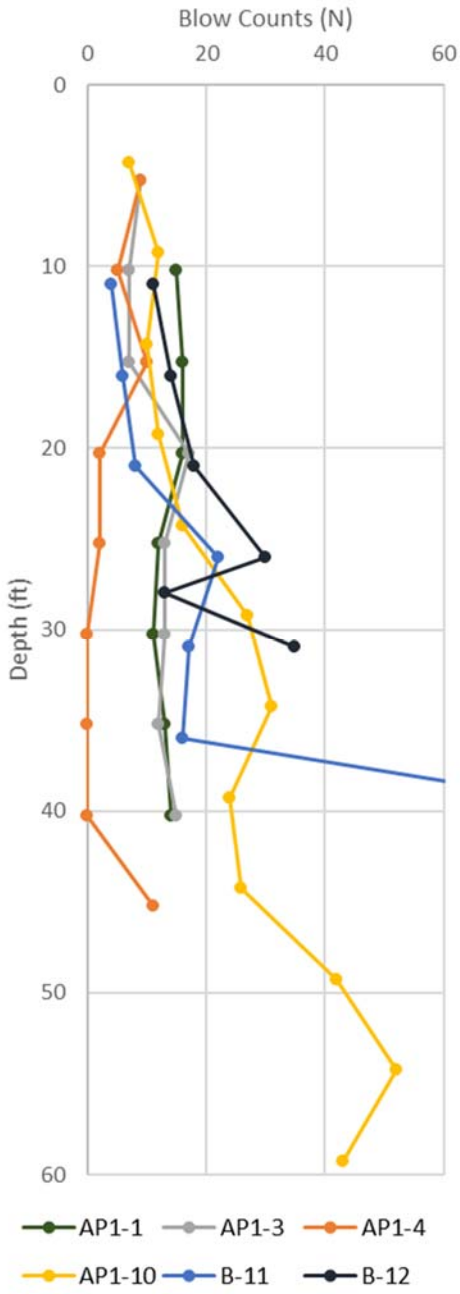
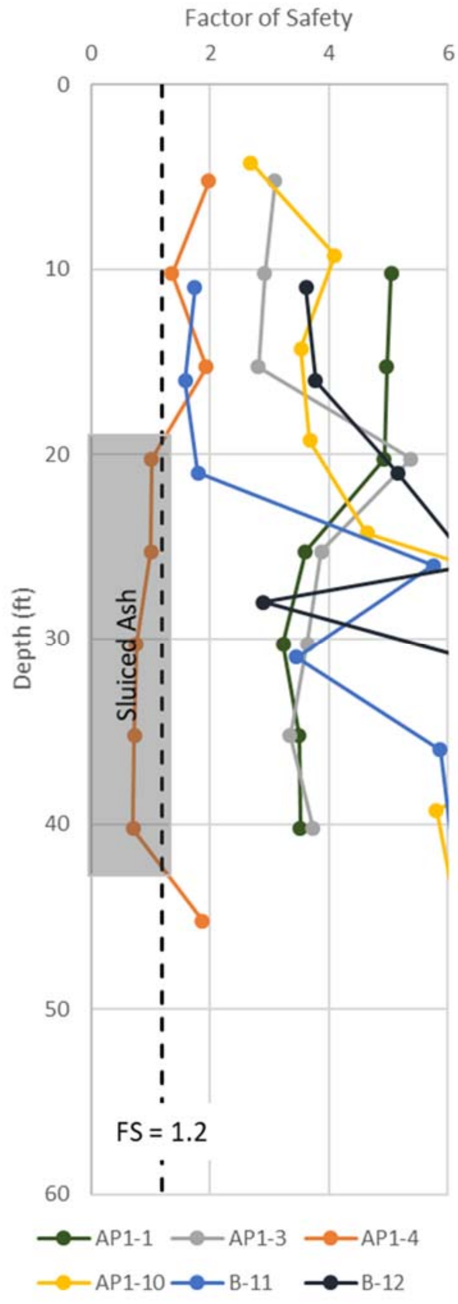
Youd, T.L. et al. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, vol. 127, No. 4, April 2001.



(a) SPT N vs Depth



(b) Factor of Safety

Figures 1. Liquefaction analysis for AP-1

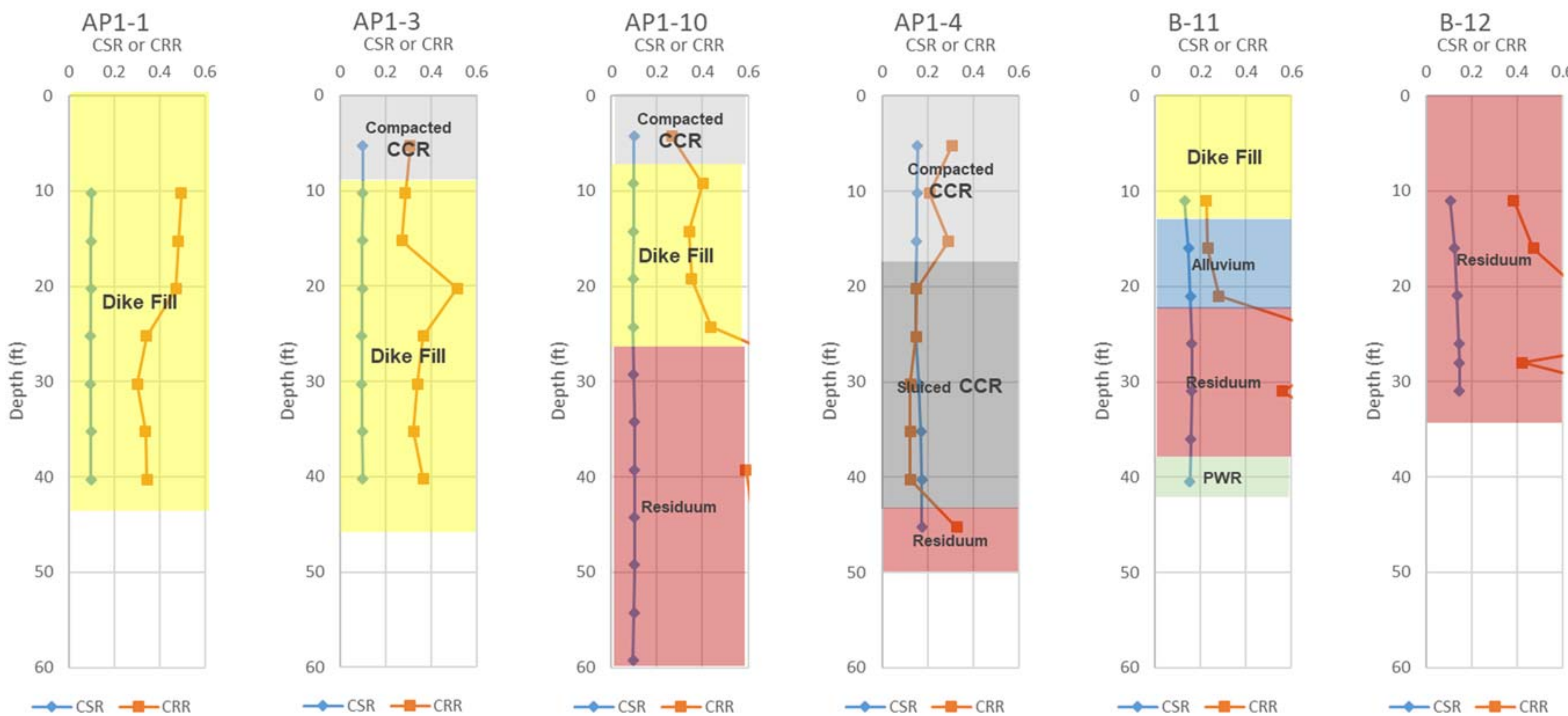


Figure 2: Calculated CSR and CRR for borings

APPENDIX E

Settlement Analysis Calculation Package

CALCULATIONS

Date:	Rev. 01 - July 2021	**Made by:**	LJ
Project No.:	1777449	**Checked by:**	JGM/GLH
Subject:	Settlement Analyses	**Reviewed by:**	GLH

PROJECT: PLANT MCDONOUGH-ATKINSON INACTIVE CCR UNIT AP-1 CLOSURE

1.0 OBJECTIVE

This calculation package summarizes the settlement analyses performed for the closed conditions of the inactive coal combustion residuals (CCR) surface impoundments AP-1 at Plant McDonough-Atkinson (Plant McDonough), located at 84.476˚W and 33.829˚N. Settlement analyses were completed to check closure cap design grades for grade reversal caused by settlement of ash.

2.0 SETTLEMENT ANALYSIS METHODOLOGY

The majority of CCR settlement is expected to occur during closure as ash is placed, graded, and dewatered prior to capping. Settlement caused by ash grading activities will largely occur before the final cap is constructed and as such was excluded from post-closure settlement calculations.

Post-closure settlement of Ash Pond 1 (AP-1) is expected to occur as the water level within the capped ash naturally drops after closure. Since the compressibility of the compacted ash layers are negligible, only settlement in sluiced ash was calculated during consolidation process.

Settlement analyses were completed by calculating settlement at discrete locations within the pond spaced on a 10-ft grid and at every one foot along drainage channels. Settlement analysis results were used to create isopach maps of the total settlement and contours of closure cap grades after settlement. To account for variation and uncertainties relating to ash cementation, a conservative approach was conducted by assuming the over consolidation ratio (OCR) of sluiced ash equals one. From Golder's experience, sluiced ash typically has an OCR value of around 2.5.

2.1 Settlement Analysis

Traditional consolidation theory with material properties based on Golder's experience at other ash storage facilities was applied to obtain a conservative settlement prediction at each discrete settlement location within the pond. The following equations were used to calculate primary settlement in 1-ft layers, then summed up for total primary settlement (Das 2007).

$$S_p = H * C'_c * \log\left(\frac{\sigma_f}{\sigma_i}\right) \; for \; \sigma_f < \sigma_p$$

$$S_p = H * \left(C'_c * \log\left(\frac{\sigma_p}{\sigma_i}\right) + C'_r * \log\left(\frac{\sigma_f}{\sigma_p}\right)\right) \; for \; \sigma_i < \sigma_p < \sigma_f$$

$$S_p = H * C'_r * \log\left(\frac{\sigma_f}{\sigma_i}\right) \; for \; \sigma_p < \sigma_i$$

Where:

S_p = Primary settlement
H = Thickness of layer
C'_c = Coefficient of consolidation (strain)
C'_r = Coefficient of recompression (strain)
σ_i = initial effective stress
σ_f = final effective stress
σ_p = pre consolidation pressure

2.2 Material Properties

The material properties used for settlement analyses are presented in Table 1. It should be noted that settlement in stacked ash, compacted ash, and backfill soil is negligible.

Table 1: Designed Layers for Settlement Analysis

Summary of Material Consolidation Properties				
Name	**Unit Weight (pcf)**	**OCR**	**C'c (strain)**	**C'r (strain)**
Sluiced ash	90	1.0	0.18	0.024
Stacked ash	110	N/A	N/A	N/A
Compacted ash	110	N/A	N/A	N/A
Backfill soil	125	N/A	N/A	N/A

3.0 RESULTS AND CONCLUSION

The settlement analysis results for the AP-1 are presented in Figure 1 below. The maximum calculated settlement in ash pond is 0.43 ft and occurs in the northwest area of AP-1. Based on the settlement evaluations of the final design, there are no identified areas of calculated grade reversal or other settlement concern.

4.0 REFERENCES

Das, Braja M. (2006), Principles of Geotechnical Engineering. Sixth Edition.

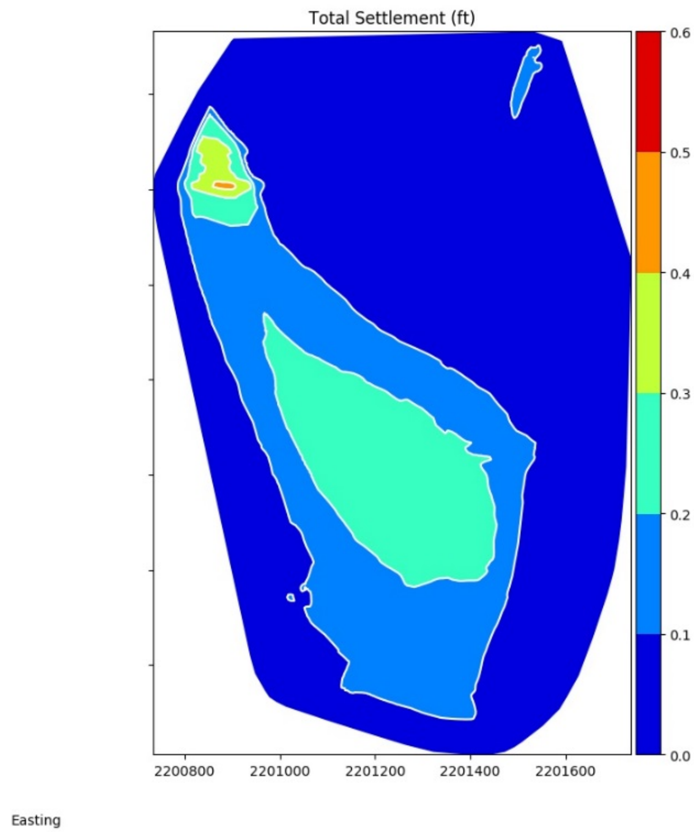


Figure 1. Settlement Analysis for AP-1.

APPENDIX F

Alternative Final Cover Evaluation

GOLDER

CALCULATIONS

Date:	November 2018	**Made by:**	LS
Project No.:	1777449	**Checked by:**	GLH
Subject:	Final Cover Equivalency	**Reviewed by:**	GLH

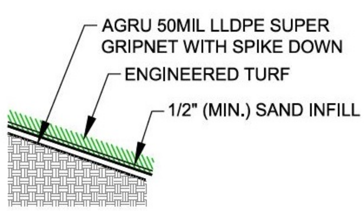
PROJECT: GEORGIA POWER COMPANY– PLANT MCDONOUGH-ATKINSON CCR UNIT AP-1 AND COMBINED CCR UNIT AP-3/4 CLOSURE

1.0 INTRODUCTION AND PURPOSE OF ANALYSIS

Golder Associates Inc. (Golder) and Southern Company Services (SCS) have designed the final closure systems for CCR Unit AP-1 and Combined CCR Unit AP-3/4 at Plant McDonough-Atkinson (Plant McDonough), located in Smyrna, GA. As part of the closure design, Golder conducted an evaluation of the percolation potential and liner performance for the final cover systems for AP-1 and AP-3/4. These analyses, with the use of the US EPA Hydrologic Evaluation of Landfill Performance (HELP) model version 3.07, provide estimates and ranges of the anticipated drainage collected from the final cover system as well as percolation rates through the cover systems on a per plan acre basis. Performance for the designed final cover systems, consisting of ClosureTurf™ is presented to demonstrate equivalent or superior performance to a CCR Unit cover system, as per regulatory requirements (Georgia Solid Waste Management Regulations, Section. 391-3-4-.10(7) and 40 CFR 257.102(d)).

2.0 ANALYSIS PARAMETERS AND CONDITIONS

Analysis for the closure systems was based on the ClosureTurf™ final cover system presented in Figures 1 and 2 below. The final closure conditions for CCR Units 1 and 3/4 at Plant McDonough both consist of sluiced CCR material overlain by stacked CCR material to a maximum combined thickness of 80 feet (representative of the maximum design final height of CCR at AP-3/4), overlain by a geomembrane, engineered turf layer, and sand infill. ClosureTurf™ with Super Gripnet® geomembrane was utilized as the main cover system at AP-1 and AP-3/4 with maximum designed side slopes of 4 ft. horizontal to 1 ft. vertical over CCR areas. MicroSpike® geomembrane of minimum thickness of 40 mils was utilized in place of the 50 mil Super Gripnet® in some areas with shallower slopes, up to a maximum deck slope of 10 degrees. The top deck evaluations of percolation all use the thinner 40 mil MicroSpike® option for conservatism in the evaluations. Areas utilizing Super Gripnet® geomembrane and MicroSpike® geomembrane as part of the cover system are identified in the Closure Design Plan Drawings.



CLOSURETURF FINAL COVER WITH SUPER GRIPNET DETAIL

Figure 1

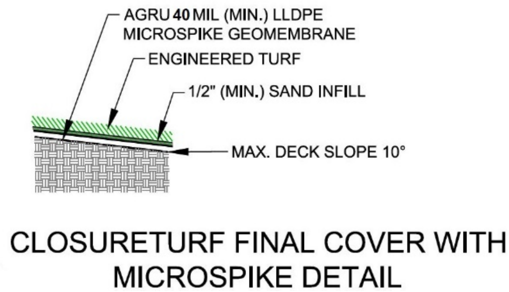


Figure 2

Additionally, Golder analyzed a cover system consisting of a 6-inch vegetative soil layer, 12-inch protective cover layer, double sided geocomposite drainage layer, textured 40-mil minimum thickness LLDPE geomembrane and drainage layer, underlain by an 18-inch compacted material layer as a final cover option as shown in Figures 3 and 4. This cover system option is included for the potential use of a vegetative cover in place of the ClosureTurf™ engineered system in future repairs.

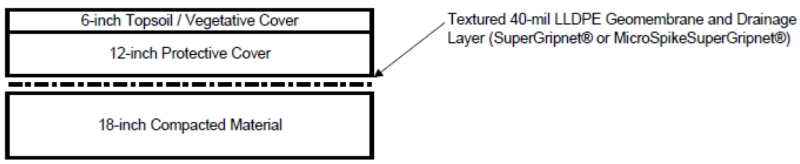


Figure 3 – Soil & Liner Closure System at Top Deck

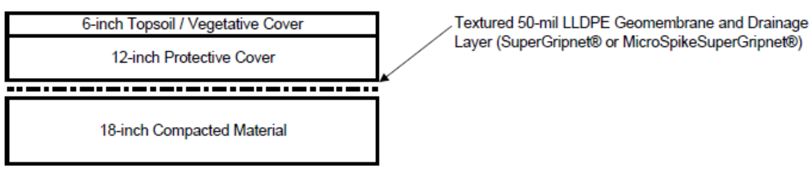


Figure 4 – Soil & Liner Closure System at Side Slopes

Finally, Golder analyzed the prescribed CCR unit final cover as presented in §257.102(d)(3)(i) consisting of a 6-inch vegetative soil layer underlain by an 18-inch soil infiltration layer with a minimum hydraulic conductivity of 1 x 10^{-5} centimeters per second (cm/s) as a base case scenario in the HELP model, as shown in Figure 5.

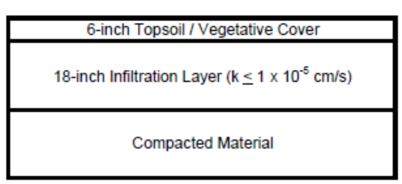


Figure 5 – CCR Unit Final Cover

2.1 Weather Data

Assumptions were made within the HELP model pertaining to weather data for the site location. Precipitation data for Atlanta, Georgia was used for monthly mean precipitation. This data took into account the 25-year, 24 hour storm for Atlanta, GA (GSMM 2001). Synthetic mean temperature data based on 5 years, solar radiation for 33.65° station latitude, and evapotranspiration data for Atlanta, GA from the HELP model database were utilized. Evaporative zone depth values representing fair vegetation quality were utilized for final conditions, and a maximum leaf area index of fair stand (2.0) for final conditions was modeled. The possibility of runoff was estimated for the site as 100% for final conditions. The evapotranspiration data parameters are summarized in Table 1 below.

Table 1: Evapotranspiration Parameters - HELP Model

Stage	% of Area Allowing Runoff	Equivalent Evaporative Zone Depth (in.)	Equivalent Maximum Leaf Area Index	Equivalent Quality of Vegetation
Final Closure Conditions - ClosureTurf™ Option	100	0.7 (1)	2(1)	Fair(2)
Final Closure Conditions – Soil/Liner Option	100	10	2	Fair
Final Closure Conditions – CCR Unit Cover	100	10	2	Fair

(1) – Equivalent properties recommended by the manufacturer of ClosureTurf™ as based on test data
(2) Assumed equal to natural grass case

2.2 Soil and Design Data

The layers summarized in Table 1 must each be designated as one of the four types of layers modeled by HELP, described below. Table 3 outlines the layer designation type for each layer of the three development conditions modeled.

Type 1 – vertical percolation

Type 2 – lateral drainage

Type 3 – barrier soil liner

Type 4 – geomembrane liner

Assumed geomembrane cover conditions of one (1) pinhole per acre, (1) installation defects per acre, and good placement quality were used for all applicable analyses. A runoff drainage length of 400 feet and a slope of 25% representative of the northern slope of AP-3/4 were used in model calculation for final conditions Layer 3 (total drainage lengths ranged from 310 at the southern slope to 400 ft at the northern slope). This length represents the longest slopes at the facility, and the model results are applied to the remaining AP-3/4 and AP-1 slopes for conservatism. It is important to note that drainage benches are located at approximately 30 vertical feet down the slope, but the model accounts for a total drainage length at a constant slope for conservatism. The SCS Run-off curve number is 98, as recommended by ClosureTurf™ manufacturers for estimates in engineering calculations representative of high runoff.

Table 2: Layers Designation in HELP Model for Development Conditions

Development Stage	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
Final Closure Conditions - ClosureTurf™	1 (ClosureTurf™ grass stand)	2 (ClosureTurf™ Geotextile backing)	4 (Geomembrane Liner)	3 (compacted CCR material)	1 (CCR material)	--
Final Closure Conditions – Soil/Liner Option	1 (6-in. vegetative layer)	1 (12-in. Protective Cover)	2 (Double Sided Geocomposite)	4 (LLDPE Geomembrane Liner)	3 (compacted CCR material)	1 (CCR material)
Final Closure Conditions – CCR Unit Cover	1 (6-in. vegetative layer)	3 (18-in. 1 x 10^{-5} cm/s infiltration layer)	1 (CCR material)	--	--	--

3.0 HELP MODEL RESULTS

A simulation period of 30 years was modeled for the final conditions at Plant McDonough. Results for the base case scenario (prescribed CCR Unit cover system) using the parameters outlined in Section 2 are presented in Tables 3 and 4 below. The average calculated percolation through the CCR Unit final cover at AP-3/4 was calculated to be approximately 184 cubic feet per acre per day, whereas the average calculated percolation through the ClosureTurf™ final cover at AP-3/4 was calculated to range from 0.002 to 0.008 cubic feet per acre per day.

Table 3: Calculated Model Results - Percolation and Depth of Water on Final Cover System Side Slopes

Development Stage	Average Daily Percolation (ft^3/day/acre)	Average Annual Percolation (ft^3/year/acre)	Maximum Percolation (ft^3/day/acre)
Final Closure Conditions - ClosureTurf™	0.002	0.606	0.017
Final Closure Conditions – Soil/Liner Option	0.468	170.9	35.7

Development Stage	Average Daily Percolation (ft³/day/acre)	Average Annual Percolation (ft³/year/acre)	Maximum Percolation (ft³/day/acre)
Final Closure Conditions – CCR Unit Cover	183.8	67,100	1,646.3

Table 4: Calculated Model Results - Percolation and Depth of Water on Final Cover System Top Deck (3%)

Development Stage	Average Daily Percolation (ft³/day/acre)	Average Annual Percolation (ft³/year/acre)	Maximum Percolation (ft³/day/acre)
Final Closure Conditions - ClosureTurf™	0.008	2.74	0.069
Final Closure Conditions – Soil/Liner Option	4.22	1,541.4	37.4
Final Closure Conditions – CCR Unit Cover	183.8	67,100	1,646.3

The evaluation of the ClosureTurf™ final cover system, a traditional soil/liner cover system, and the prescribed Soil CCR Unit Cover system indicates that the ClosureTurf™ cover is calculated to have significant performance improvements as compared to the other systems. The ClosureTurf™ cover system results in significant calculated percolation improvements for both the side slope and top deck conditions as compared to a Soil/Liner and Soil CCR Unit Cover system.

4.0 ATTACHMENTS

1. HELP Model Version 3.07 Outputs - Plant McDonough ClosureTurf™ Sideslope
2. HELP Model Version 3.07 Outputs - Plant McDonough Soil and Liner Cover Sideslope
3. HELP Model Version 3.07 Outputs - Plant McDonough CCR Unit Cover Sideslope
4. HELP Model Version 3.07 Outputs - Plant McDonough ClosureTurf™ Top Deck
5. HELP Model Version 3.07 Outputs - Plant McDonough Soil and Liner Cover Top Deck
6. HELP Model Version 3.07 Outputs - Plant McDonough CCR Unit Cover Top Deck

5.0 REFERENCES

US EPA (1994). Hydrologic Evaluation of Landfill Performance (HELP) Model Version 3.07. *United States Environmental Protection Agency.*

US EPA (1994). Hydrologic Evaluation of Landfill Performance (HELP) Model Version, User's Guide for Version 3. Publication No. EPA/600/R-94/168A, 1994

Watershed Geo (2017). ClosureTurf™ with 50mil Super Gripnet® Product Data Sheet.

Watershed Geo (2017). ClosureTurf™ with 40mil Micro spike® Product Data Sheet.

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 **                                                                          **
 **                                                                          **
 **              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
 **                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
 **                DEVELOPED BY ENVIRONMENTAL LABORATORY                     **
 **                  USAE WATERWAYS EXPERIMENT STATION                       **
 **             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
 **                                                                          **
 **                                                                          **
 ******************************************************************************
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 PRECIPITATION DATA FILE:     c:\MCD1118.D4
 TEMPERATURE DATA FILE:       C:\MCD1118.D7
 SOLAR RADIATION DATA FILE:   C:\MCD1118.D13
 EVAPOTRANSPIRATION DATA:     C:\MCD1118.D11
 SOIL AND DESIGN DATA FILE:   C:\MCDSGSSC.D10
 OUTPUT DATA FILE:            C:\MCDSGS18.OUT


 TIME:  19:12     DATE:  11/ 7/2018


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**

         TITLE:  Plant McDonough Closure Turf & SuperGripnet Slopes

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      NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
               COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   1
            THICKNESS                  =      0.50   INCHES
            POROSITY                   =      0.4170 VOL/VOL
            FIELD CAPACITY             =      0.0450 VOL/VOL
            WILTING POINT              =      0.0180 VOL/VOL
            INITIAL SOIL WATER CONTENT =      0.0174 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.999999978000E-02 CM/SEC
          NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                   FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------
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```
              TYPE 2 - LATERAL DRAINAGE LAYER
                MATERIAL TEXTURE NUMBER  34
THICKNESS                   =      0.24   INCHES
POROSITY                    =      0.8500 VOL/VOL
FIELD CAPACITY              =      0.0100 VOL/VOL
WILTING POINT               =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0071 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =   33.0000000000     CM/SEC
SLOPE                       =     25.00   PERCENT
DRAINAGE LENGTH             =    400.0    FEET


                        LAYER  3
                        --------

              TYPE 4 - FLEXIBLE MEMBRANE LINER
                MATERIAL TEXTURE NUMBER  36
THICKNESS                   =      0.05   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                        LAYER  4
                        --------

                TYPE 3 - BARRIER SOIL LINER
                MATERIAL TEXTURE NUMBER   6
THICKNESS                   =     12.00   INCHES
POROSITY                    =      0.4530 VOL/VOL
FIELD CAPACITY              =      0.1900 VOL/VOL
WILTING POINT               =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.4530 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.720000011000E-03 CM/SEC


                        LAYER  5
                        --------

            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   6
THICKNESS                   =    948.00   INCHES
POROSITY                    =      0.4530 VOL/VOL
FIELD CAPACITY              =      0.1900 VOL/VOL
WILTING POINT               =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.1900 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.720000011000E-03 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	98.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	0.7	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	0.010	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	0.378	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.010	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	185.566	INCHES
TOTAL INITIAL WATER	=	185.566	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ATLANTA GEORGIA

STATION LATITUDE	=	33.65	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	77	
END OF GROWING SEASON (JULIAN DATE)	=	316	
EVAPORATIVE ZONE DEPTH	=	0.7	INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	67.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	76.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	69.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.91	4.43	5.91	4.43	4.02	3.41
4.73	3.41	3.17	2.53	3.43	4.23

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
41.90	44.90	52.50	61.80	69.30	75.80
78.60	78.20	73.00	62.20	52.00	44.50

```
          NOTE:  SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
                      COEFFICIENTS FOR    ATLANTA             GEORGIA
                         AND STATION LATITUDE  =  33.65 DEGREES



     HEAD  #1:  AVERAGE HEAD ON TOP OF LAYER  3
     DRAIN #1:  LATERAL DRAINAGE FROM LAYER  2 (RECIRCULATION AND COLLECTION)
     LEAK  #1:  PERCOLATION OR LEAKAGE THROUGH LAYER  4
     LEAK  #2:  PERCOLATION OR LEAKAGE THROUGH LAYER  5

************************************************************************************
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                                       DAILY OUTPUT FOR YEAR    1

------------------------------------------------------------------------------------
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          S
  DAY  A  O  RAIN  RUNOFF   ET   E. ZONE   HEAD      DRAIN     LEAK      HEAD
DRAIN     LEAK
       I  I                       WATER     #1        #1        #1        #2
#2        #2
       R  L   IN.    IN.    IN.  IN./IN.    IN.       IN.       IN.       IN.
IN.       IN.
  ---  -  -  ----- ------ ------ ------- --------- --------- --------- ---------
--------- ---------

    1         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    2         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    3         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    4         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    5         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    6         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    7         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    8         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
    9         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
   10         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
   11         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
   12         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
   13         0.00  0.000  0.000  0.0143    0.0000 .0000E+00 .0000E+00    0.0000
.0000E+00 .0000E+00
   14  *      0.20  0.000  0.044  0.0403    0.0000 .1444E-02 .2535E-07    0.0000
.0000E+00 .0000E+00
   15  *      0.03  0.000  0.037  0.0682    0.0000 .1364E-03 .1173E-07    0.0000
```

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MONTHLY TOTALS (IN INCHES) FOR YEAR 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION	4.39	4.06	3.07	4.71	4.72	5.00
	4.71	2.65	2.13	3.40	2.30	2.95
RUNOFF	2.753	2.606	1.937	2.091	3.080	2.388
	2.934	1.457	0.922	1.531	1.023	1.388
EVAPOTRANSPIRATION	0.412	0.302	0.496	1.004	0.488	0.741
	0.478	0.352	0.631	0.544	0.306	0.480
LATERAL DRAINAGE COLLECTED	1.2248	1.1488	0.6398	1.6150	1.1523	1.7896
FROM LAYER 2	1.3023	0.9186	0.5774	1.3250	0.9622	1.0912
PERCOLATION/LEAKAGE THROUGH	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LAYER 5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS (INCHES)

AVERAGE DAILY HEAD ON	0.000	0.000	0.000	0.000	0.000	0.001
TOP OF LAYER 3	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATION OF DAILY	0.001	0.001	0.001	0.001	0.001	0.001
HEAD ON TOP OF LAYER 3	0.001	0.001	0.001	0.001	0.001	0.001

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ANNUAL TOTALS FOR YEAR 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.09	160046.703	100.00
RUNOFF	24.110	87518.148	54.68
EVAPOTRANSPIRATION	6.233	22626.211	14.14
DRAINAGE COLLECTED FROM LAYER 2	13.7470	49901.781	31.18
PERC./LEAKAGE THROUGH LAYER 4	0.000154	0.559	0.00
AVG. HEAD ON TOP OF LAYER 3	0.0003		

PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
CHANGE IN WATER STORAGE	0.000	0.554	0.00
SOIL WATER AT START OF YEAR	185.566	673603.812	
SOIL WATER AT END OF YEAR	185.566	673604.375	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.007	0.00

****

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	1.936	2.380	3.241	2.661	2.065	1.639
	2.479	1.569	2.043	1.054	1.718	2.054
STD. DEVIATIONS	1.503	1.661	1.905	1.776	1.105	0.920
	1.550	1.028	1.667	0.920	1.268	1.699
EVAPOTRANSPIRATION						
TOTALS	0.706	0.752	0.908	0.796	0.764	0.905
	1.221	0.761	0.732	0.418	0.408	0.652
STD. DEVIATIONS	0.279	0.312	0.337	0.445	0.348	0.418
	0.524	0.433	0.406	0.283	0.185	0.195
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.4202	1.4509	1.5446	1.3864	1.1044	1.2231
	1.2673	0.9315	1.0157	0.8061	1.1033	1.4875
STD. DEVIATIONS	0.6168	0.5038	0.4946	0.4991	0.3535	0.4473
	0.3722	0.3690	0.4901	0.3892	0.4206	0.6812
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

```
                        0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

   STD. DEVIATIONS      0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                        0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

 PERCOLATION/LEAKAGE THROUGH LAYER  5
 ------------------------------------
   TOTALS               0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                        0.0000   0.0000   0.0001   0.0000   0.0000   0.0000

   STD. DEVIATIONS      0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                        0.0000   0.0001   0.0002   0.0001   0.0001   0.0000


-------------------------------------------------------------------------------
              AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
-------------------------------------------------------------------------------


 DAILY AVERAGE HEAD ON TOP OF LAYER  3
 -------------------------------------
   AVERAGES             0.0004   0.0005   0.0005   0.0004   0.0003   0.0004
                        0.0004   0.0003   0.0003   0.0002   0.0003   0.0004

   STD. DEVIATIONS      0.0002   0.0002   0.0001   0.0002   0.0001   0.0001
                        0.0001   0.0001   0.0001   0.0001   0.0001   0.0002

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   AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS    1 THROUGH   30
-------------------------------------------------------------------------------
                                      INCHES           CU. FEET       PERCENT
                                 -------------------   -------------  ---------
 PRECIPITATION                    48.60    (   6.647)    176413.1     100.00

 RUNOFF                           24.837   (  4.5236)     90156.81     51.105

 EVAPOTRANSPIRATION                9.021   (  1.5228)     32745.91     18.562

 LATERAL DRAINAGE COLLECTED       14.74100 (  1.57368)    53509.812   30.33210
   FROM LAYER  2

 PERCOLATION/LEAKAGE THROUGH       0.00017 (  0.00002)        0.606     0.00034
   LAYER  4

 AVERAGE HEAD ON TOP               0.000 (    0.000)
   OF LAYER  3

 PERCOLATION/LEAKAGE THROUGH       0.00016 (  0.00033)        0.584     0.00033
   LAYER  5

 CHANGE IN WATER STORAGE           0.000   (  0.4901)        0.02      0.000
*******************************************************************************


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```

PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.71	17097.301
RUNOFF	4.212	15289.8486
DRAINAGE COLLECTED FROM LAYER 2	0.44590	1618.62012
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000005	0.01679
AVERAGE HEAD ON TOP OF LAYER 3	0.004	
MAXIMUM HEAD ON TOP OF LAYER 3	0.132	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000804	2.91770
SNOW WATER	5.40	19602.5937

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1397

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0143

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	0.0087	0.0174
2	0.0017	0.0071
3	0.0000	0.0000
4	5.4360	0.4530
5	180.1196	0.1900
SNOW WATER	0.000	

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     c:\MCD1810.D4
TEMPERATURE DATA FILE:       C:\MCD1810.D7
SOLAR RADIATION DATA FILE:   C:\MCD1810.D13
EVAPOTRANSPIRATION DATA:     C:\MCD1810.D11
SOIL AND DESIGN DATA FILE:   C:\MCDSGS18.D10
OUTPUT DATA FILE:            C:\MCDSG2sl.OUT


TIME:  20:50     DATE:  11/ 7/2018


******************************************************************************

          TITLE:  Plant McDonough Soil-Liner Cover Slope Nov. 2018

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   8
           THICKNESS                 =      6.00   INCHES
           POROSITY                  =      0.4630 VOL/VOL
           FIELD CAPACITY            =      0.2320 VOL/VOL
           WILTING POINT             =      0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT  =    0.2022 VOL/VOL
           EFFECTIVE SAT. HYD. COND.   =  0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                    FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------
```

```
           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  12
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4710 VOL/VOL
FIELD CAPACITY               =      0.3420 VOL/VOL
WILTING POINT                =      0.2100 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.3750 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC


                        LAYER  3
                        --------

             TYPE 2 - LATERAL DRAINAGE LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0930 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   1.04999995000     CM/SEC
SLOPE                        =     25.00   PERCENT
DRAINAGE LENGTH              =    400.0    FEET


                        LAYER  4
                        --------

             TYPE 4 - FLEXIBLE MEMBRANE LINER
              MATERIAL TEXTURE NUMBER  36
THICKNESS                    =      0.05   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                        LAYER  5
                        --------

              TYPE 3 - BARRIER SOIL LINER
              MATERIAL TEXTURE NUMBER   6
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4530 VOL/VOL
FIELD CAPACITY               =      0.1900 VOL/VOL
WILTING POINT                =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.4530 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC
```

LAYER 6

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 6

THICKNESS	=	948.00 INCHES
POROSITY	=	0.4530 VOL/VOL
FIELD CAPACITY	=	0.1900 VOL/VOL
WILTING POINT	=	0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1900 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.720000011000E-03 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	61.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.605	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	4.662	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.536	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	191.287	INCHES
TOTAL INITIAL WATER	=	191.287	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ATLANTA GEORGIA

STATION LATITUDE	=	33.65 DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	77
END OF GROWING SEASON (JULIAN DATE)	=	316
EVAPORATIVE ZONE DEPTH	=	10.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	65.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	67.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	76.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	69.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.91	4.43	5.91	4.43	4.02	3.41

4.73	3.41	3.17	2.53	3.43	4.23

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
41.90	44.90	52.50	61.80	69.30	75.80
78.60	78.20	73.00	62.20	52.00	44.50

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA
AND STATION LATITUDE = 33.65 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 4
DRAIN #1: LATERAL DRAINAGE FROM LAYER 3 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 5
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 6

****

DAILY OUTPUT FOR YEAR 1

DAY	A I R	S O I L	RAIN IN.	RUNOFF IN.	ET IN.	E. ZONE WATER IN./IN.	HEAD #1 IN.	DRAIN #1 IN.	LEAK #1 IN.	HEAD #2 IN.	DRAIN #2 IN.	LEAK #2 IN.
1			0.00	0.000	0.055	0.2519	0.0189	.6628E-01	.1859E-04	0.0000	.0000E+00	.0000E+00
2			0.00	0.000	0.051	0.2446	0.0160	.5600E-01	.1597E-04	0.0000	.0000E+00	.0000E+00
3			0.00	0.000	0.053	0.2384	0.0152	.5333E-01	.1529E-04	0.0000	.0000E+00	.0000E+00
4			0.00	0.000	0.040	0.2344	0.0143	.4991E-01	.1440E-04	0.0000	.0000E+00	.0000E+00
5			0.00	0.000	0.050	0.2294	0.0110	.3838E-01	.1137E-04	0.0000	.0000E+00	.0000E+00
6			0.00	0.000	0.063	0.2231	0.0089	.3100E-01	.9382E-05	0.0000	.0000E+00	.0000E+00
7			0.00	0.000	0.056	0.2175	0.0074	.2595E-01	.7998E-05	0.0000	.0000E+00	.0000E+00

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.09	160046.703	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	30.154	109459.742	68.39
DRAINAGE COLLECTED FROM LAYER 3	14.2938	51886.387	32.42
PERC./LEAKAGE THROUGH LAYER 5	0.043305	157.197	0.10
AVG. HEAD ON TOP OF LAYER 4	0.2055		
PERC./LEAKAGE THROUGH LAYER 6	0.043381	157.471	0.10
CHANGE IN WATER STORAGE	-0.401	-1456.963	-0.91
SOIL WATER AT START OF YEAR	191.419	694850.437	
SOIL WATER AT END OF YEAR	191.017	693393.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.059	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	0.083	0.092	0.173	0.059	0.015	0.000
	0.021	0.000	0.054	0.000	0.012	0.086
STD. DEVIATIONS	0.305	0.399	0.495	0.308	0.080	0.000
	0.113	0.000	0.202	0.000	0.052	0.332
EVAPOTRANSPIRATION						
TOTALS	1.804	2.054	3.320	3.478	3.347	3.397
	4.115	3.124	2.581	1.624	1.451	1.503

STD. DEVIATIONS	0.238	0.367	0.493	1.009	0.889	1.431
	1.221	1.250	1.283	0.466	0.277	0.210

LATERAL DRAINAGE COLLECTED FROM LAYER 3

TOTALS	2.4517	2.3119	2.5232	1.5007	1.0559	0.2374
	0.6195	0.2009	0.8515	0.8190	1.2369	2.3598
STD. DEVIATIONS	2.1231	1.9048	1.7752	1.1959	1.2013	0.3141
	1.0164	0.2822	1.1846	1.1747	1.4237	2.0578

PERCOLATION/LEAKAGE THROUGH LAYER 5

TOTALS	0.0073	0.0091	0.0071	0.0045	0.0026	0.0001
	0.0011	0.0001	0.0019	0.0023	0.0034	0.0077
STD. DEVIATIONS	0.0110	0.0147	0.0092	0.0072	0.0048	0.0002
	0.0029	0.0001	0.0051	0.0073	0.0059	0.0129

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0071	0.0061	0.0099	0.0059	0.0038	0.0007
	0.0010	0.0001	0.0013	0.0025	0.0029	0.0058
STD. DEVIATIONS	0.0120	0.0090	0.0105	0.0119	0.0072	0.0036
	0.0028	0.0004	0.0034	0.0065	0.0047	0.0068

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.4066	0.5573	0.3900	0.2549	0.1439	0.0033
	0.0567	0.0019	0.1048	0.1296	0.1923	0.4284
STD. DEVIATIONS	0.6219	0.9242	0.5222	0.4192	0.2698	0.0082
	0.1619	0.0026	0.2978	0.4136	0.3404	0.7340

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.60 (6.647)	176413.1	100.00
RUNOFF	0.595 (0.8378)	2160.72	1.225
EVAPOTRANSPIRATION	31.797 (2.9534)	115422.42	65.427
LATERAL DRAINAGE COLLECTED FROM LAYER 3	16.16854 (4.81010)	58691.816	33.26953
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.04707 (0.02786)	170.861	0.09685

```
 AVERAGE HEAD ON TOP              0.222 (    0.137)
   OF LAYER  4

 PERCOLATION/LEAKAGE THROUGH      0.04705 (  0.02790)       170.785     0.09681
   LAYER  6

 CHANGE IN WATER STORAGE         -0.009   (  0.9941)        -32.60     -0.018

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                PEAK DAILY VALUES FOR YEARS    1 THROUGH   30
   ------------------------------------------------------------------------
                                                  (INCHES)      (CU. FT.)
                                                 ----------   -------------
      PRECIPITATION                                 4.71        17097.301

      RUNOFF                                        1.954        7093.9360

      DRAINAGE COLLECTED FROM LAYER  3              0.71877      2609.13184

      PERCOLATION/LEAKAGE THROUGH LAYER  5          0.009840       35.72081

      AVERAGE HEAD ON TOP OF LAYER  4              17.403

      MAXIMUM HEAD ON TOP OF LAYER  4              32.156

      LOCATION OF MAXIMUM HEAD IN LAYER  3
            (DISTANCE FROM DRAIN)                   7.4 FEET

      PERCOLATION/LEAKAGE THROUGH LAYER  6          0.004105       14.90285

      SNOW WATER                                    5.40        19602.5937

      MAXIMUM VEG. SOIL WATER (VOL/VOL)                      0.4662

      MINIMUM VEG. SOIL WATER (VOL/VOL)                      0.1536

       ***  Maximum heads are computed using McEnroe's equations.  ***

            Reference:  Maximum Saturated Depth over Landfill Liner
                        by Bruce M. McEnroe, University of Kansas
                        ASCE Journal of Environmental Engineering
                        Vol. 119, No. 2, March 1993, pp. 262-270.

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                   FINAL WATER STORAGE AT END OF YEAR   30
    ----------------------------------------------------------------------
```

LAYER	(INCHES)	(VOL/VOL)
1	1.1364	0.1894
2	4.3172	0.3598
3	0.0086	0.0430
4	0.0000	0.0000
5	5.4360	0.4530
6	180.1193	0.1900
SNOW WATER	0.000	

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:     c:\MCD1810.D4
TEMPERATURE DATA FILE:       C:\MCD1810.D7
SOLAR RADIATION DATA FILE:   C:\MCD1810.D13
EVAPOTRANSPIRATION DATA:     C:\MCD1810.D11
SOIL AND DESIGN DATA FILE:   C:\MCDCCR2.D10
OUTPUT DATA FILE:            C:\MCDCCR2.OUT


TIME:  12:15     DATE:  11/18/2018


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     TITLE:  Plant McDonough CCR Cover Slope Nov. 2018

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
            COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                         TYPE 1 - VERTICAL PERCOLATION LAYER
                             MATERIAL TEXTURE NUMBER   8
           THICKNESS                 =      6.00   INCHES
           POROSITY                  =      0.4630 VOL/VOL
           FIELD CAPACITY            =      0.2320 VOL/VOL
           WILTING POINT             =      0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT  =    0.1824 VOL/VOL
           EFFECTIVE SAT. HYD. COND.   =  0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------
```

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                TYPE 3 - BARRIER SOIL LINER
                MATERIAL TEXTURE NUMBER   0
   THICKNESS                    =     18.00   INCHES
   POROSITY                     =      0.4710 VOL/VOL
   FIELD CAPACITY               =      0.3420 VOL/VOL
   WILTING POINT                =      0.2100 VOL/VOL
   INITIAL SOIL WATER CONTENT   =      0.4710 VOL/VOL
   EFFECTIVE SAT. HYD. COND.    =  0.999999975000E-05 CM/SEC


                         LAYER  3
                         --------

            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   6
   THICKNESS                    =    948.00   INCHES
   POROSITY                     =      0.4530 VOL/VOL
   FIELD CAPACITY               =      0.1900 VOL/VOL
   WILTING POINT                =      0.0850 VOL/VOL
   INITIAL SOIL WATER CONTENT   =      0.2088 VOL/VOL
   EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

 NOTE:  SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER            =     61.00
FRACTION OF AREA ALLOWING RUNOFF   =    100.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE =      1.000  ACRES
EVAPORATIVE ZONE DEPTH             =      6.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE  =      1.094  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE =      2.778  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE =      0.696  INCHES
INITIAL SNOW WATER                 =      0.000  INCHES
INITIAL WATER IN LAYER MATERIALS   =    207.521  INCHES
TOTAL INITIAL WATER                =    207.521  INCHES
TOTAL SUBSURFACE INFLOW            =      0.00   INCHES/YEAR


           EVAPOTRANSPIRATION AND WEATHER DATA
           -----------------------------------

 NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
         ATLANTA              GEORGIA

     STATION LATITUDE                         =  33.65 DEGREES
     MAXIMUM LEAF AREA INDEX                  =   2.00
     START OF GROWING SEASON (JULIAN DATE)    =     77
     END OF GROWING SEASON (JULIAN DATE)      =    316
     EVAPORATIVE ZONE DEPTH                   =   6.0  INCHES
     AVERAGE ANNUAL WIND SPEED                =   9.10 MPH
     AVERAGE 1ST QUARTER RELATIVE HUMIDITY    =  65.00 %
     AVERAGE 2ND QUARTER RELATIVE HUMIDITY    =  67.00 %
```

AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 76.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 69.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.91	4.43	5.91	4.43	4.02	3.41
4.73	3.41	3.17	2.53	3.43	4.23

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
41.90	44.90	52.50	61.80	69.30	75.80
78.60	78.20	73.00	62.20	52.00	44.50

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA
AND STATION LATITUDE = 33.65 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 2
DRAIN #1: LATERAL DRAINAGE FROM LAYER 1 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 2
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 3

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                              DAILY OUTPUT FOR YEAR    1

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          S
  DAY  A  O  RAIN  RUNOFF   ET   E. ZONE   HEAD      DRAIN     LEAK      HEAD
DRAIN     LEAK
       I  I                       WATER     #1        #1        #1        #2
#2        #2
       R  L   IN.    IN.    IN.  IN./IN.    IN.       IN.       IN.       IN.
IN.       IN.
  ---  -  -  ----- ------ ------ ------- --------- --------- --------- ---------
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    1        0.00  0.000  0.057  0.1729    0.0000 .0000E+00 .0000E+00    0.0000
```

PERC./LEAKAGE THROUGH LAYER 2	16.097435	58433.687	36.51
AVG. HEAD ON TOP OF LAYER 2	0.2481		
PERC./LEAKAGE THROUGH LAYER 3	17.811485	64655.691	40.40
CHANGE IN WATER STORAGE	-1.907	-6923.842	-4.33
SOIL WATER AT START OF YEAR	246.118	893410.125	
SOIL WATER AT END OF YEAR	244.211	886486.312	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.055	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	0.340	0.450	0.497	0.348	0.189	0.000
	0.109	0.000	0.196	0.059	0.193	0.293
STD. DEVIATIONS	0.738	0.921	0.926	0.675	0.381	0.000
	0.327	0.000	0.558	0.219	0.408	0.827
EVAPOTRANSPIRATION						
TOTALS	1.655	1.851	2.761	2.849	2.612	2.943
	3.624	2.600	2.256	1.393	1.430	1.467
STD. DEVIATIONS	0.291	0.466	0.632	0.906	0.776	1.225
	1.036	0.976	1.147	0.493	0.291	0.259
PERCOLATION/LEAKAGE THROUGH LAYER 2						
TOTALS	2.2258	2.2432	2.4603	1.7338	1.3416	0.6587
	1.1720	0.6575	1.3523	0.8995	1.3531	2.3870
STD. DEVIATIONS	1.7759	1.4823	1.4541	1.1805	0.9374	0.5109
	1.1377	0.7004	1.1853	1.0308	1.2602	1.8109

PERCOLATION/LEAKAGE THROUGH LAYER 3

TOTALS	1.1347	1.1664	1.1579	1.4544	1.6286	1.7181
	1.7196	1.6642	1.4776	1.4749	1.3994	1.2645
STD. DEVIATIONS	0.6071	0.5225	0.6145	0.7082	0.7281	0.7019
	0.6029	0.4726	0.4514	0.5870	0.4882	0.4660

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 2

AVERAGES	0.3077	0.4521	0.4447	0.3005	0.2445	0.0789
	0.1929	0.0679	0.2467	0.1377	0.2528	0.3788
STD. DEVIATIONS	0.3048	0.4442	0.3369	0.2691	0.2674	0.0807
	0.2690	0.0964	0.2881	0.2370	0.3022	0.3916

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.60 (6.647)	176413.1	100.00
RUNOFF	2.675 (2.1379)	9711.14	5.505
EVAPOTRANSPIRATION	27.440 (2.7693)	99607.50	56.463
PERCOLATION/LEAKAGE THROUGH LAYER 2	18.48482 (4.51527)	67099.891	38.03566
AVERAGE HEAD ON TOP OF LAYER 2	0.259 (0.084)		
PERCOLATION/LEAKAGE THROUGH LAYER 3	17.26032 (4.89562)	62654.957	35.51604
CHANGE IN WATER STORAGE	1.223 (6.2957)	4439.57	2.517

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.71	17097.301

RUNOFF	3.422	12422.7168
PERCOLATION/LEAKAGE THROUGH LAYER 2	0.453537	1646.34021
AVERAGE HEAD ON TOP OF LAYER 2	6.000	
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.218412	792.83582
SNOW WATER	5.40	19602.5937

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4630
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1160

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	1.0498	0.1750
2	8.4780	0.4710
3	234.6834	0.2476
SNOW WATER	0.000	

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 **              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE              **
 **                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)               **
 **                  DEVELOPED BY ENVIRONMENTAL LABORATORY                  **
 **                    USAE WATERWAYS EXPERIMENT STATION                    **
 **             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
 **                                                                          **
 **                                                                          **
 ******************************************************************************
 ******************************************************************************


 PRECIPITATION DATA FILE:     c:\MCD1118.D4
 TEMPERATURE DATA FILE:       C:\MCD1118.D7
 SOLAR RADIATION DATA FILE:   C:\MCD1118.D13
 EVAPOTRANSPIRATION DATA:     C:\MCD1118.D11
 SOIL AND DESIGN DATA FILE:   C:\MCDMSTDC.D10
 OUTPUT DATA FILE:            C:\MCDTDCT.OUT


 TIME:  21:40     DATE:  11/ 7/2018


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         TITLE:  Plant McDonough Closure Turf & MicroSpike Top Deck

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      NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
               COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER   1
            THICKNESS                  =      0.50   INCHES
            POROSITY                   =      0.4170 VOL/VOL
            FIELD CAPACITY             =      0.0450 VOL/VOL
            WILTING POINT              =      0.0180 VOL/VOL
            INITIAL SOIL WATER CONTENT =      0.0174 VOL/VOL
            EFFECTIVE SAT. HYD. COND.  =  0.999999978000E-02 CM/SEC
          NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                   FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
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              TYPE 2 - LATERAL DRAINAGE LAYER
                MATERIAL TEXTURE NUMBER  34
THICKNESS                    =      0.24   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0071 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   33.0000000000     CM/SEC
SLOPE                        =      3.00   PERCENT
DRAINAGE LENGTH              =    275.0    FEET


                        LAYER  3
                        --------

              TYPE 4 - FLEXIBLE MEMBRANE LINER
                MATERIAL TEXTURE NUMBER  36
THICKNESS                    =      0.04   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                        LAYER  4
                        --------

                TYPE 3 - BARRIER SOIL LINER
                MATERIAL TEXTURE NUMBER   6
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4530 VOL/VOL
FIELD CAPACITY               =      0.1900 VOL/VOL
WILTING POINT                =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.4530 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC


                        LAYER  5
                        --------

            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   6
THICKNESS                    =    948.00   INCHES
POROSITY                     =      0.4530 VOL/VOL
FIELD CAPACITY               =      0.1900 VOL/VOL
WILTING POINT                =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.1900 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	98.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	0.7	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	0.010	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	0.378	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.010	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	185.566	INCHES
TOTAL INITIAL WATER	=	185.566	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ATLANTA GEORGIA

STATION LATITUDE	=	33.65	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	77	
END OF GROWING SEASON (JULIAN DATE)	=	316	
EVAPORATIVE ZONE DEPTH	=	0.7	INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	67.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	76.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	69.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.91	4.43	5.91	4.43	4.02	3.41
4.73	3.41	3.17	2.53	3.43	4.23

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
41.90	44.90	52.50	61.80	69.30	75.80
78.60	78.20	73.00	62.20	52.00	44.50

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ATLANTA GEORGIA
AND STATION LATITUDE = 33.65 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 3
DRAIN #1: LATERAL DRAINAGE FROM LAYER 2 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 4
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 5

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DAILY OUTPUT FOR YEAR 1

DAY	AIR	SOIL	RAIN IN.	RUNOFF IN.	ET IN.	E. ZONE WATER IN./IN.	HEAD #1 IN.	DRAIN #1 IN.	LEAK #1 IN.	HEAD #2 IN.	DRAIN #2 IN.	LEAK #2 IN.
1			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
2			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
3			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
4			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
5			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
6			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
7			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
8			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
9			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
10			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
11			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
12			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
13			0.00	0.000	0.000	0.0143	0.0000	.0000E+00	.0000E+00	0.0000	.0000E+00	.0000E+00
14	*		0.20	0.000	0.044	0.0423	0.0000	.6951E-04	.8305E-08	0.0000	.0000E+00	.0000E+00
15	*		0.03	0.000	0.037	0.0700	0.0000	.3132E-03	.2437E-07	0.0000		

PERC./LEAKAGE THROUGH LAYER 5	0.000802	2.913	0.00
CHANGE IN WATER STORAGE	0.000	-0.388	0.00
SOIL WATER AT START OF YEAR	185.566	673603.875	
SOIL WATER AT END OF YEAR	185.566	673603.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.000	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	1.944	2.393	3.256	2.673	2.075	1.644
	2.488	1.574	2.053	1.058	1.725	2.064
STD. DEVIATIONS	1.510	1.670	1.911	1.793	1.110	0.924
	1.552	1.032	1.677	0.922	1.272	1.711
EVAPOTRANSPIRATION						
TOTALS	0.689	0.736	0.880	0.773	0.729	0.897
	1.229	0.758	0.719	0.422	0.392	0.636
STD. DEVIATIONS	0.272	0.303	0.334	0.420	0.342	0.400
	0.524	0.438	0.394	0.277	0.177	0.190
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	1.4308	1.4512	1.5554	1.3988	1.1311	1.2225
	1.2526	0.9274	1.0182	0.7997	1.1103	1.4921
STD. DEVIATIONS	0.6244	0.5031	0.4960	0.5054	0.3509	0.4463
	0.3745	0.3665	0.4982	0.3888	0.4215	0.6760
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001

	0.0001	0.0000	0.0001	0.0000	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 5

TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0000	0.0000	0.0001	0.0000	0.0000	0.0001
STD. DEVIATIONS	0.0002	0.0002	0.0002	0.0002	0.0003	0.0003
	0.0001	0.0000	0.0002	0.0000	0.0001	0.0003

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3

AVERAGES	0.0023	0.0025	0.0025	0.0023	0.0018	0.0020
	0.0020	0.0015	0.0017	0.0013	0.0018	0.0024
STD. DEVIATIONS	0.0010	0.0009	0.0008	0.0008	0.0006	0.0007
	0.0006	0.0006	0.0008	0.0006	0.0007	0.0011

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.60 (6.647)	176413.1	100.00
RUNOFF	24.948 (4.5447)	90560.19	51.334
EVAPOTRANSPIRATION	8.860 (1.5094)	32162.26	18.231
LATERAL DRAINAGE COLLECTED FROM LAYER 2	14.79008 (1.55873)	53687.984	30.43310
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00076 (0.00008)	2.742	0.00155
AVERAGE HEAD ON TOP OF LAYER 3	0.002 (0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00078 (0.00033)	2.819	0.00160
CHANGE IN WATER STORAGE	0.000 (0.4939)	-0.08	0.000

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.71	17097.301
RUNOFF	4.209	15280.4375
DRAINAGE COLLECTED FROM LAYER 2	0.40115	1456.18042
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000019	0.06897
AVERAGE HEAD ON TOP OF LAYER 3	0.020	
MAXIMUM HEAD ON TOP OF LAYER 3	0.039	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	3.7 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000806	2.92590
SNOW WATER	5.40	19602.5937

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1850
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0143

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	0.0087	0.0174
2	0.0017	0.0071
3	0.0000	0.0000
4	5.4360	0.4530
5	180.1193	0.1900
SNOW WATER	0.000	

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:    c:\MCD1810.D4
TEMPERATURE DATA FILE:      C:\MCD1810.D7
SOLAR RADIATION DATA FILE:  C:\MCD1810.D13
EVAPOTRANSPIRATION DATA:    C:\MCD1810.D11
SOIL AND DESIGN DATA FILE:  C:\MCDMTN18.D10
OUTPUT DATA FILE:           C:\MCDmtn18.OUT


TIME:  22:12     DATE:  11/ 7/2018


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        TITLE:  Plant McDonough Soil Liner Cover Top Deck Nov 2018

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                    LAYER  1
                                    --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER   8
           THICKNESS                    =      6.00   INCHES
           POROSITY                     =      0.4630 VOL/VOL
           FIELD CAPACITY               =      0.2320 VOL/VOL
           WILTING POINT                =      0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.1977 VOL/VOL
           EFFECTIVE SAT. HYD. COND.    =  0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                    FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                    LAYER  2
                                    --------
```

```
              TYPE 1 - VERTICAL PERCOLATION LAYER
                  MATERIAL TEXTURE NUMBER  12
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4710 VOL/VOL
FIELD CAPACITY               =      0.3420 VOL/VOL
WILTING POINT                =      0.2100 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.4385 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC


                        LAYER  3
                        --------

                TYPE 2 - LATERAL DRAINAGE LAYER
                  MATERIAL TEXTURE NUMBER   0
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.8500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   1.04999995000     CM/SEC
SLOPE                        =      3.00   PERCENT
DRAINAGE LENGTH              =    275.0    FEET


                        LAYER  4
                        --------

                TYPE 4 - FLEXIBLE MEMBRANE LINER
                  MATERIAL TEXTURE NUMBER  36
THICKNESS                    =      0.04   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                        LAYER  5
                        --------

                  TYPE 3 - BARRIER SOIL LINER
                  MATERIAL TEXTURE NUMBER   6
THICKNESS                    =     12.00   INCHES
POROSITY                     =      0.4530 VOL/VOL
FIELD CAPACITY               =      0.1900 VOL/VOL
WILTING POINT                =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.4530 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC
```

```
                    LAYER  6
                    --------

              TYPE 1 - VERTICAL PERCOLATION LAYER
                 MATERIAL TEXTURE NUMBER   6
   THICKNESS                   =    948.00   INCHES
   POROSITY                    =      0.4530 VOL/VOL
   FIELD CAPACITY              =      0.1900 VOL/VOL
   WILTING POINT               =      0.0850 VOL/VOL
   INITIAL SOIL WATER CONTENT  =      0.1902 VOL/VOL
   EFFECTIVE SAT. HYD. COND.   =  0.720000011000E-03 CM/SEC



          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

  NOTE:  SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

 SCS RUNOFF CURVE NUMBER             =     61.00
 FRACTION OF AREA ALLOWING RUNOFF    =    100.0    PERCENT
 AREA PROJECTED ON HORIZONTAL PLANE  =      1.000  ACRES
 EVAPORATIVE ZONE DEPTH              =     10.0    INCHES
 INITIAL WATER IN EVAPORATIVE ZONE   =      2.681  INCHES
 UPPER LIMIT OF EVAPORATIVE STORAGE  =      4.662  INCHES
 LOWER LIMIT OF EVAPORATIVE STORAGE  =      1.536  INCHES
 INITIAL SNOW WATER                  =      0.000  INCHES
 INITIAL WATER IN LAYER MATERIALS    =    192.383  INCHES
 TOTAL INITIAL WATER                 =    192.383  INCHES
 TOTAL SUBSURFACE INFLOW             =      0.00   INCHES/YEAR



           EVAPOTRANSPIRATION AND WEATHER DATA
           -----------------------------------

  NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
           ATLANTA               GEORGIA

      STATION LATITUDE                       =  33.65 DEGREES
      MAXIMUM LEAF AREA INDEX                =   2.00
      START OF GROWING SEASON (JULIAN DATE)  =     77
      END OF GROWING SEASON (JULIAN DATE)    =    316
      EVAPORATIVE ZONE DEPTH                 =  10.0  INCHES
      AVERAGE ANNUAL WIND SPEED              =   9.10 MPH
      AVERAGE 1ST QUARTER RELATIVE HUMIDITY  =  65.00 %
      AVERAGE 2ND QUARTER RELATIVE HUMIDITY  =  67.00 %
      AVERAGE 3RD QUARTER RELATIVE HUMIDITY  =  76.00 %
      AVERAGE 4TH QUARTER RELATIVE HUMIDITY  =  69.00 %


  NOTE:  PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
           COEFFICIENTS FOR    ATLANTA             GEORGIA

           NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL     FEB/AUG     MAR/SEP     APR/OCT     MAY/NOV     JUN/DEC
-------     -------     -------     -------     -------     -------
  4.91        4.43        5.91        4.43        4.02        3.41
```

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.09	160046.703	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	30.436	110482.867	69.03
DRAINAGE COLLECTED FROM LAYER 3	14.3803	52200.496	32.62
PERC./LEAKAGE THROUGH LAYER 5	0.353109	1281.784	0.80
AVG. HEAD ON TOP OF LAYER 4	1.6781		
PERC./LEAKAGE THROUGH LAYER 6	0.408814	1483.997	0.93
CHANGE IN WATER STORAGE	-1.135	-4120.639	-2.57
SOIL WATER AT START OF YEAR	192.177	697602.625	
SOIL WATER AT END OF YEAR	191.042	693482.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.024	0.00

****

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	0.337	0.426	0.376	0.095	0.057	0.000
	0.018	0.000	0.094	0.047	0.025	0.310
STD. DEVIATIONS	0.725	1.161	0.852	0.267	0.232	0.000
	0.100	0.000	0.356	0.236	0.081	0.926
EVAPOTRANSPIRATION						
TOTALS	1.805	2.056	3.324	3.513	3.400	3.376
	4.096	3.092	2.592	1.629	1.452	1.500

```
  STD. DEVIATIONS        0.240    0.382    0.491    1.006    0.878    1.421
                         1.250    1.241    1.278    0.455    0.276    0.210

LATERAL DRAINAGE COLLECTED FROM LAYER  3
----------------------------------------
  TOTALS                 2.1867   1.7739   2.4443   1.4374   1.1196   0.3532
                         0.5719   0.2762   0.7356   0.7892   1.0003   1.9046

  STD. DEVIATIONS        1.2436   1.0482   1.1582   0.9844   1.0765   0.5011
                         0.7915   0.4120   0.9046   0.9244   1.0292   1.4123

PERCOLATION/LEAKAGE THROUGH LAYER  5
------------------------------------
  TOTALS                 0.0730   0.0561   0.0852   0.0365   0.0282   0.0031
                         0.0102   0.0022   0.0156   0.0165   0.0274   0.0705

  STD. DEVIATIONS        0.0840   0.0566   0.0757   0.0429   0.0406   0.0126
                         0.0247   0.0069   0.0320   0.0394   0.0460   0.0816

PERCOLATION/LEAKAGE THROUGH LAYER  6
------------------------------------
  TOTALS                 0.0342   0.0247   0.0366   0.0286   0.0449   0.0492
                         0.0486   0.0447   0.0329   0.0289   0.0296   0.0286

  STD. DEVIATIONS        0.0389   0.0253   0.0291   0.0239   0.0313   0.0303
                         0.0376   0.0419   0.0451   0.0387   0.0404   0.0332


-------------------------------------------------------------------------------
              AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
-------------------------------------------------------------------------------


DAILY AVERAGE HEAD ON TOP OF LAYER  4
-------------------------------------
  AVERAGES               4.1240   3.4706   4.8135   2.1154   1.5814   0.1732
                         0.5631   0.1184   0.8975   0.9192   1.5931   3.9902

  STD. DEVIATIONS        4.7968   3.5126   4.3205   2.5193   2.2937   0.7319
                         1.3880   0.3826   1.8750   2.2362   2.7011   4.6477

*******************************************************************************


*******************************************************************************

    AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS    1 THROUGH   30
-------------------------------------------------------------------------------
                                     INCHES              CU. FEET       PERCENT
                               -------------------    -------------   ---------
 PRECIPITATION                  48.60     (   6.647)     176413.1     100.00

 RUNOFF                          1.784    (  1.9302)       6476.96      3.671

 EVAPOTRANSPIRATION             31.835    (  2.9511)     115559.42     65.505

 LATERAL DRAINAGE COLLECTED     14.59292 (  3.47176)     52972.297   30.02741
   FROM LAYER  3

 PERCOLATION/LEAKAGE THROUGH     0.42464 (  0.19292)      1541.441     0.87377
   LAYER  5
```

```
 AVERAGE HEAD ON TOP             2.030 (    0.938)
   OF LAYER  4

 PERCOLATION/LEAKAGE THROUGH     0.43162 (  0.16306)      1566.789     0.88814
   LAYER  6

 CHANGE IN WATER STORAGE        -0.045   (  1.4563)       -162.32     -0.092
******************************************************************************


*****************************************************************************

               PEAK DAILY VALUES FOR YEARS    1 THROUGH   30
   ------------------------------------------------------------------------
                                                (INCHES)      (CU. FT.)
                                               ----------   -------------
      PRECIPITATION                               4.71         17097.301

      RUNOFF                                      2.151         7806.7280

      DRAINAGE COLLECTED FROM LAYER  3            0.13372        485.42148

      PERCOLATION/LEAKAGE THROUGH LAYER  5        0.010297        37.37726

      AVERAGE HEAD ON TOP OF LAYER  4            18.200

      MAXIMUM HEAD ON TOP OF LAYER  4            24.992

      LOCATION OF MAXIMUM HEAD IN LAYER  3
            (DISTANCE FROM DRAIN)                86.0 FEET

      PERCOLATION/LEAKAGE THROUGH LAYER  6        0.006188        22.46172

      SNOW WATER                                  5.40         19602.5937


      MAXIMUM VEG. SOIL WATER (VOL/VOL)                   0.4662

      MINIMUM VEG. SOIL WATER (VOL/VOL)                   0.1536

       ***  Maximum heads are computed using McEnroe's equations.  ***

            Reference:  Maximum Saturated Depth over Landfill Liner
                        by Bruce M. McEnroe, University of Kansas
                        ASCE Journal of Environmental Engineering
                        Vol. 119, No. 2, March 1993, pp. 262-270.

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                  FINAL WATER STORAGE AT END OF YEAR   30
    ----------------------------------------------------------------------
```

LAYER	(INCHES)	(VOL/VOL)
1	1.1427	0.1905
2	4.2937	0.3578
3	0.0500	0.2500
4	0.0000	0.0000
5	5.4360	0.4530
6	180.1195	0.1900
SNOW WATER	0.000	

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****

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******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     c:\MCD1810.D4
TEMPERATURE DATA FILE:       C:\MCD1810.D7
SOLAR RADIATION DATA FILE:   C:\MCD1810.D13
EVAPOTRANSPIRATION DATA:     C:\MCD1810.D11
SOIL AND DESIGN DATA FILE:   C:\MCDCCR3.D10
OUTPUT DATA FILE:            C:\MCDCCR3.OUT


TIME:  12:42     DATE:  11/18/2018


******************************************************************************

     TITLE:  Plant McDonough CCR Cover Top Deck Nov 2018

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
               COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   8
           THICKNESS                  =      6.00   INCHES
           POROSITY                   =      0.4630 VOL/VOL
           FIELD CAPACITY             =      0.2320 VOL/VOL
           WILTING POINT              =      0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT =      0.1824 VOL/VOL
           EFFECTIVE SAT. HYD. COND.  =  0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
                  FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------
```

```
                TYPE 3 - BARRIER SOIL LINER
                MATERIAL TEXTURE NUMBER   0
   THICKNESS                    =     18.00   INCHES
   POROSITY                     =      0.4710 VOL/VOL
   FIELD CAPACITY               =      0.3420 VOL/VOL
   WILTING POINT                =      0.2100 VOL/VOL
   INITIAL SOIL WATER CONTENT   =      0.4710 VOL/VOL
   EFFECTIVE SAT. HYD. COND.    =  0.999999975000E-05 CM/SEC


                          LAYER  3
                          --------

            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   6
   THICKNESS                    =    948.00   INCHES
   POROSITY                     =      0.4530 VOL/VOL
   FIELD CAPACITY               =      0.1900 VOL/VOL
   WILTING POINT                =      0.0850 VOL/VOL
   INITIAL SOIL WATER CONTENT   =      0.2088 VOL/VOL
   EFFECTIVE SAT. HYD. COND.    =  0.720000011000E-03 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

 NOTE:  SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER              =     61.00
FRACTION OF AREA ALLOWING RUNOFF     =    100.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE   =      1.000  ACRES
EVAPORATIVE ZONE DEPTH               =      6.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE    =      1.094  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE   =      2.778  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE   =      0.696  INCHES
INITIAL SNOW WATER                   =      0.000  INCHES
INITIAL WATER IN LAYER MATERIALS     =    207.521  INCHES
TOTAL INITIAL WATER                  =    207.521  INCHES
TOTAL SUBSURFACE INFLOW              =      0.00   INCHES/YEAR


           EVAPOTRANSPIRATION AND WEATHER DATA
           -----------------------------------

 NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
          ATLANTA               GEORGIA

     STATION LATITUDE                          =  33.65 DEGREES
     MAXIMUM LEAF AREA INDEX                   =   2.00
     START OF GROWING SEASON (JULIAN DATE)     =     77
     END OF GROWING SEASON (JULIAN DATE)       =    316
     EVAPORATIVE ZONE DEPTH                    =   6.0  INCHES
     AVERAGE ANNUAL WIND SPEED                 =   9.10 MPH
     AVERAGE 1ST QUARTER RELATIVE HUMIDITY     =  65.00 %
     AVERAGE 2ND QUARTER RELATIVE HUMIDITY     =  67.00 %
```

AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 76.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 69.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.91	4.43	5.91	4.43	4.02	3.41
4.73	3.41	3.17	2.53	3.43	4.23

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ATLANTA GEORGIA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
41.90	44.90	52.50	61.80	69.30	75.80
78.60	78.20	73.00	62.20	52.00	44.50

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ATLANTA GEORGIA AND STATION LATITUDE = 33.65 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 2
DRAIN #1: LATERAL DRAINAGE FROM LAYER 1 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 2
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 3

****

DAILY OUTPUT FOR YEAR 1

DAY	A I R	S O I L	RAIN IN.	RUNOFF IN.	ET IN.	E. ZONE WATER IN./IN.	HEAD #1 IN.	DRAIN #1 IN.	LEAK #1 IN.	HEAD #2 IN.	DRAIN #2 IN.	LEAK #2 IN.
1			0.00	0.000	0.057	0.1729	0.0000	.0000E+00	.0000E+00	0.0000		

PERC./LEAKAGE THROUGH LAYER 2	16.097435	58433.687	36.51
AVG. HEAD ON TOP OF LAYER 2	0.2481		
PERC./LEAKAGE THROUGH LAYER 3	17.811485	64655.691	40.40
CHANGE IN WATER STORAGE	-1.907	-6923.842	-4.33
SOIL WATER AT START OF YEAR	246.118	893410.125	
SOIL WATER AT END OF YEAR	244.211	886486.312	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.055	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.99	4.61	5.67	4.85	3.91	3.79
	4.96	3.26	3.78	2.27	3.25	4.24
STD. DEVIATIONS	2.19	2.21	2.45	2.59	1.61	1.63
	2.13	1.71	2.38	1.38	1.73	2.34
RUNOFF						
TOTALS	0.340	0.450	0.497	0.348	0.189	0.000
	0.109	0.000	0.196	0.059	0.193	0.293
STD. DEVIATIONS	0.738	0.921	0.926	0.675	0.381	0.000
	0.327	0.000	0.558	0.219	0.408	0.827
EVAPOTRANSPIRATION						
TOTALS	1.655	1.851	2.761	2.849	2.612	2.943
	3.624	2.600	2.256	1.393	1.430	1.467
STD. DEVIATIONS	0.291	0.466	0.632	0.906	0.776	1.225
	1.036	0.976	1.147	0.493	0.291	0.259
PERCOLATION/LEAKAGE THROUGH LAYER 2						
TOTALS	2.2258	2.2432	2.4603	1.7338	1.3416	0.6587
	1.1720	0.6575	1.3523	0.8995	1.3531	2.3870
STD. DEVIATIONS	1.7759	1.4823	1.4541	1.1805	0.9374	0.5109
	1.1377	0.7004	1.1853	1.0308	1.2602	1.8109

PERCOLATION/LEAKAGE THROUGH LAYER 3

TOTALS	1.1347	1.1664	1.1579	1.4544	1.6286	1.7181
	1.7196	1.6642	1.4776	1.4749	1.3994	1.2645
STD. DEVIATIONS	0.6071	0.5225	0.6145	0.7082	0.7281	0.7019
	0.6029	0.4726	0.4514	0.5870	0.4882	0.4660

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 2

AVERAGES	0.3077	0.4521	0.4447	0.3005	0.2445	0.0789
	0.1929	0.0679	0.2467	0.1377	0.2528	0.3788
STD. DEVIATIONS	0.3048	0.4442	0.3369	0.2691	0.2674	0.0807
	0.2690	0.0964	0.2881	0.2370	0.3022	0.3916

****

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	48.60	(6.647)	176413.1	100.00
RUNOFF	2.675	(2.1379)	9711.14	5.505
EVAPOTRANSPIRATION	27.440	(2.7693)	99607.50	56.463
PERCOLATION/LEAKAGE THROUGH LAYER 2	18.48482	(4.51527)	67099.891	38.03566
AVERAGE HEAD ON TOP OF LAYER 2	0.259	(0.084)		
PERCOLATION/LEAKAGE THROUGH LAYER 3	17.26032	(4.89562)	62654.957	35.51604
CHANGE IN WATER STORAGE	1.223	(6.2957)	4439.57	2.517

****

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.71	17097.301

```
     RUNOFF                                        3.422        12422.7168

     PERCOLATION/LEAKAGE THROUGH LAYER  2          0.453537      1646.34021

     AVERAGE HEAD ON TOP OF LAYER  2               6.000

     PERCOLATION/LEAKAGE THROUGH LAYER  3          0.218412       792.83582

     SNOW WATER                                    5.40         19602.5937

     MAXIMUM VEG. SOIL WATER (VOL/VOL)                     0.4630

     MINIMUM VEG. SOIL WATER (VOL/VOL)                     0.1160
******************************************************************************



******************************************************************************
                  FINAL WATER STORAGE AT END OF YEAR   30
    ----------------------------------------------------------------------
                   LAYER       (INCHES)       (VOL/VOL)
                   -----       --------       ---------
                     1           1.0498         0.1750

                     2           8.4780         0.4710

                     3         234.6834         0.2476

                  SNOW WATER     0.000

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APPENDIX G

Veneer Stability Analyses Calculation Package

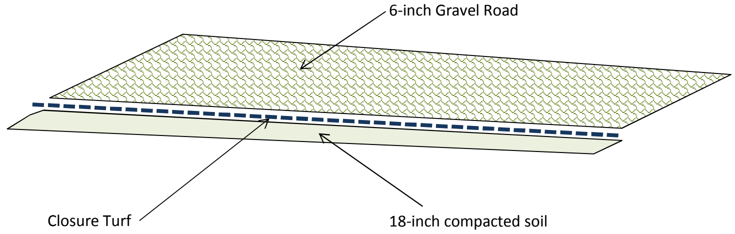
GOLDER	SUBJECT: Stability of Cover System - Veneer Stability				
	Job No. 1777449	Prepared by	DM		
	Ref. : Plant McDonough-Atkinson Closed CCR Surface Impoundment Units AP-1 and AP-3/4	Checked by	LJ / LS	Date	7/19/2018
		Reviewed by	GLH		

OBJECTIVE:

Analyze the stability of the cover system for the closed conditions of CCR surface impoundments AP-1 and AP-3/4.
Use design strength parameters and analyze for conditions with and without seepage forces.

GEOMETRY (Final Cover System):

Slope is 6 H:1V Maximum Road Grade is 10%



GOLDER RECOMMENDED FACTORS OF SAFETY FOR LANDFILL (CLOSED CCR SURFACE IMPOUNDMENT) FINAL COVER

Shear Strength	Long Term (w/ Seepage)	Long Term [b]
Design	N/A [a]	1.5

[a] The gravel road is comprised of free draining No. 89 Stone.
[b] Recommended factor of safety with and w/out vehicle loading

If the calculated factors of safety based on the final cover conditions are higher than the recommended factors of safety, the stability of the final cover meets the requirement.

Based on Proposed Final Grades (representative of AP-1 and AP-3/4 closed unit conditions):

Top Elevation of Final Grades: 896 ft
Approx. Internal Toe Elevation: 844 ft
* These apply to the condition of roads placed on top of closure turf

Material Properties (ref 4)

Material	c (psf)	c_a (psf)	ϕ (°)	δ (°)	γ (pcf)	Thickness (ft)
Gravel Road (GM)[(1)]	0	-	36	-	130	0.50
Closure Turf [(2)]	-	0	-	27	-	0.03

[(1)] Used gravel material properties based on past experience with similar type of material.

[(2)] Conservaitvely downgraded interface stregnth as 75% of gravel material properties.

Where:
- c = Cohesion of the protective cover soil
- c_a = Adhesion between protective cover soil of the active wedge and the geomembrane
- δ = Interface friction angle between cover soil and geomembrane
- ϕ = Friction Angle of protective cover soil
- γ = Unit weight of the protective cover soil

Slope Angle =	β (°) =	10.0	
Slope Height =	52	ft	(H)

CALCULATIONS:

LONG TERM VENEER STABILITY based on Koerner/Soong Method (page 487 to 490, ref. 1)

Using the Koerner/Soong Method, the factor of safety is calculated using the following equation (Eq. 13.9, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_a - N_a \times \cos\beta)\cos\beta$

$b = -[(W_a - N_a \times \cos\beta) \times \sin\beta \tan\phi + (N_a \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan\phi) \times \sin\beta]$

$c = (N_a \times \tan\delta + C_a) \times \sin^2\beta \times \tan\phi$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

$N_a = W_a \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

Where:

W_a = Total weight of the active wedge

N_a= Effective force normal to the failure plane of the active wedge
C_a = Adhesive force between protective cover soil of the active wedge and the geomembrane
W_p = Total weight of the passive wedge
C = Cohesive force along the failure plane of the passive wedge
γ = Unit Weight of protective cover soil
h = Thickness of protective cover soil
β = Slope Angle
L = Length of slope measured along the geosynthetic interface
c = Cohesion of the protective cover soil
c_a = Adhesion between protective cover soil of active wedge and geomembrane
δ = Interface friction angle between protective cover soil and geomembrane
ϕ = Friction Angle of protective cover soil

Where:
h = Thickness of Prot. Cover (ft) = 0.50
β = Cover Slope Angle (°) = 10.0
H_{max} = Maximum height = 52.0 feet
L= 300.9 feet

Since h and L are known for **LONG-TERM Conditions,** solve for the FS:

W_a (lbs/ft) = 19,369
N_a (lbs/ft) = 19,078
C_a (lbs/ft) = 298 x c_a

W_p (lbs/ft) = 95
C (lbs/ft) = 0

$(W_a - N_a \times \cos\beta)$ = 578
$(C + W_p \times \tan\phi)$ = 69
cos β = 0.98
sin β = 0.17
sin β x tan ϕ = 0.13
$\sin^2\beta \times \tan\phi$ = 0.02
sin β x cos β = 0.17
tan ϕ = 0.73

a= 569.7

Solve for FS with different combinations of δ an c_a:

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	(N_a x tan δ + C_a)	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
27.0	*0*	*0.510*	*0*	*9,721*	*-1,739*	*211*	*1594.9*	*2.9*

VEHICLE LOADING ON ROAD CONDITIONS (***Dozer on the slope with acceleration)***

Veneer Stability based on Koerner/Soong Method (page 490-497, ref. 1) for the case of vehicle loading acceleration

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

a = $(W_{a+e} \times \sin\beta + Fe) \cos\beta$
b = $-[(Na{+}e \times \tan\delta + Ca) \times \cos\beta + [(W_{a+e} \times \sin\beta + Fe) \times \sin\beta \times \tan\phi] + (C + W_p \times \tan\phi)]$
c = $(N_{a+e} \times \tan\delta + C_a) \times \sin\beta \times \tan\phi$

Fe = We x (a/g) - Dynamic force per unit width parallel to the slope
a = acceleration of the construction equipment
g = acceleration due to gravity
$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$
W_e = Equivalent Equipment Force per unit width at geomembrane interface
$W_{a+e} = W_a + W_e$
$N_{a+e} = W_{a+e} \times \cos\beta$
$C_a = c_a \times (L - h/\sin\beta)$
$W_p = (\gamma \times h^2) / \sin 2\beta$
C = c x h / sin β
The definitions of all the parameters are as same as those in long term FS calculation except W_e, W_{a+e}, and N_{a+e}

$L_{short\ term}$= 300.9 ft
$h_{short\ term}$= 0.50 ft
ϕ = 36.00 degrees

c = 0.00 psf
$\gamma_{soil\ cover}$ = 130.00 pcf

Determination of W_e (See dozer specifications from manufacturer, ref. 2):

Specifications for D6H LGP Series II Crawler Tractor

Width of Dozer Track =	3.00	ft
Contact Area =	64.26	sq.ft.
Ground Pressure =	4.8	psi
Influence factor (I) =	1.00	(obtained from Figure 13.7, page 493, ref. 2)
Ground Pressure at Geosynthetics =	686.7	psf
Length of Dozer Track =	10.7	ft
W_e=	**7355**	**lbs/ft**

W_a+W_e (lbs/ft) =	26,725		
N_{a+e} (lbs/ft) =	26,322	W_p (lbs/ft) =	95
C_a (lbs/ft) =	298 x c_a	C (lbs/ft) =	0
(W_{a+e} - N_{a+e} x cos β) =	798	(Wa+e x sin β + Fe) =	6,825
(C + W_p x tan ϕ) =	69	(C + W_p x tan ϕ) =	0
cos β =	0.98	cos β =	0.98
sin β =	0.17	sin β =	0.17
sin β x tan ϕ =	0.13	sin β x tan ϕ =	0.13
$\sin^2$ β x tan ϕ =	0.02	$\sin^2$ β x tan ϕ =	0.02
sin β x cos β =	0.17	sin β x cos β =	0.17
tan ϕ =	0.73	tan ϕ =	0.73
		a =	0.30 **g (from Figure 13.9)**
a=	6721.9	Fe =	2206.55 **lbs/ft**

Solve for FS :

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	(N_{a+e} x tan δ + C_a)	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
27.0	*0*	*0.510*	0.00	*13,412*	*-14,067*	*291*	*13,786*	*2.1*

SUMMARY OF RESULTS

CASE ANALYZED	REQUIRED FACTOR OF SAFETY	ACTUAL FACTOR OF SAFETY	MEET REQUIREMENT
Long Term using Design Shear Strength	1.5	2.9	Yes
Long Term using Design Shear Strength - Dozer on Road w/ acceleration	1.5	2.1	Yes

The stability of the final cover system meets the recommended factors of safety. These results are based on strength parameters for the soils encountered on site during Golder's geotechnical investigation.

References:

1. Qian, X., Koerner, R. M., Gray, D. H., Geotechnical Aspects of Landfill Design and Construction, Prentice Hall, New Jersey, US, 2002.

2. Dozer Specifications from Manufacturer

3. Golder Associates Inc., Unpublished Database of Direct Shear Laboratory Results.

ATTACHMENT A

VENEER STABILITY REFERENCE INFORMATION

13.4 VENEER SLOPE STABILITY ANALYSES

This section treats the standard veneer slope stability problem [as shown in Figure 13.1(a) and (b)] and then superimposes upon it a number of situations, all which tend to destabilize slopes. Included are gravitational, construction equipment, seepage and seismic forces, respectively. Each will be illustrated by a design graph and a numeric example.

13.4.1 Cover Soil (Gravitational) Forces

Figure 13.3 illustrates the common situation of a finite-length, uniformly-thick cover soil placed over a liner material at a slope angle β. It includes a passive wedge at the toe and has a tension crack on the crest. The analysis that follows is from Koerner and Soong (1998), but it is similar to Koerner and Hwu (1991). Comparable analyses are also available from Giroud and Beech (1989), McKelvey and Deutsch (1991), and others.

The symbols used in Figure 13.3 are defined a follows:

W_A = total weight of the active wedge
W_P = total weight of the passive wedge
N_A = effective force normal to the failure plane of the active wedge
N_P = effective force normal to the failure plane of the passive wedge
γ = unit weight of the cover soil
h = thickness of the cover soil
L = length of slope measured along the geomembrane
β = soil slope angle beneath the geomembrane

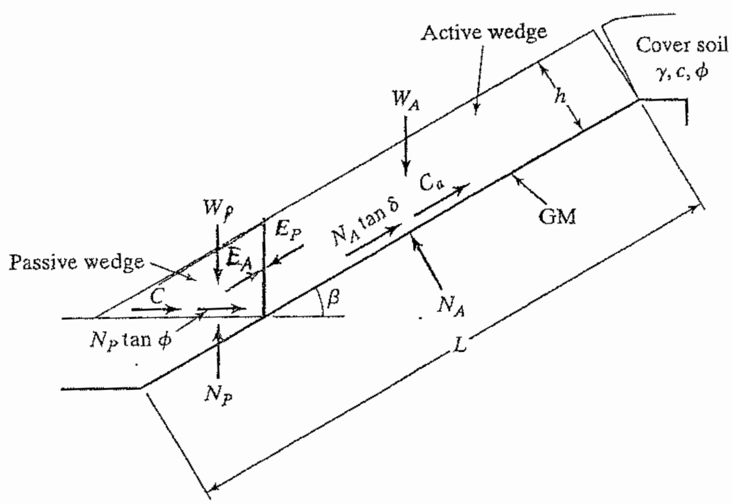


FIGURE 13.3 Limit Equilibrium Forces Involved in a Finite Length Slope Analysis for a Uniformly Thick Cover Soil

ϕ = friction angle of the cover soil
δ = interface friction angle between cover soil and geomembrane
C_a = adhesive force between cover soil of the active wedge and the geomembrane
c_a = adhesion between cover soil of the active wedge and the geomembrane
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil
E_A = interwedge force acting on the active wedge from the passive wedge
E_P = interwedge force acting on the passive wedge from the active wedge
FS = factor of safety against cover soil sliding on the geomembrane.

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge, the forces acting on it are

$$W_A = \gamma \cdot h^2 \cdot (L/h - 1/\sin\beta - \tan\beta/2) \tag{13.4}$$

$$N_A = W_A \cdot \cos\beta \tag{13.5}$$

$$C_a = c_a \cdot (L - h/\sin\beta) \tag{13.6}$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \cdot \sin\beta = (W_A - N_A \cdot \cos\beta) - \frac{(N_A \cdot \tan\delta + C_a)}{FS}$$

Hence, the interwedge force acting on the active wedge is

$$E_A = \frac{(FS)(W_A - N_A \cdot \cos\beta) - (N_A \cdot \tan\delta + C_a) \cdot \sin\beta}{\sin\beta \cdot (FS)} \tag{13.7}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta}$$

$$N_P = W_P + E_P \cdot \sin\beta$$

$$C = \frac{c \cdot h}{\sin\beta} \tag{13.8}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cdot \cos\beta = \frac{C + N_P \cdot \tan\phi}{FS}$$

Hence, the interwedge force acting on the passive wedge is

$$E_P = \frac{C + W_P \cdot \tan\phi}{\cos\beta \cdot (FS) - \sin\beta \cdot \tan\phi}$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$, which in this case, using FS-values, results in

$$a \cdot FS^2 + b \cdot FS + c = 0$$

The resulting FS-value is then obtained from the conventional solution of the quadratic equation, which gives

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.9}$$

where
$$\begin{aligned} a &= (W_A - N_A \cdot \cos\beta) \cdot \cos\beta \\ b &= -[(W_A - N_A \cdot \cos\beta) \cdot \sin\beta \cdot \tan\phi + (N_A \cdot \tan\delta + C_a) \cdot \sin\beta \cdot \cos\beta \\ &\quad + (C + W_P \cdot \tan\phi) \cdot \sin\beta] \\ c &= (N_A \cdot \tan\delta + C_a) \cdot \sin^2\beta \cdot \tan\phi \end{aligned}$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated. Thus, a value of greater than 1.0 must be targeted as being the minimum factor of safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. Recommendations for minimum allowable FS-values under different conditions are available in Koerner and Soong (1998). In order to better illustrate the implications of Equations 13.9, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 13.4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example 13.1 illustrates the use of the analytic development and the

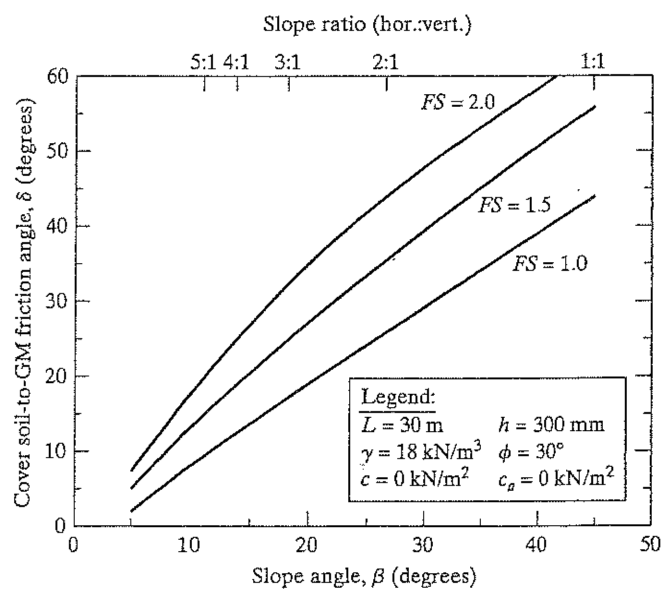


FIGURE 13.4 Design Curves for Stability of Uniform-Thickness Cohesionless Cover Soils on Linear Failure Planes for Various Global Factors of Safety

will be considered as compared.

EXAMPLE 13.1

The following are given: a 30-m slope with a uniformly thick 300-mm-deep cover soil at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). The cover soil is placed directly on a geomembrane as shown in Figure 13.3. Direct shear testing has resulted in an interface friction angle between the cover soil and geomembrane of 22° with zero adhesion. What is the *FS*-value at a slope angle of 3(H)-to-1(V) (i.e., 18.4°)?

Solution Using Equation 13.9 to solve for the *FS*-value results in a value of 1.25, which is seen to be in agreement with the curves of Figure 13.4:

$$a = 14.7 \text{ kN/m}$$

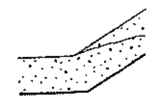
$$b = -21.3 \text{ kN/m}$$

$$c = 3.5 \text{ kN/m}$$

Thus, $FS = 1.25$

This value can be confirmed using Figure 13.4.

Comment In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. There are many possible options to increase the value (e.g., changing the geometry of the situation, the use of toe berms, tapered cover soil thickness, and veneer reinforcement, see Koerner and Soong, 1998). Nevertheless, this general problem will be used throughout this section for comparison with other cover soil slope stability situations.



13.4.2 Tracked Construction Equipment Forces

The placement of cover soil on a slope with a relatively low shear strength interface (like a geomembrane) should always start at the toe and move upward to the crest. Figure 13.5(a) shows the recommended method. In doing so, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever-present passive wedge and a stable lower portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction in the *FS*-value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the *FS*-value—in some cases, to a great extent. Figure 13.5(b) shows this procedure. Unless absolutely necessary, it is not recommended that cover soil be placed on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the possible dynamic force of the specific type of construction equipment and its manner of operation.

For the *first case* of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 13.6(a). The analysis uses a

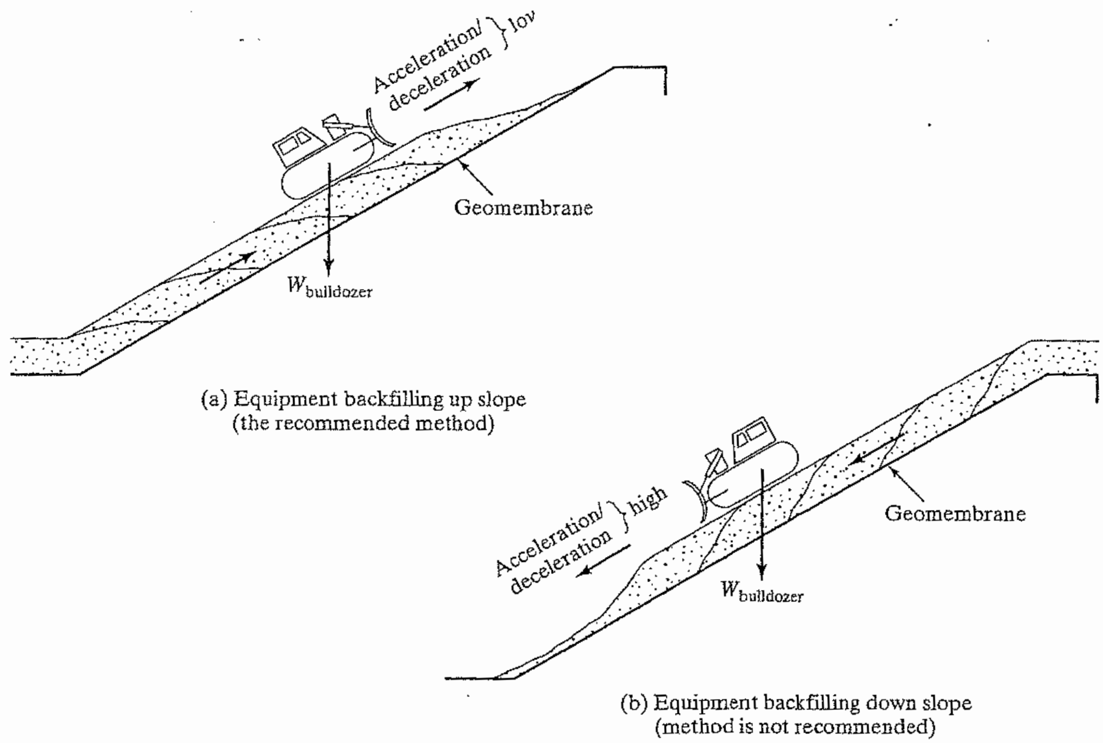


FIGURE 13.5 Construction Equipment Placing Cover Soil on Slopes Containing Geosynthetics

known type of construction equipment (such as a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used (see Poulos and Davis, 1974). This results in an equipment force per unit width of

$$W_e = q \cdot w \cdot I \tag{13.10}$$

where W_e = equivalent equipment force per unit width at the geomembrane interface;
$q = W_b/(2 \cdot w \cdot b)$;
W_b = actual weight of equipment (e.g., a bulldozer);
w = length of equipment track;
b = width of equipment track;
I = influence factor at the geomembrane interface (see Figure 13.7).

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in Section 13.3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term (W_e) to the W_A-force in Equation 13.4. Note, however, that this involves the generation of a resisting force as well. Thus the net effect of increasing the driving force

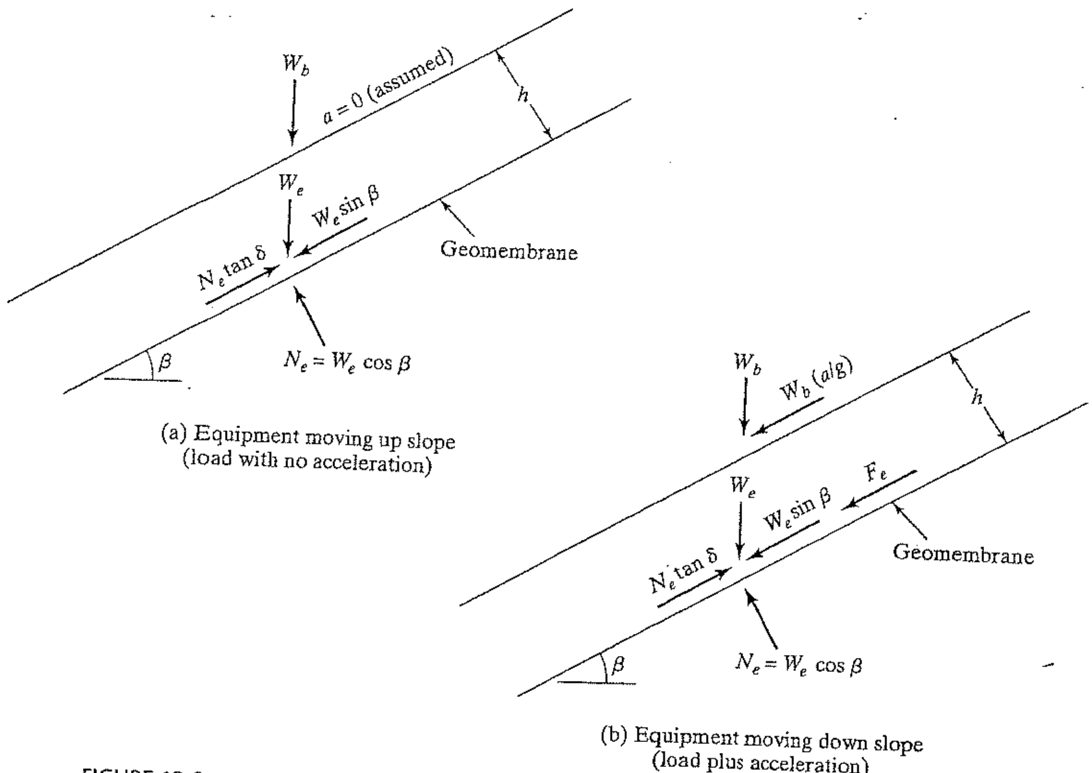


FIGURE 13.6 Additional (to Gravitational Forces) Limit Equilibrium Forces due to Construction Equipment Moving on Cover Soil (see Figure 13.3 for the gravitational soil force to which the above forces are added).

concerned. It should also be noted that no acceleration/deceleration forces are included in this analysis, which is somewhat idealistic. Using these concepts (the same equations used in Section 13.3.1 are used here), typical design curves for various *FS*-values as a function of equivalent ground contact equipment pressures and cover soil thicknesses are given in Figure 13.8. Note that the curves are developed specifically for the variables stated in the legend. Example 13.2 illustrates the use of the formulation.

EXAMPLE 13.2

The following are given: a 30-m-long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22° with zero adhesion. What is the *FS*-value at a slope angle 3(H)-to-1(V)(i.e., 18.4°)?

Solution This problem follows Example 13.1 exactly except for the addition of the bulldozer moving up the slope. Using the additional equipment load, Equation 13.10 substituted into Equation 13.9 results in the following:

$$a = 73.1 \text{ kN/m}$$
$$b = -104.3 \text{ kN/m}$$
$$c = 17.0 \text{ kN/m}$$

Thus, $FS = 1.24$

This value can be confirmed using Figure 13.8.

Comment While the resulting FS-value is still low, the result is important to assess by comparing it with Example 13.1 (i.e., the same problem except without the bulldozer). It is seen that the FS-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope with negligible acceleration/deceleration forces does not significantly decrease the factor-of-safety.

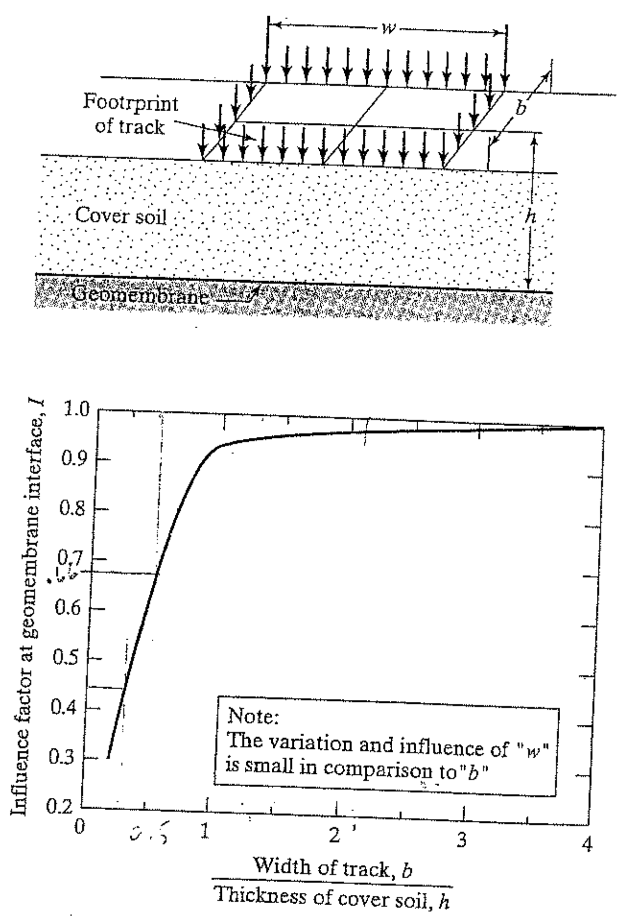


FIGURE 13.7 Values of Influence Factor, "I", for Use in Equation 13.10 to Dissipate Surface Force through the Cover Soil to the Geomembrane Interface (after Soong and Koerner, 1996)

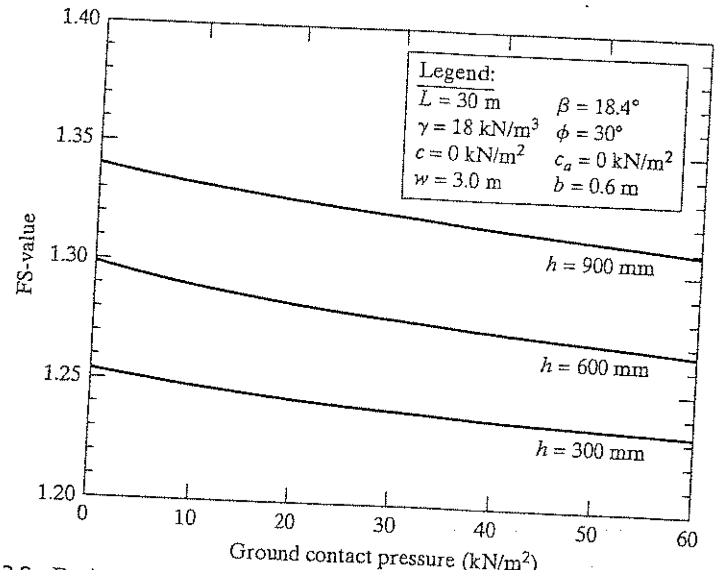


FIGURE 13.8 Design Curves for Stability of Different Thickness of Cover Soil for Various Construction Equipment Ground Contact Pressure

For the *second case* of a bulldozer pushing cover soil down from the crest of the slope to the toe as shown in Figure 13.5b, the analysis uses the force diagram of Figure 13.6(b). While the weight of the equipment is treated as just described, the lack of a passive wedge along with an additional force due to acceleration (or deceleration) of the equipment significantly decreases the resulting *FS*-values. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to $W_b \cdot (a/g)$, where W_b = the weight of the bulldozer, a = acceleration of the bulldozer, and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed (see Figure 13.9).

The acceleration of the bulldozer, coupled with an influence factor I from Figure 13.7, results in the dynamic force per unit width at the cover soil to geomembrane interface F_e. The relationship is given by

$$F_e = W_e \cdot (a/g) \tag{13.11}$$

where F_e = dynamic force per unit width parallel to the slope at the geomembrane interface;
W_e = equivalent equipment (e.g., bulldozer) force per unit width at geomembrane interface, recall Equation 13.10;
β = soil slope angle beneath geomembrane;
a = acceleration of the construction equipment;
g = acceleration due to gravity.

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover soil to the interface of the geomembrane. Again, a

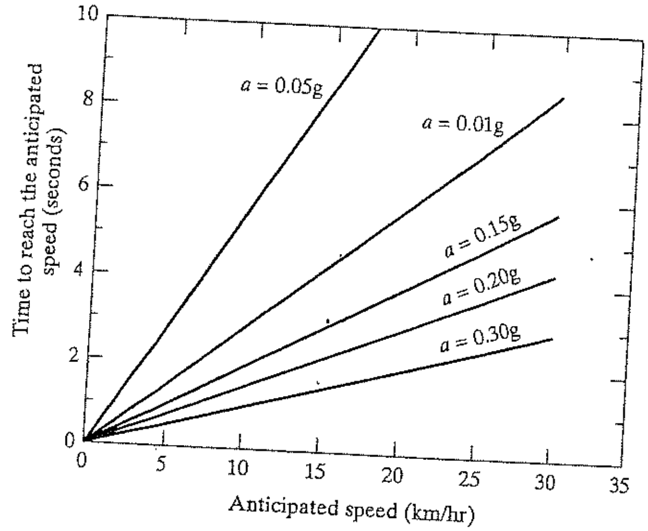


FIGURE 13.9 Graphic Relationship of Construction Equipment Speed and Rise Time to Obtain Equipment Acceleration.

Boussinesq analysis is used (see Poulos and Davis, 1974). The expression for determining the *FS*-value is derived next.

Considering the active wedge and balancing the forces in the direction parallel to the slope, the resulting formulation is

$$E_A + \frac{(N_e + N_A)\cdot \tan\delta + C_a}{FS} = (W_A + W_e)\cdot \sin\beta + F_e$$

where

N_e = effective equipment force normal to the failure plane of the active wedge.

$$N_e = W_E \cdot \cos\beta \tag{13.12}$$

Note that all the other symbols have been previously defined.

The interwedge force acting on the active wedge can now be expressed as

$$E_A = \frac{(FS)[(W_A + W_e)\cdot \sin\beta + F_e]}{FS} - \frac{[(N_A + N_e)\cdot \tan\delta + C_a]}{FS}$$

The passive wedge can be treated in a similar manner. The following formulation of the interwedge force acting on the passive wedge results:

$$E_P = \frac{C + W_P \cdot \tan\phi}{\cos\beta \cdot (FS) - \sin\beta \cdot \tan\phi}$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$ which in this case, using *FS*-values, is

$$a \cdot FS^2 + b \cdot FS + c = 0$$

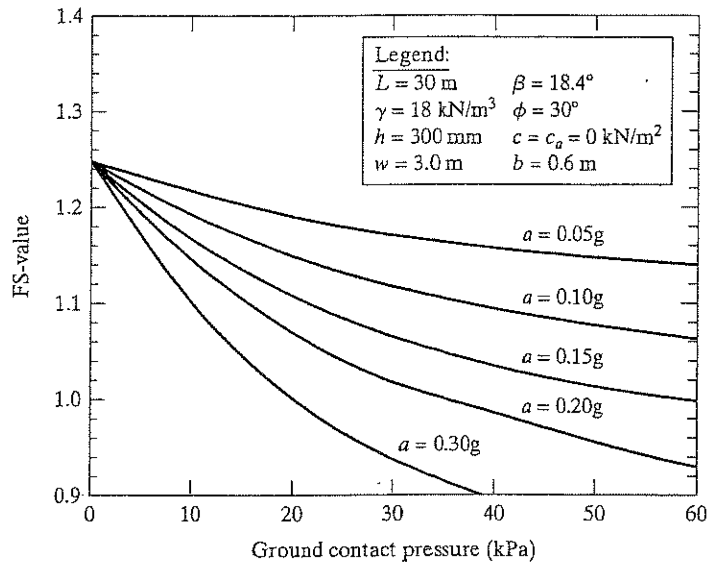


FIGURE 13.10 Design Curves for Stability of Different Construction Equipment Ground Contact Pressure for Various Equipment Accelerations

The resulting *FS*-value is then obtained from the conventional solution of the quadratic equation

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.13}$$

where
$$a = [(W_A + W_e) \cdot \sin\beta + F_e] \cdot \cos\beta$$
$$b = -\{[(N_A + N_e) \cdot \tan\delta + C_a] \cdot \cos\beta + [(W_A + W_e) \cdot \sin\beta + F_e] \cdot \sin\beta \cdot \tan\phi + (C + W_P \cdot \tan\phi)\}$$
$$c = [(N_A + N_e) \cdot \tan\delta + C_a] \cdot \sin\beta \cdot \tan\phi$$

Using these concepts, typical design curves for various *FS*-values as a function of equipment ground contact pressure and equipment acceleration can be developed (see Figure 13.10). Note that the curves are developed specifically for the variables stated in the legend. Example 13.3 illustrates the use of the formulation.

EXAMPLE 13.3

The following are given: a 30-m-long slope with uniform cover soil of 300-mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr, and the time to reach this speed is 3.0 seconds. The cover soil to geomembrane friction angle is 22 degrees with zero adhesion. What is the *FS*-value at a slope angle of 3(H)-to-1(V) (i.e., 18.4°)?

Solution Using the design curves of Figure 13.10 along with Equation 13.13, the solution can be obtained.

- From Figure 13.9, at 20 km/hr and 3.0 seconds, the bulldozer's acceleration is 0.19g.
- From Equation 13.13,

$$a = 88.8 \text{ kN/m}$$
$$b = -107.3 \text{ kN/m}$$
$$c = 17.0 \text{ kN/m}$$

Thus, $FS = 1.03$

This value can be confirmed using Figure 13.10.

Comment This problem solution can now be compared with those of the previous two examples:

Example 13.1.	Cover soil along with no bulldozer loading:	$FS = 1.25$
Example 13.2.	Cover soil plus bulldozer moving up slope:	$FS = 1.24$
Example 13.3.	Cover soil plus bulldozer moving down slope:	$FS = 1.03$

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguably the more severe condition, due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the precise conditions made in the design. The maximum weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations. Truck traffic on the slopes can also give stresses as high or even higher than illustrated here and should be avoided in all circumstances.

13.4.3 Inclusion of Seepage Forces

The previous sections presented the general problem of slope stability analysis of cover soils placed on slopes under different conditions. The tacit assumption throughout was that either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently and safely remove permeating water away from the cross section. The amount of water to be removed is obviously a site-specific situation. Note that, in extremely arid areas, or with very low permeability cover soils, drainage may not be required, although this is generally the exception.

Unfortunately, adequate drainage of final covers has sometimes not been available and seepage-induced slope stability problems have occurred. Figure 13.11 shows a final cover slope failure during a heavy raining. The following situations have resulted in seepage-induced slides:

- Drainage soils with hydraulic conductivity (permeability) too low for site-specific conditions.
- Inadequate drainage capacity at the toe of long slopes, where seepage quantities accumulate and are at their maximum.

FIGURE 13.11 Final Cover Slope Failure during a Heavy Raining

- Fine, cohesionless, cover soil particles migrating through the filter (if one is present) either clogging the drainage layer, or accumulating at the toe of the slope, thereby decreasing the as-constructed outlet permeability over time.
- Freezing of the outlet drainage at the toe of the slope, while the top of the slope thaws, thereby mobilizing seepage forces against the ice wedge at the toe.

If seepage forces of the types described occur, a variation in slope stability design methodology is required. Such an analysis is the focus of this subsection. (See Koerner and Soong, 1998; and Qian, 1997; also, Thiel and Stewart, 1993; and Soong and Koerner, 1996.)

Consider a cover soil of uniform thickness placed directly above a geomembrane at a slope angle of β, as shown in Figure 13.12. What is different from previous examples, however, is that within the cover soil there can exist a saturated soil zone for part or all of the thickness. The saturated boundary is shown as two possibly different phreatic surface orientations. This is because seepage can be built up in the cover soil in two different ways: a horizontal buildup from the toe upward, or a parallel-to-slope buildup outward. These two hypotheses are defined and quantified as a horizontal submergence ratio (HSR) and a parallel submergence ratio (PSR). The dimensional definitions of both ratios are given in Figure 13.12.

When analyzing the stability of slopes using the limit equilibrium method, free-body diagrams of the passive and active wedges are taken with the appropriate forces

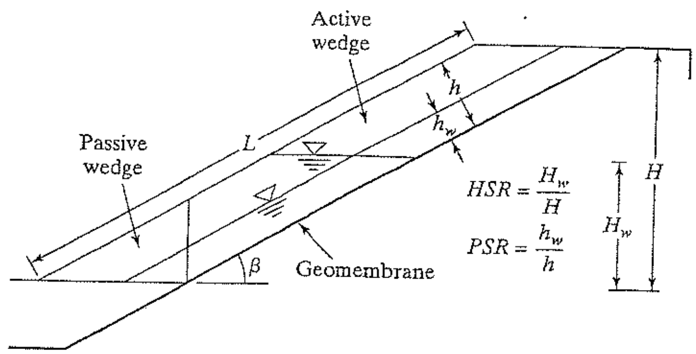


FIGURE 13.12 Cross Section of a Uniform Thickness Cover Soil on a Geomembrane Illustrating Different Submergence Assumptions and Related Definitions (Soong and Koerner, 1996)

being applied (now including pore water pressures). The formulation for the resulting factor of safety for horizontal seepage buildup and also for parallel-to-slope seepage buildup is described next.

13.4.3.1 The Case of the Horizontal Seepage Buildup. Figure 13.13 shows the free-body diagram of both the active and passive wedge assuming horizontal seepage building. Horizontal seepage buildup can occur when toe blockage occurs due to inadequate outlet capacity, contamination or physical blocking of outlets, or freezing conditions at the outlets.

All symbols used in Figure 13.13 were previously defined except the following:

γ_{sat} = saturated unit weight of the cover soil
γ_t = dry unit weight of the cover soil
γ_w = unit weight of water
H = vertical height of the slope measured from the toe
H_w = vertical height of the free water surface measured from the toe
U_h = resultant of the pore pressures acting on the interwedge surfaces
U_n = resultant of the pore pressures acting perpendicular to the slope
U_v = resultant of the vertical pore pressures acting on the passive wedge

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge,

$$W_A = \frac{\gamma_{sat} \cdot h \cdot (2 \cdot H_w \cdot \cos\beta - h)}{\sin 2\beta} + \frac{\gamma_{dry} \cdot h \cdot (H - H_w)}{\sin\beta} \tag{13.14}$$

$$U_n = \frac{\gamma_w \cdot h \cdot \cos\beta \cdot (2 \cdot H_w \cdot \cos\beta - h)}{\sin 2\beta} \tag{13.15}$$

$$U_h = 0.5 \cdot \gamma_w \cdot h^2 \tag{13.16}$$

$$N_A = W_A \cdot \cos\beta + U_h \cdot \sin\beta - U_n \tag{13.17}$$

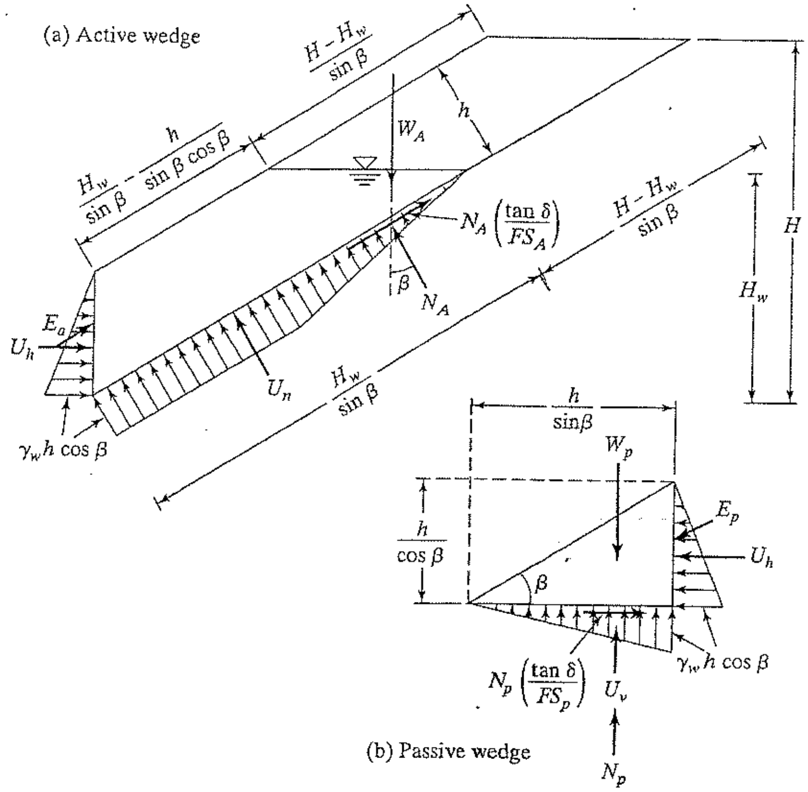


FIGURE 13.13 Limit Equilibrium Forces Involved in a Finite Length Slope of Uniform Cover Soil with Horizontal Seepage Buildup

The interwedge force acting on the active wedge can then be expressed as

$$E_A = W_A \cdot \sin\beta + U_h \cdot \cos\beta - \frac{N_A \cdot \tan\delta}{FS}$$

The passive wedge can be considered in a similar manner and the following expressions result:

$$W_P = \frac{\gamma_{sat} \cdot h^2}{\sin 2\beta} \tag{13.18}$$

$$U_v = U_h \cdot \cot\beta \tag{13.19}$$

The interwedge force acting on the passive wedge can then be expressed as

$$E_P = \frac{U_h \cdot (FS) - (W_P - U_v) \cdot \tan\phi}{\sin\beta \cdot \tan\phi - \cos\beta \cdot (FS)}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2 + bx + c = 0$, which in this case is

$$a \cdot FS^2 + b \cdot FS + c = 0$$

The resulting FS-value is then obtained from the conventional solution of the quadratic equation as

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.20}$$

where
$$a = W_A \cdot \sin\beta \cdot \cos\beta - U_h \cdot \cos^2\beta + U_h$$
$$b = -W_A \cdot \sin^2\beta \cdot \tan\phi + U_h \cdot \sin\beta \cdot \cos\beta \cdot \tan\phi - N_A \cdot \cos\beta \cdot \tan\delta - (W_P - U_v) \cdot \tan\phi$$
$$c = N_A \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$$

13.4.3.2 The Case of Parallel-to-Slope Seepage Buildup. Figure 13.14 shows the free body diagrams of both the active and passive wedges with seepage buildup in the direction parallel to the slope. Parallel seepage buildup can occur when soils placed above a geomembrane are initially too low in their hydraulic conductivity, or become too low due to long-term clogging from overlying soils that are not filtered. The individual forces, friction angles, and slope angles involved in Figure 13.14 are listed as follows:

W_A = weight of the active wedge (area times unit weight), lb/ft or kN/m;
W_P = weight of the passive wedge (area times unit weight), lb/ft or kN/m;
β = angle of the slope, degree;
H = height of the cover soil slope from the toe of the cover soil to the top of the slope (see Figure 13.14), ft or m;
h = thickness of the soil layer (perpendicular to the slope), ft or m;
h_w = depth of seepage water in the soil layer (perpendicular to the slope), ft or m;
γ = moisture unit weight of the soil layer, lb/ft^3 or kN/m^3;
γ_{sat} = saturated unit weight of the soil layer, lb/ft^3 or kN/m^3;
γ_w = unit weight of water, 62.4 lb/ft^3 or 9.81 kN/m^3;
ϕ = friction angle of the cover soil, degree;
δ = interface friction angle between the soil layer and geomembrane, degree;
N_A = normal force acting on bottom of the active wedge, lb/ft or kN/m;
F_A = frictional force acting on bottom of the active wedge, lb/ft;
U_{AN} = resultant of the pore water pressures acting on bottom of the active wedge (perpendicular to the slope), lb/ft or kN/m;
U_{AH} = resultant of the pore water pressures acting on lower lateral side of the active wedge (perpendicular to the interface between the active and passive wedges), lb/ft or kN/m;
E_A = force from passive wedge acting on active wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m;
N_P = normal force acting on the bottom of passive wedge, lb/ft or kN/m;
F_P = frictional force acting on the bottom of passive wedge, lb/ft or kN/m;

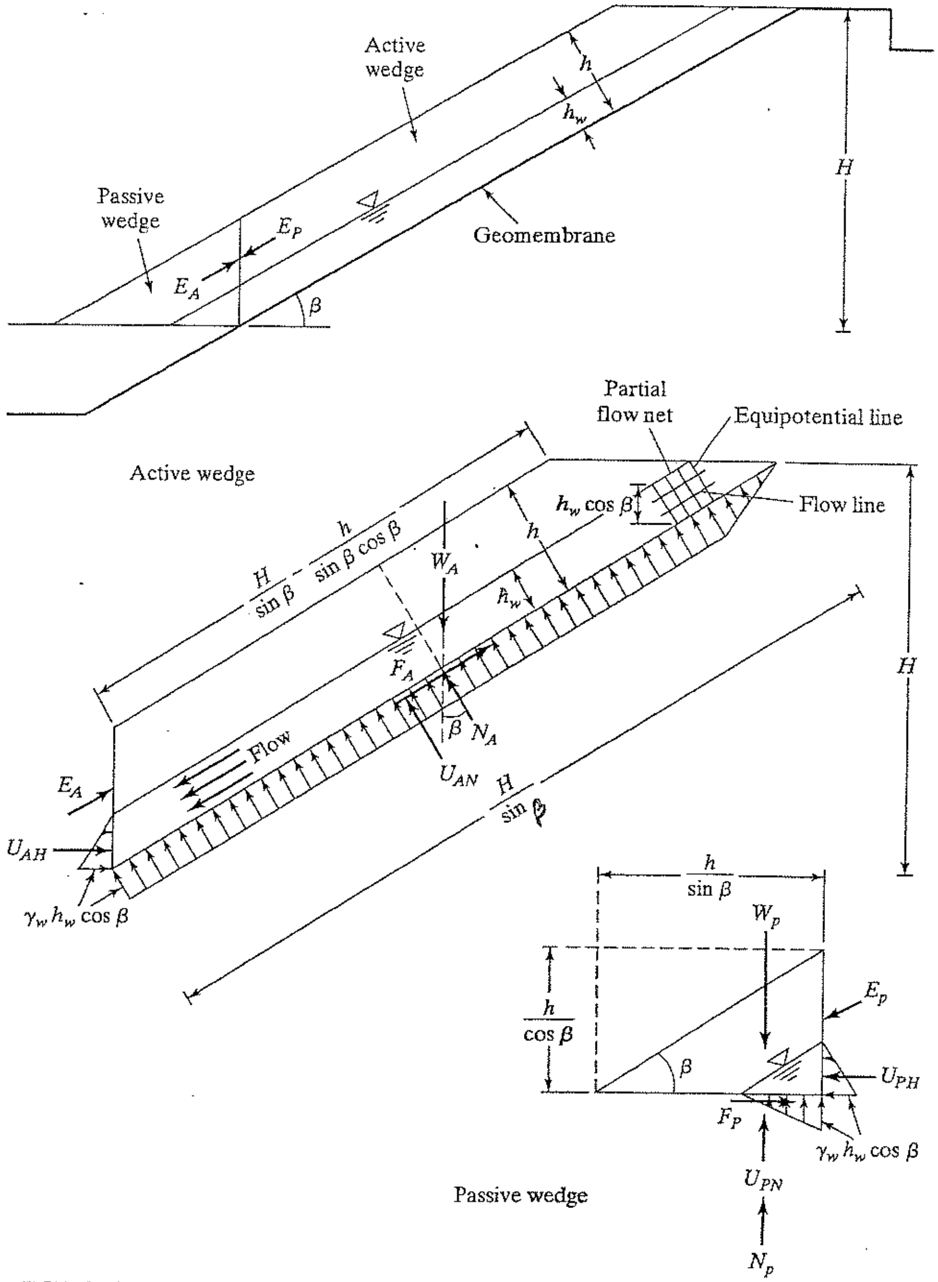


FIGURE 13.14 Cross Section of Sand Layer over Geomembrane on Side Slope with Seepage Parallel to Slope.

U_H = resultant of the pore water pressures acting on lateral side of the active wedge or passive wedge (perpendicular to the lateral side), lb/ft or kN/m, $U_H = U_{AH} = U_{PH}$;

U_{PN} = resultant of the pore water pressures acting on bottom of the passive wedge (perpendicular to bottom of the passive wedge), lb/ft or kN/m;

E_P = force from active wedge acting on passive wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m, $E_A = E_P$;

FS = factor of safety for stability of the cover soil mass.

Considering the force equilibrium of the active wedge (Figure 13.14), we obtain

$\Sigma F_Y = 0$: $$N_A + U_{AN} = W_A \cdot \cos\beta + U_{AH} \cdot \sin\beta$$

$$N_A = W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta \tag{13.21}$$

$\Sigma F_X = 0$: $$F_A + E_A + U_{AH} \cdot \cos\beta = W_A \cdot \sin\beta$$

$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - F_A \tag{13.22}$$

$$F_A = N_A \cdot \tan\delta / FS \tag{13.23}$$

Substituting Equation 13.21 into Equation 13.23 gives

$$F_A = (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS \tag{13.24}$$

Substituting Equation 13.24 into Equation 13.22 gives

$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS \tag{13.25}$$

Considering the force equilibrium of the passive wedge (Figure 13.14) yields

$$E_P = E_A \tag{13.26}$$

$\Sigma F_Y = 0$: $$N_P + U_{PN} = W_P + E_P \cdot \sin\beta \tag{13.27}$$

Substituting Equation 13.26 into Equation 13.27 gives

$$N_P = W_P + E_A \cdot \sin\beta - U_{PN} \tag{13.28}$$

Substituting Equation 13.25 into Equation 13.28 gives

$$N_P = W_P - U_{PN} + [W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS] \cdot \sin\beta$$

$$N_P = W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta / FS \tag{13.29}$$

$\Sigma F_X = 0$: $$F_P = U_{PH} + E_P \cdot \cos\beta \tag{13.30}$$

Substituting Equation 13.26 into Equation 13.30 gives

$$F_P = U_{PH} + E_A \cdot \cos\beta \tag{13.31}$$

Substituting Equation 13.25 into Equation 13.31 gives

$$F_P = U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta - (W_A \cdot \cos\beta - U_{AN}$$

$$FS = \frac{N_P \cdot \tan\phi}{F_P} \tag{13.33}$$

Substituting Equations 13.29 and 13.32 into Equation 13.33 gives

$$FS = \frac{(W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \cdot \tan\phi - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS}{U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \cdot \tan\delta / FS}$$

$$(U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta) \cdot FS - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \cdot \tan\delta = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \cdot \tan\phi - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS$$

$$(W_A \cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \cdot \tan\delta \cdot FS = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \cdot \tan\phi \cdot FS - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$$

$$(W_A \cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_{AH} \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0 \tag{13.34}$$

Because $U_H = U_{PH} = U_{AH}$,

$$[W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)] \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0 \tag{13.35}$$

Using $a \cdot x^2 + b \cdot x + c = 0$

The resulting FS can be expressed as

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$

where

$$a = W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)$$

$$b = -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)]$$

$$c = (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$$

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\, h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$

$$U_H = 0.5 \cdot \gamma_w \cdot {h_w}^2 \tag{13.38}$$

$$U_{PN} = 0.5 \cdot \gamma_w \cdot {h_w}^2/\tan\beta \tag{13.39}$$

$$W_A = 0.5 \cdot [\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) + \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta) \tag{13.40}$$

$$W_P = 0.5 \cdot [\gamma \cdot (h^2 - {h_w}^2) + \gamma_{sat} \cdot {h_w}^2]/(\sin\beta \cdot \cos\beta) \tag{13.41}$$

EXAMPLE 13.4

A 44-ft (13.2-m) high and 3(H):1(V) slope has cover sand with a uniform thickness of 2 ft (0.6 m) at a unit weight of 110 lb/ft^3 (17.3 kN/m^3). The cover sand has a friction angle of 32 degrees and zero cohesion. Seepage occurs parallel to the slope and the seepage water head in the sand layer is 6 inches (0.15 m). The saturated unit weight of sand is 115 lb/ft^3 (18 kN/m^3). The interface friction angle between sand drainage layer and geomembrane is 22 degrees and zero adhesion. What is the factor of safety at a slope of 3(H)-to-1(V)?

Solution The side slope angle is at 18.4° for a 3(H):1(V) slope. Hence,

$$\sin\beta = \sin(18.4°) = 0.316,\ \cos\beta = \cos(18.4°) = 0.949,\ \tan\beta = \tan(18.4°) = 0.333.$$

$$H = 44\ \text{ft}\ (13.2\ \text{m}),\ h = 2\ \text{ft}\ (0.6\ \text{m}),\ h_w = 0.5\ \text{ft}\ (0.15\ \text{m}),\ \gamma = 110\ \text{lb/ft}^3\ (17.3\ \text{kN/m}^3),$$

$$\gamma_{sat} = 115\ \text{lb/ft}^3\ (18\ \text{kN/m}^3),\ \gamma_w = 62.4\ \text{lb/ft}^3\ (9.81\ \text{kN/m}^3),\ \phi = 32°,\ \delta = 22°.$$

$$\tan\phi = \tan(32°) = 0.625,\ \tan\delta = \tan(22°) = 0.404.$$

$$\begin{aligned} U_{AN} &= \gamma_w \cdot h_w \cdot (H - 0.5\,h_w \cdot \cos\beta)/\tan\beta \\ &= (62.4)(0.5)[44 - (0.5)(0.5)(0.949)]/(0.333) = 4{,}100.3\ \text{lb/ft}\ (58.02\ \text{kN/m}) \end{aligned} \tag{13.37}$$

$$\begin{aligned} U_H &= 0.5 \cdot \gamma_w \cdot h_w^2 \\ &= (0.5)(62.4)(0.5)^2 = 7.8\ \text{lb/ft}\ (0.11\ \text{kN/m}) \end{aligned} \tag{13.38}$$

$$\begin{aligned} U_{PN} &= 0.5 \cdot \gamma_w \cdot h_w^2/\tan\beta \\ &= (0.5)(62.4)(0.5)^2/(0.333) = 23.4\ \text{lb/ft}\ (0.33\ \text{kN/m}) \end{aligned} \tag{13.39}$$

$$\begin{aligned} W_A &= 0.5 \cdot [\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) \\ &\quad + \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta) \\ &= (0.5)\{(110)(2 - 0.5)[(2)(44)(0.949) - 2 - 0.5] \\ &\quad + (115)(0.5)[(2)(44)(0.949) - 0.5]\}/[(0.316)(0.949)] \\ &= (0.5)(13{,}366.98 + 4{,}773.19)/[(0.316)(0.949)] = 30{,}245.3\ \text{lb/ft}\ (427.6\ \text{kN/m}) \end{aligned} \tag{13.40}$$

$$\begin{aligned} W_P &= 0.5 \cdot [\gamma \cdot (h^2 - h_w^2) + \gamma_{sat} \cdot h_w^2]/(\sin\beta \cdot \cos\beta) \\ &= (0.5)\{(110)[(2)^2 - (0.5)^2] + (115)(0.5)^2\}/[(0.316)(0.949)] = 735.7\ \text{lb/ft}\ (10.4\ \text{kN/m}) \end{aligned} \tag{13.41}$$

Using Equation 13.36,

$$\begin{aligned} a &= W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta) \\ &= (30{,}245.3)(0.316)(0.949) + (7.8)[1 - (0.949)^2] = 9{,}071\ (128\ \text{for SI units}) \end{aligned}$$

$$\begin{aligned} b &= -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi \\ &\quad + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)] \\ &= -\{(735.7)(0.625) + (30{,}245.3)[(0.316)^2(0.625) + (0.949)^2(0.104)] - (4{,}100.3)(0.949)(0.404) \\ &\quad - (23.4)(0.625) + (7.8)(0.316)(0.949)(0.625 - 0.404)\} \\ &= -(459.8 + 12{,}892.1 - 1{,}572.0 - 14.6 + 0.5) = -11{,}766\ (-166\ \text{for SI units}) \end{aligned}$$

$$\begin{aligned} c &= (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi \\ &= [(30{,}245.3)(0.949) - 4{,}100.3 + (7.8)(0.316)](0.316)(0.625)(0.404) = 1{,}963\ (28\ \text{for SI units}) \end{aligned}$$

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$

FIGURE 13.15 Sand Layer Failure along Sideslope Caused by Seepage Force

$$= \frac{11{,}766 + 8{,}198}{(2)(9{,}071)}$$
$$= 1.10$$

Comment The seriousness of seepage forces in a slope of this type is immediately obvious. Had the saturation been 100% of the drainage layer thickness, the FS-value would have been still lower. Furthermore, the result using a horizontal assumption of saturated cover soil with the same saturation ratio will give essentially identical low *FS*-values. Clearly, the teaching of this example problem is that adequate long-term drainage above the barrier layer in cover soil slopes must be provided to avoid seepage forces from occurring. Figure 13.15 shows a sand layer sliding failure along sideslope caused by seepage force.

An incremental placement method should be implemented for sideslopes higher than the maximum height that can be built in a single lift with a minimum required factor of safety, such as the previous example. Based on the incremental placement method, the first step is to place the sand drainage layer on the sideslope to the maximum unsupported height. As waste is filled against the sideslope to approximately 2 feet (0.6 m) below the protective layer, the next lift of the layer can proceed. This procedure that is illustrated in Figure 13.16 should be continued until the protective layer reaches the top of the sideslope. The heights of the following lifts of the sand drainage layer should not be higher than the calculated maximum unsupported height minus 2

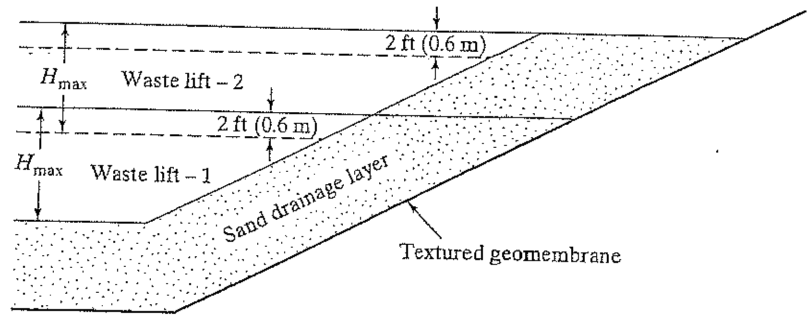


FIGURE 13.16 Incremental Placement of Soil Drainage Layer on Sideslope

feet (0.6 m). The height of the first lift of sand placement can be calculated as shown in the equations that follow (Qian, 1997):

In U.S. units,

$$H = (H_{total} - 2)/n + 2 \tag{13.42}$$

In SI units,

$$H = (H_{total} - 0.6)/n + 0.6 \tag{13.43}$$

where H = height of the first step of sand placement on the sideslope (see Figure 13.14), ft or m;
H_{total} = total height of the cover sand slope from the toe of the cover sand to the top of the slope (see Figure 13.16), ft or m;
n = number of the placement steps.

EXAMPLE 13.5

Continue the calculations of Example 13.4 and use the incremental method to achieve a factor of safety no less than 1.2 for the cover sand resting on the sideslope?

Solution Use the incremental method to place drainage sand on the side slope to achieve a minimum factor of safety of 1.2. Try three steps of sand placement ($n = 3$) on the sideslope.

$$H = (H_{total} - 2)/n + 2 \tag{13.42}$$
$$= (44 - 2)/3 + 2 = 14 + 2 = \mathbf{16}\text{ ft } (4.8\text{ m})$$

So,

$H = 16$ ft (4.8 m), $h = 2$ ft (0.6 m), $h_w = 0.5$ ft (0.15 m), $\gamma = 110$ lb/ft^3 (17.3 kN/m^3), $\gamma_{sat} = 115$ lb/ft^3 (18 kN/m^3), $\gamma_w = 62.4$ lb/ft^3 (9.81 kN/m^3), $\phi = 32°$, $\delta = 22°$.

$\tan\phi = \tan(32°) = 0.625$, $\tan\delta = \tan(22°) = 0.404$,

$\sin\beta = \sin(18.4°) = 0.316$

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\ h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$
$$= (62.4)(0.5)[16 - (0.5)(0.5)(0.949)]/(0.333) = 1{,}476.9 \text{ lb/ft } (20.90 \text{ kN/m})$$
$$U_H = 0.5 \cdot \gamma_w \cdot h_w^{\ 2} \tag{13.38}$$
$$= (0.5)(62.4)(0.5)^2 = 7.8 \text{ lb/ft } (0.11 \text{ kN/m})$$
$$U_{PN} = 0.5 \cdot \gamma_w \cdot h_w^{\ 2}/\tan\beta \tag{13.39}$$
$$= (0.5)(62.4)(0.5)^2/(0.333) = 23.4 \text{ lb/ft } (0.33 \text{ kN/m})$$
$$W_A = 0.5 \cdot [\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) + \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta) \tag{13.40}$$
$$= (0.5)\{(110)(2 - 0.5)[(2)(16)(0.949) - 2 - 0.5] + (115)(0.5)[(2)(16)(0.949) - 0.5]\}/[(0.316)(0.949)]$$
$$= (0.5)(4{,}598.22 + 1{,}717.41)/[(0.316)(0.949)] = 10{,}530.1 \text{ lb/ft } (148.9 \text{ kN/m})$$
$$W_P = 0.5 \cdot [\gamma \cdot (h^2 - h_w^{\ 2}) + \gamma_{sat} \cdot h_w^2]/(\sin\beta \cdot \cos\beta) \tag{13.41}$$
$$= (0.5)\{(110)[(2)^2 - (0.5)^2] + (115)(0.5)^2\}/[(0.316)(0.949)] = 735.7 \text{ lb/ft } (10.4 \text{ kN/m})$$

Equation 13.36 yields

$$a = W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)$$
$$= (10{,}530.1)(0.316)(0.949) + (7.8)[1 - (0.949)^2] = 3{,}159 \text{ (45 for SI units)}$$
$$b = -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)]$$
$$= -\{(735.7)(0.625) + (10{,}530.1)[(0.316)^2(0.625) + (0.949)^2(0.404)] - (1{,}476.9)(0.949)(0.404) - (23.4)(0.625) + (7.8)(0.316)(0.949)(0.625 - 0.404)\}$$
$$= -(459.8 + 4{,}488.5 - 566.2 - 14.6 + 0.5) = -4{,}368 \text{ (}-62 \text{ for SI units)}$$
$$c = (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$$
$$= [(10{,}530.1)(0.949) - 1{,}476.9 + (7.8)(0.316)](0.316)(0.625)(0.404) = 680 \text{ (10 for SI units)}$$
$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$
$$= \frac{4{,}368 + [(-4{,}368)^2 - (4)(3{,}159)(680)]^{0.5}}{(2)(3{,}159)}$$
$$= \frac{4{,}368 + 3{,}238}{(2)(3{,}159)}$$
$$= 1.20$$

Thus, based on the above calculation, the first step is to place the drainage sand on the sideslope to a height of 16 feet (4.8 m). As waste is filled against the sideslope to approximately 2 feet (0.6 m) below the protective layer, the next lift of 14 feet (4.2 m) can be placed. This procedure should be continued until the protective layer reaches the top of the sideslope.

13.4.4 Inclusion of Seismic Forces

In areas of anticipated earthquake activity, the slope stability analysis of a final cover soil over an engineered landfill, abandoned dump, or remediated site must consider seismic forces. In the United States, the Environmental Protection Agency (EPA)

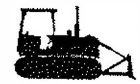
ATTACHMENT B

DOZER SPECIFICATIONS FROM MANUFACTURER

D5C LGP

1 kW	90 HP
00 kg	19,800 lb
—	
3204	
2400	
4	
4 mm	4.5″
.7 mm	5″
.2 L	318 in³
6	
0 mm	26″
14 m	7′0.4″
83 m²	4389 in²
72 m	5′8″
.75 m	5′9.2″
[illegible]	8′11″
[illegible]	13′4″
.95	9′9.8″
—	
.38 m	7′10″
.4 mm	14.2″
—	
—	
.26 m	10′8″
.95 m	9′8″
67 L	44 U.S. gal

tor. D3C LGP Series II

MODEL	D4H LGP Series III		D5H LGP Series II		D6D LGP		D6H LGP Series II	
Flywheel Power	86 kW	116 HP	97 kW	130 HP	104 kW	140 HP	127 kW	170 HP
Operating Weight*								
(Power Shift)	12 196 kg	26,830 lb	15 337 kg	33,818 lb	17 373 kg	38,300 lb	19 814 kg	43,590 lb
(Direct Drive)	12 356 kg	27,180 lb	15 419 kg	33,999 lb	—		19 989 kg	43,976 lb
(Power Shift Differential Steer)	—		—		—		20 060 kg	44,131 lb
Engine Model	3304		3304		3306		3306	
Rated Engine RPM	2200		2200		1900		1900	
No. of Cylinders	4		4		6		6	
Bore	121 mm	4.75″	121 mm	4.75″	121 mm	4.75″	121 mm	4.75″
Stroke	152 mm	6″	152 mm	6″	152 mm	6″	152 mm	6″
Displacement	7 L	425 in³	7 L	425 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		7		8	
Width of Standard Track Shoe	760 mm	30″	860 mm	34″	910 mm	36″	915 mm	36″
Length of Track on Ground	2.62 m	8′7″	3.12 m	10′3″	2.87 m	9′5″	3.27 m	10′8.5″
Ground Contact Area (W/Std. Shoe)	3.98 m²	6170 in²	5.37 m²	8320 in²	5.25 m²	8136 in²	5.97 m²	9254 in²
Track Gauge	2.00 m	6′6″	2.16 m	7′1″	2.11 m	6′9″	2.23 m	7′3″
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.20 m	7′3″	2.30 m	7′6.5″	2.05 m	6′8″	2.32 m	7′7″
Height (To Top of ROPS Canopy)	3.63 m	9′11.4″	3.12 m	10′3″	2.92 m	9′7.5″	3.16 m	10′5″
Height (To Top of Cab ROPS)	—		3.18 m	10′5″	—		3.16 m	10′5″
Overall Length (With P Blade)	4.77 m	15′8″	5.30 m	17′6.3″	—		5.18 m	17′0″
(Without Blade)	—		4.13 m	13′7″	—		4.49 m	14′9″
Overall Length (With S Blade)	—		—		5.16 m	16′11″	—	
(Without Blade)	—		—		3.94 m	12′11″	—	
Width (Over Trunnion)	—		3.26 m	10′8.4″	—		3.43 m	11′3″
Width (W/O Trunnion — Std. Shoe)	2.76 m	9′1″	3.02 m	9′11″	—		3.14 m	10′3.6″
Width (With Standard Shoe)	—		—		3.02 m	9′11″	—	
Ground Clearance	363 mm	14.3″	529 mm	20.8″	310 mm	12.2″	382 mm	15″
Blade Types and Widths:								
Straight	3.26 m	10′8.2″	3.65 m	12′0″	3.71 m	12′2″	3.99 m	13′1″
Angle	—		—		—		—	
Power Angle & Tilt	—		3.98 m	13′0.1″	—		—	
"P" Straight	3.26 m	10′8.2″	—		—		—	
Angled	3.00 m	9′10.1″	3.66 m	11′11.9″	—		—	
Fuel Tank Refill Capacity	200 L	52 U.S. gal	246 L	65 U.S. gal	295 L	78 U.S. gal	337 L	89 U.S. gal

*Operating Weight includes lubricants, coolant, full fuel tank, straight bulldozer, hydraulic controls and fluid, ROPS canopy and operator and rigid drawbar. D5H Series II with P-blade.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.
Note: D4H LGP Series III has P-blade.

APPENDIX H

Anchor Trench Calculation Package

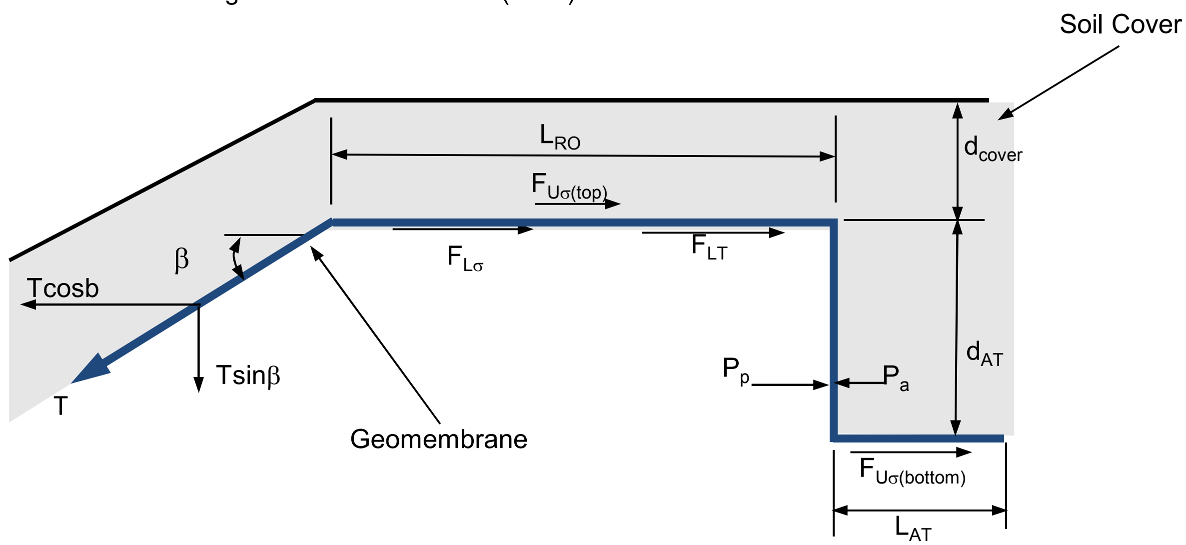
Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 4:1 Slope	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

OBJECTIVE

Determine the runout length, trench width, and trench depth required to prevent wind and water from moving under the geosynthetic of the final cover system.

METHOD

The anchor trench design is based on Koerner (1998) and is summarized below:



$$T_{allow} \cos\beta = F_{U\sigma(top)} + F_{L\sigma} + F_{LT} - P_a + P_p + F_{U\sigma(bottom)}$$

where : T_{allow} = allowable force in geomembrane = $\sigma_{allow}t$

σ_{allow} = allowable stress in geomembrane

t = thickness of geomembrane

β = side slope angle

$F_{U\sigma(top)}$ = shear force above geomembrane due to cover soil (note that for thin cover soils, tensile cracking will occur, and this value will be negligible)

$$F_{U\sigma(top)} = \sigma_n \tan\delta_U (L_{RO})$$

$F_{L\sigma}$ = shear force below geomembrane due to cover soil

$$F_{L\sigma} = \sigma_n \tan\delta_L (L_{RO})$$

F_{LT} = shear force below geomembrane due to vertical component of T_{allow}

$$F_{LT} = 0.5\left(\frac{2T_{allow}\sin\beta}{L_{RO}}\right)(L_{RO})\tan\delta_L$$

$F_{U\sigma(bottom)}$ = shear force above geomembrane in trench due to cover soil

$$F_{U\sigma(bottom)} = [\gamma_{AT}(d_{AT} + d_{cover})]\tan\delta_L L_{AT}$$

L_{RO} = length of geomembrane runout

P_a = active earth pressure against the backfill side of the anchor trench

$$P_a = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_a d_{AT}$$

P_p = passive earth pressure against the in-situ side of the anchor trench

$$P_p = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_p d_{AT}$$

γ_{AT} = unit weight of soil in anchor trench
d_{AT} = depth of the anchor trench
σ_n = applied normal stress from cover soil

$$\sigma_n = \gamma_{AT} d_{cover}$$

K_a = coefficient of active earth pressure

$$K_a = \tan^2\left(45 - \phi/2\right)$$

K_p = coefficient of passive earth pressure

$$K_p = \tan^2\left(45 + \phi/2\right)$$

ϕ = angle of shearing resistance of respective soil
δ = angle of shearing resistance between geomembrane and adjacent material (i.e. soil or geotextile)

ASSUMPTIONS

The 50-mil LLDPE Super Gripnet® geomembrane will be used as the final cover liner.

Geomembrane:

T_{allow} =	9.2	kN/m	or	52.5	lb/in	for 50-mil LLDPE Geomembrane Tensile Strength at Break = 105 lb/in, for FS = 2, T_{allow} = 52.5 lb/in
t =	1.3	mm	or	50	mil	for 40-mil LLDPE Geomembrane Tensile Strength at Break = 112 lb/in, for FS = 2, Tallow = 56 lb/in

Soil cover:

d_{cover} =	0	m	or	0	ft	No Cover soil for Closure Turf System
γ_{AT} =	17.28	kN/m^3	or	110	lb/ft^3	

Slope angle:

β =	14.0	deg (4H:1V)	Slopes range from 2.5H:1V to 4H:1V Shallower slope controls the design

GOLDER

Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 4:1 Slope	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

Friction angle of soil and interface between soil and geomembrane:

δ_U = 0 deg (friction angle between geomembrane and soil above geomembrane, set to zero assuming soil cracking occurs)

δ_L = 25 deg (conservative interface friction angle between geomembrane and materials below geomembrane, based on Technical Specification)

ϕ = 25 deg (conservative friction angle of soil)

Length of runout and length of anchor trench:

L_{RO} = 0.91 m or 3 ft — As no material is above the liner, L_{RO} does not factor in design, but set to typical minimum of 3 ft.

d_{AT} = 0.61 m or 2 ft — Anchor trench depth of 2 ft set

CALCULATIONS

Determine the depth of the anchor trench (d_{AT}) such that:

$$T_{allow}\cos\beta = F_{U\sigma(top)} + F_{L\sigma} + F_{LT} - P_a + P_p + F_{U\sigma(bottom)}$$

T_{allow} = 9.2 kN/m

σ_n = 0 kPa — No Cover soil for Closure Turf System

$F_{U\sigma}$ = 0 kN/m

$F_{L\sigma}$ = 0.0 kN/m

F_{LT} = 1.0 kN/m

$F_{U\sigma(bottom)}$ = 4.9 L_{AT}

K_a = 0.406 kN/m

P_a = 1.303 kN/m

K_p = 2.464 kN/m

P_p = 7.911 kN/m

$T_{allow}\cos\beta$ = 8.9 kN/m

$8.9 = 0.0 + 0.0 + 1.0 - 1.303 + 7.911 + 4.9\ L_{AT}$

$1.3 = 4.9\ L_{AT}$

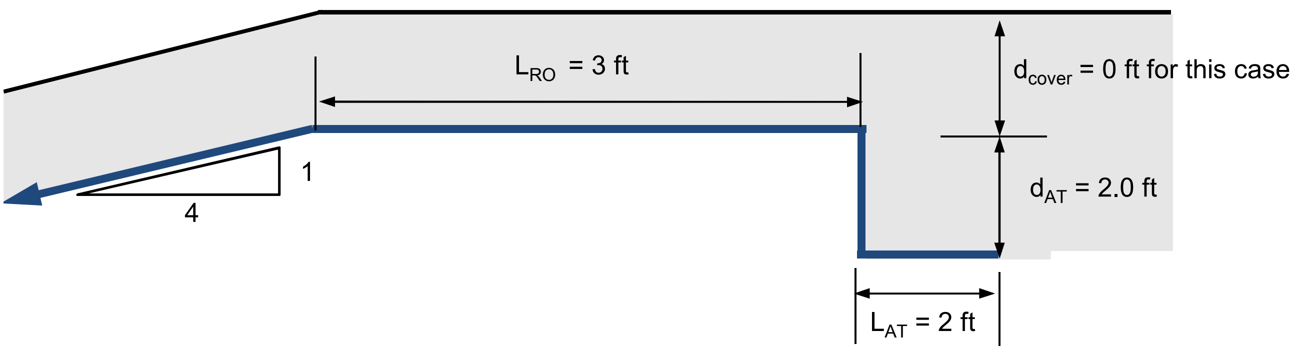
Solve for the minimum width / length of the anchor trench:

Min Calculated L_{AT} = 0.3 m 0.8 ft — Minimum anchor trench width must be greater than calculated minimum

Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 4:1 Slope	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

SUMMARY

Anchor trenchs with a length greater than the calculated minimum of 0.8 ft and a depth of 2 ft are calculated to be adequate. Therefore, the proposed depth and width of the anchor trench 2 ft x 2 ft meet the slope geometry requirements



REFERENCES

Koerner, R.M. (1998) *Designing with Geosynthetics* , 4th ed., Prentice Hall, New Jersey.

GOLDER

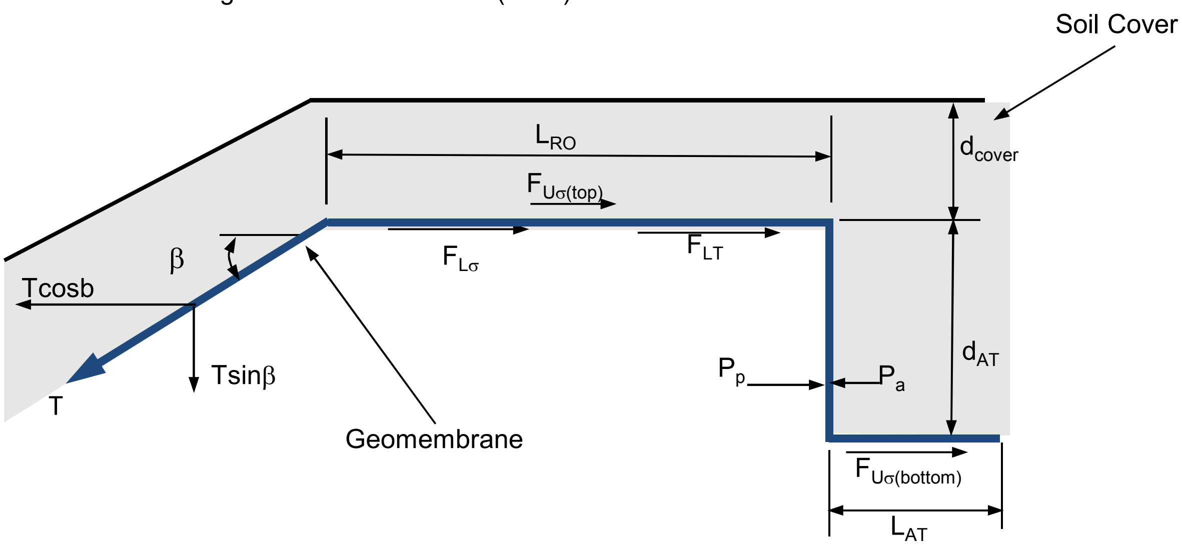
Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 2.5:1 Case	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

OBJECTIVE

Determine the runout length, trench width, and trench depth required to prevent wind and water from moving under the geosynthetic of the final cover system.

METHOD

The anchor trench design is based on Koerner (1998) and is summarized below:



$$T_{allow}\cos\beta = F_{U\sigma(top)} + F_{L\sigma} + F_{LT} - P_a + P_p + F_{U\sigma(bottom)}$$

where : T_{allow} = allowable force in geomembrane = $\sigma_{allow}t$

σ_{allow} = allowable stress in geomembrane

t = thickness of geomembrane

β = side slope angle

$F_{U\sigma(top)}$ = shear force above geomembrane due to cover soil (note that for thin cover soils, tensile cracking will occur, and this value will be negligible)

$$F_{U\sigma(top)} = \sigma_n \tan\delta_U (L_{RO})$$

$F_{L\sigma}$ = shear force below geomembrane due to cover soil

$$F_{L\sigma} = \sigma_n \tan\delta_L (L_{RO})$$

F_{LT} = shear force below geomembrane due to vertical component of T_{allow}

$$F_{LT} = 0.5\left(\frac{2T_{allow}\sin\beta}{L_{RO}}\right)(L_{RO})\tan\delta_L$$

$F_{U\sigma(bottom)}$ = shear force above geomembrane in trench due to cover soil

$$F_{U\sigma(bottom)} = [\gamma_{AT}(d_{AT} + d_{cover})]\tan\delta_L L_{AT}$$

L_{RO} = length of geomembrane runout

P_a = active earth pressure against the backfill side of the anchor trench

$$P_a = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_a d_{AT}$$

P_p = passive earth pressure against the in-situ side of the anchor trench

$$P_p = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_p d_{AT}$$

γ_{AT} = unit weight of soil in anchor trench
d_{AT} = depth of the anchor trench
σ_n = applied normal stress from cover soil

$$\sigma_n = \gamma_{AT} d_{cover}$$

K_a = coefficient of active earth pressure

$$K_a = \tan^2\left(45 - \phi/2\right)$$

K_p = coefficient of passive earth pressure

$$K_p = \tan^2\left(45 + \phi/2\right)$$

ϕ = angle of shearing resistance of respective soil
δ = angle of shearing resistance between geomembrane and adjacent material (i.e. soil or geotextile)

ASSUMPTIONS

The 50-mil LLDPE Super Gripnet® geomembrane will be used as the final cover liner.

Geomembrane:

T_{allow} =	9.2	kN/m	or	52.5	lb/in	for 50-mil LLDPE Geomembrane Tensile Strength at Break = 105 lb/in, for FS = 2, T_{allow} = 52.5 lb/in
t =	1.3	mm	or	50	mil	for 40-mil LLDPE Geomembrane Tensile Strength at Break = 112 lb/in, for FS = 2, Tallow = 56 lb/in

Soil cover:

d_{cover} =	0	m	or	0	ft	No Cover soil for Closure Turf System
γ_{AT} =	17.28	kN/m^3	or	110	lb/ft^3	

Slope angle:

β =	21.8	deg (4H:1V)	Slopes range from 2.5H:1V to 4H:1V Shallower slope controls the design

Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 2.5:1 Case	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

Friction angle of soil and interface between soil and geomembrane:

δ_U = 0 deg (friction angle between geomembrane and soil above geomembrane, set to zero assuming soil cracking occurs)

δ_L = 25 deg (conservative interface friction angle between geomembrane and materials below geomembrane, based on Technical Specification)

ϕ = 25 deg (conservative friction angle of soil)

Length of runout and length of anchor trench:

L_{RO} = 0.91 m or 3 ft — As no material is above the liner, L_{RO} does not factor in design, but set to typical minimum of 3 ft.

d_{AT} = 0.61 m or 2 ft — Anchor trench depth of 2 ft set

CALCULATIONS

Determine the depth of the anchor trench (d_{AT}) such that:

$$T_{allow}\cos\beta = F_{U\sigma(top)} + F_{L\sigma} + F_{LT} - P_a + P_p + F_{U\sigma(bottom)}$$

T_{allow} =	9.2	kN/m	
σ_n =	0	kPa	No Cover soil for Closure Turf System
$F_{U\sigma}$ =	0	kN/m	
$F_{L\sigma}$ =	0.0	kN/m	
F_{LT} =	1.6	kN/m	
$F_{U\sigma(bottom)}$ =	4.9	L_{AT}	
K_a =	0.406	kN/m	
P_a =	1.303	kN/m	
K_p =	2.464	kN/m	
P_p =	7.911	kN/m	

$T_{allow}\cos\beta$ = 8.5 kN/m

$8.5 = 0.0 + 0.0 + 1.6 - 1.303 + 7.911 + 4.9\ L_{AT}$

$0.3 = 4.9\ L_{AT}$

Solve for the minimum width / length of the anchor trench:

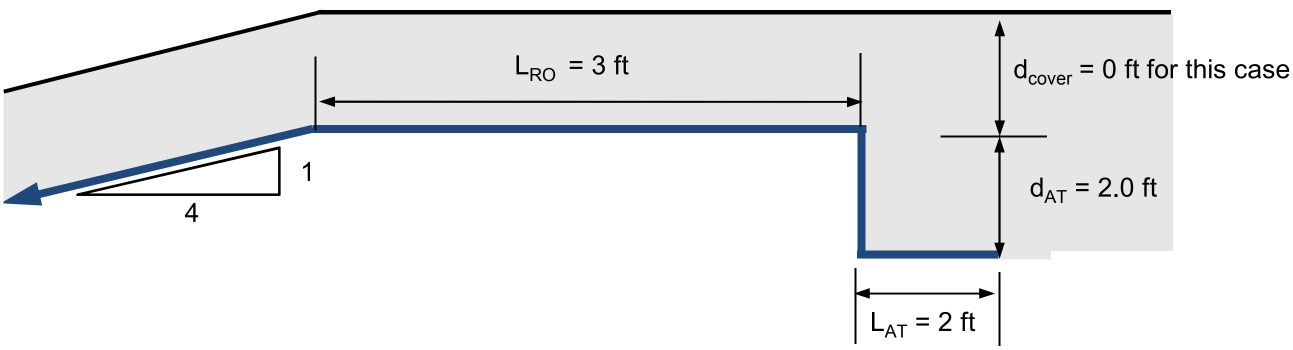
Min Calculated L_{AT} = 0.1 m 0.2 ft — Minimum anchor trench width must be greater than calculated minimum

GOLDER

Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 2.5:1 Case	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

SUMMARY

Anchor trenchs with a length greater than the calculated minimum of 0.2 ft and a depth of 2 ft are calculated to be adequate. Therefore, the proposed depth and width of the anchor trench 2 ft x 2 ft meet the slope geometry requirements



REFERENCES

Koerner, R.M. (1998) *Designing with Geosynthetics* , 4th ed., Prentice Hall, New Jersey.

GEOSYNTHETICS

agru
The Plastics Experts.

Super Gripnet® Liner

LOW DENSITY POLYETHYLENE

AGRU America's structured geomembranes are manufactured on state-of-the-art manufacturing equipment using the flat die calender manufacturing process, a method that produces a more consistent core thickness than other processes, such as the blown film extrusion process. AGRU uses only the highest-grade HDPE and LLDPE resins manufactured in North America.

PRODUCT DATA

Property	Test Method	Frequency	Minimum Average Values			
Thickness (nominal), mil (mm)	ASTM D5994	Per Roll	50 (1.25)	60 (1.5)	80 (2.0	100 (2.5)
Thickness (min avg), mil (mm)			47.5 (1.19)	57 (1.43)	76 (1.9)	95 (2.38)
Thickness (min 8 of 10), mil (mm)			45 (1.12)	54 (1.35)	72 (1.8)	90 (2.25)
Thickness (lowest individual), mil (mm)			42.5 (1.06)	51 (1.28)	68 (1.7)	85 (2.13)
Drainage Stud Height, mil (mm)	ASTM D7466	2nd Roll	130 (3.3)	130 (3.3)	130 (3.3)	130 (3.3)
Friction Spike Height, mil (mm)	ASTM D7466	2nd Roll	175 (4.45)	175 (4.45)	175 (4.45)	175 (4.45)
Density, g/cc, maximum	ASTM D792, Method B	200,000 lb	0.939	0.939	0.939	0.939
Tensile Properties (both directions)	ASTM D6693, Type IV 2 in/minute	20,000 lb				
Strength @ Break, lb/in width (N/mm)			105 (18.4)	126 (22.1)	168 (29.4)	210 (36.8)
Elongation @ Break, % (GL=2.0in)			300	300	300	300
Tear Resistance, lb,s. (N)	ASTM D1004	45,000 lb	30 (133)	40 (178)	53 (236)	64 (285)
Puncture Resistance, lbs. (N)	ASTM D4833	45,000 lb	55 (245)	70 (311)	90 (400)	110 (489)
Carbon Black Content, % (range)	ASTM D4218	20,000 lb	2-3	2-3	2-3	2-3
Carbon Black Dispersion (Category)	ASTM D5596	45,000 lb	Only near spherical agglomerates: 10 views Cat. 1 or 2			
Oxidative Induction Time, minutes	ASTM D3895, 200°C, 1 atm O_2	200,000 lb	≥140	≥140	≥140	≥140

AGRU America's geomembranes are certified to pass Low Temp. Brittleness via ASTM D746 (-80°C), Dimensional Stability via ASTM D1204 (±2% @ 100°C). Oven Aging and UV Resistance are tested per GRI GM 17. These product specifications meet or exceed GRI GM 17.

SUPPLY INFORMATION (STANDARD ROLL DIMENSIONS)

THICKNESS		WIDTH		LENGTH		AREA (APPROX.)	
mil	mm	ft	m	ft	m	ft^2	m^2
50	1.25	23	7	500	152	11,500	1,068
60	1.5	23	7	500	152	11,500	1,068
80	2.0	23	7	300	91.4	6,900	640
100	2.5	23	7	300	91.4	6,900	640

Note:

Average roll weight is 5,000 lbs (2,268 kg) for 50 and 60 mil and 4,000 lbs (1,814 kg) for other thicknesses. All rolls are supplied with two slings. Rolls are wound on a 6" core. Special length available upon request. Roll length and width have a tolerance of ±1%. The weight values may change due to project specifications (i.e. absolute minimum thickness or special length) or shipping requirements (i.e. international contanerized shipments).

All information, recommendations and suggestions appearing in this literature concerning the use of our products are based upon tests and data believed to be reliable; however, it is the users responsibility to determine the suitability for their own use of the products described herein. Since the actual use by others is beyond our control, no guarantee or warranty of any kind, expressed or implied, is made by AGRU America as to the effects of such use or the results to be obtained, nor does AGRU America assume any liability in connection herewith. Any statement made herein may not be absolutely complete since additional information may be necessary or desirable when particular or exceptional conditions or circumstances exist or because of applicable laws or government regulations. Nothing herein is to be construed as permission or as a recommendation to infringe any patent.

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The Plastics Experts.

MicroSpike® Liner

LINEAR LOW DENSITY POLYETHYLENE

AGRU America's structured geomembranes are manufactured on state-of-the-art manufacturing equipment using the flat die calender manufacturing process, a method that produces a more consistent core thickness than other processes, such as the blown film extrusion process. AGRU uses only the highest-grade HDPE and LLDPE resins manufactured in North America.

PRODUCT DATA

Property	Test Method	Frequency	Minimum Average Values			
Thickness (nominal), mil (mm)	ASTM D5994	Per Roll	40 (1.0)	60 (1.5)	80 (2.0)	100 (2.5)
Thickness (min avg), mil (mm)			38 (0.95)	57 (1.43)	76 (1.9)	95 (2.38)
Thickness (min 8 of 10), mil (mm)			36 (0.90)	54 (1.35)	72 (1.8)	90 (2.25)
Thickness (lowest individual), mil (mm)			34 (0.85)	51 (1.28)	68 (1.7)	85 (2.13)
Asperity Height mils, (mm)	ASTM D7466	2nd Roll	20 (0.51)	20 (0.51)	18 (0.46)	18 (0.46)
Density, g/cc, maximum	ASTM D792, Method B	200,000 lb	0.939	0.939	0.939	0.939
Tensile Properties (both directions)	ASTM D6693, Type IV					
Strength @ Break, lb/in width (N/mm)	2 in/minute	20,000 lb	112 (19.6)	168 (29.4)	224 (39.2)	280 (49)
Elongation @ Break, % (GL=2.0in)			400	400	400	400
Tear Resistance, lb,s. (N)	ASTM D1004	45,000 lb	25 (111)	36 (160)	50 (222)	60 (267)
Puncture Resistance, lbs. (N)	ASTM D4833	45,000 lb	50 (222)	70 (310)	90 (400)	115 (512)
Carbon Black Content, % (range)	ASTM D4218	20,000 lb	2-3	2-3	2-3	2-3
Carbon Black Dispersion (Category)	ASTM D5596	45,000 lb	Only near spherical agglomerates: 10 views Cat.1 or 2			
Oxidative Induction Time, minutes	ASTM D3895, 200°C, 1 atm O_2	200,000 lb	≥140	≥140	≥140	≥140

AGRU America's geomembranes are certified to pass Low Temp. Brittleness via ASTM D746 (-80°C), Dimensional Stability via ASTM D1204 (±2% @ 100°C). Oven Aging and UV Resistance are tested per GRI GM 17. These product specifications meet or exceed GRI's GM17.

SUPPLY INFORMATION (STANDARD ROLL DIMENSIONS)

THICKNESS		WIDTH			LENGTH		AREA (APPROX.)	
mil	mm	ft	m		ft	m	ft²	m²
40	1.0	23	7	Double-Sided	750	229	17,250	1,603
				Single-Sided	800	244	18,400	1,709
60	1.5	23	7	Double-Sided	540	165	12,420	1,154
				Single-Sided	560	171	12,880	1,197
80	2.0	23	7	Double-Sided	410	125	9,430	876
				Single-Sided	425	130	9,775	908
100	2.5	23	7	Double-Sided	335	102	7,705	716
				Single-Sided	340	104	7,820	726

Note:

Average roll weight is 3,900 lbs (1,770 kg). All rolls are supplied with two slings. Rolls are wound on a 6" core. Special length available upon request. Roll length and width have a tolerance of ±1%. The weight values may change due to project specifications (i.e. average or absolute minimum thickness) or shipping requirements (i.e. international contanerized shipments).

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Revision Date: March 21, 2018

APPENDIX I

Hydrology and Hydraulic Design for AP-1

Subject:	**CALCULATIONS A: H&H ANALYSIS FOR THE NORTH POND**		
Date:	August 12, 2021	**Made By:**	EC
Project No.:	19124362	**Checked By:**	JDG
Project Short Title:	AP-1 Final Closure Design 2021	**Reviewed By:**	GLH

OBJECTIVE

This calculations sheet covers the hydrologic and hydraulic analysis for the proposed conditions of the AP-1 North Stormwater Attenuation Pond at Plant McDonough-Atkinson (Plant McDonough) located in Cobb County, Georgia.

METHOD

For this analysis, we used The National Engineering Handbook (NEH) part 630 (Hydrology) and the NRCS Technical Release No. 55 (Urban Hydrology for Small Watersheds).

CALCULATIONS

1) RAINFALL

The design storm precipitations are from the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server (PFDS). The table below shows the precipitation events considered for this design.

Agency	Event	Depth (in)	Distribution	ARC	Source
FEMA	100-year, 24 hour	7.54	Type II	II	NOAA Atlas 14
FEMA	25-year, 24 hour	6.02	Type II	II	NOAA Atlas 14
FEMA	2-year, 24 hour	3.75	Type II	II	NOAA Atlas 14

2) WATERSHED

The watershed draining to the AP-1 North Pond was delineated using the topography and contours elevations provided by Georgia Land Department and Metro Engineering and Surveying Co, Inc. The watershed was divided into three basins. Two of the watersheds have a culvert structure that conveys runoff to the North Pond (refer to attached basins maps). The tables below present a summary of the hydrologic properties of each subbasin.

Table 1-A. Basin #1

Area and delineation	10.85	acres
Time of Concentration	22.87	minutes
CN Computation	78	

Table 1-B. Basin #2

Area and delineation	7.44	acres
Time of Concentration	12.17	minutes
CN Computation	89	

Table 1-C. Basin #3

Area and delineation	10.75	acres
Time of Concentration	21.58	minutes
CN Computation	95	

2.1) Curve Number

The curve number (ARC II) was chosen based on the recommended curve numbers given in Table 9-1, 9-2, and 9-5 of the National Engineering Handbook (NEH) for each land use. The land use areas were delineated using ArcMap. The soil type on the watershed is considered type B (Moderately Low Runoff Potential) and D (high runoff potential). The land cover present in the watershed is listed below.

Impervious (roads/buildings)
Fair Grass Cover
Forest
Closure Turf

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Table 2.A- Curve Number Sub-basin #1

Soil Group	Land Use	CN	Area	CNxArea
B	Impervious (roads/buildings)	98	3.3	323.4
	Fair Grass Cover	75	0.62	46.5
	Forest	60	5.08	304.8
D	Fair Grass Cover	83	0.74	61.42
	Gravel	95	1.11	105.45
TOTALS			10.85	841.57
			CN	**78**

Table 2.B- Curve Number Sub-basin #2

Soil Group	Land Use	CN	Area	CNxArea
B	Impervious (roads/buildings)	98	0.35	34.3
	Gravel	95	2.1	199.5
	Fair Grass Cover	75	1.73	129.75
	Forest	60	0.4	24
D	Closure Turf	95	2.86	271.7
TOTALS			7.44	659.25
			CN	**89**

Table 2.C- Curve Number Sub-basin #3

Soil Group	Land Use	CN	Area	CNxArea
D	Closure Turf	95	10.75	1021.25
TOTALS			10.75	1021.25
			CN	**95**

2.2) Time of Concentration

The time of concentration is calculated within Autodesk Storm and Sanitary Analysis 2019 (SSA) using the Velocity Method described in the NRCS technical release No. 55. The flow length and slope were estimated using the ArcMap Spatial Analysis tools and AutoCAD Civil 3D Watershed Analysis tools. Sheet, Concentrated, and Channel flows properties were determined using imagery information and the provided topography. Tables 3A - 3C exhibit the flow properties considered and the time of concentration results.

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Table 3-A. Time of Concentration Sub-basin #1

Flow Type	Flow Properties	Segment 1	Segment 2	Segment 3
Sheet Flow	Flow Length (ft)	100	-	-
	Flow Slope (%)	3.00	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.4 (Avg. Grass Cover)	-	-
	Computed Flow Time (min)	16.91	-	-
Shallow Concentr-ated Flow	Flow Length (ft)	98	411	-
	Flow Slope (%)	8.0	6.0	-
	Surface Type	Grassed waterway	Grassed waterway	-
	Flow Velocity (ft/sec)	4.24	3.67	-
	Computed Flow Time (min)	0.39	1.87	-
Channel Flow	Flow Length (ft)	518	1,001	-
	Channel Slope (%)	2.50	2.50	-
	Cross Section Area (ft2)	16.34	16.34	-
	Manning's roughness	0.03 (Grass)	0.03 (Grass)	-
	Wetted perimeter (ft)	20.14	20.14	-
	Computed Flow Time (min)	1.26	2.44	-

TOTAL 22.87 min

Table 3-B. Time of Concentration Sub-basin #2

Flow Type	Flow Properties	Segment 1	Segment 2	Segment 3
Sheet Flow	Flow Length (ft)	100.0	-	-
	Flow Slope (%)	30	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.4 (Avg. Grass Cover)	-	-
	Computed Flow Time (min)	6.73	-	-
Shallow Concentr-ated Flow	Flow Length (ft)	394	47	234
	Flow Slope (%)	2.0	33.0	2.0
	Surface Type	Unpaved	Unpaved	Unpaved
	Flow Velocity (ft/sec)	2.28	9.27	2.28
	Computed Flow Time (min)	2.88	0.08	1.71
Channel Flow	Flow Length (ft)	422	-	-
	Channel Slope (%)	2.00	-	-
	Cross Section Area (ft2)	16.34	-	-
	Manning's roughness	0.02 (closure turf)	-	-
	Wetted perimeter (ft)	20.14	-	-
	Computed Flow Time (min)	0.77	-	-

TOTAL 12.17 min

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Table 3-C. Time of Concentration Sub-basin #3

Flow Type	Flow Properties	Segment 1	Segment 2	Segment 3
Sheet Flow	Flow Length (ft)	100.0	-	-
	Flow Slope (%)	1.6	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.22 (closure turf)	-	-
	Computed Flow Time (min)	13.48	-	-
Shallow Concentr-ated Flow	Flow Length (ft)	338	122	90
	Flow Slope (%)	3.8	0.5	17.6
	Surface Type	Unpaved	Unpaved	Unpaved
	Flow Velocity (ft/sec)	3.15	1.14	6.77
	Computed Flow Time (min)	1.79	1.78	0.22
Channel Flow	Flow Length (ft)	919.00	-	-
	Channel Slope (%)	0.50	-	-
	Cross Section Area (ft2)	10.00	-	-
	Manning's roughness	0.02 (closure turf)	-	-
	Wetted perimeter (ft)	18.00	-	-
	Computed Flow Time (min)	4.30	-	-

TOTAL 21.58 min

3) NORTH POND

3.1) Stage-Storage

The stage-storage curve was computed within AutoCAD Civil 3D using the Average Area Method. Surface information was obtained from Golder's North Pond gradings.

Table 4 Stage-storage relationship for the proposed north pond.

Elev. (ft)	Stage (ft)	Contour Area (Ac)	Incremental Vol AC-ft	Cumulative Vol. AC-ft	
774	**0**	**0.00**	**N/A**	**0.00**	**Bottom of Pond**
775	1	0.02	0.01	0.01	
776	2	0.06	0.04	0.05	
777	3	0.14	0.10	0.14	
778	4	0.26	0.20	0.34	
779	5	0.44	0.35	0.69	
780	6	0.66	0.55	1.24	
781	7	0.81	0.74	1.98	
782	8	0.93	0.87	2.85	
783	9	1.03	0.98	3.83	
784	10	1.13	1.08	4.91	
785	11	1.20	1.17	6.07	**Auxiliary Spillway Crest**

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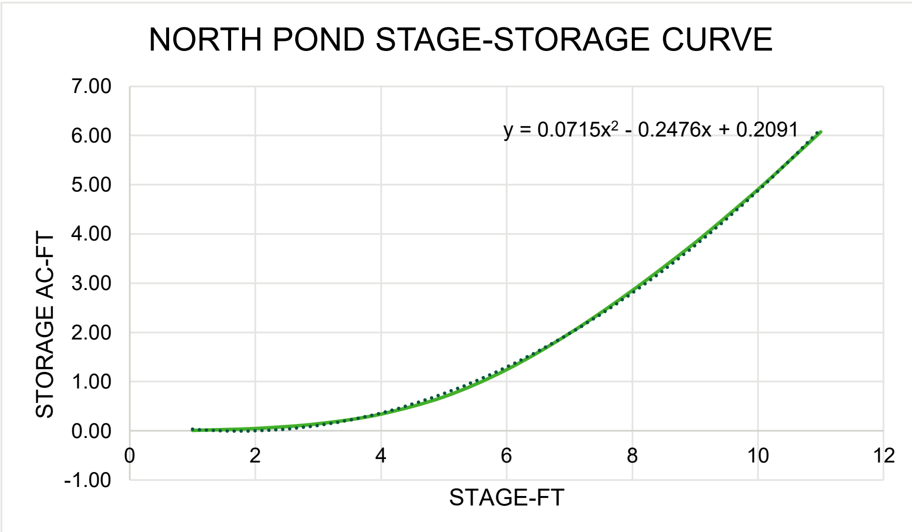


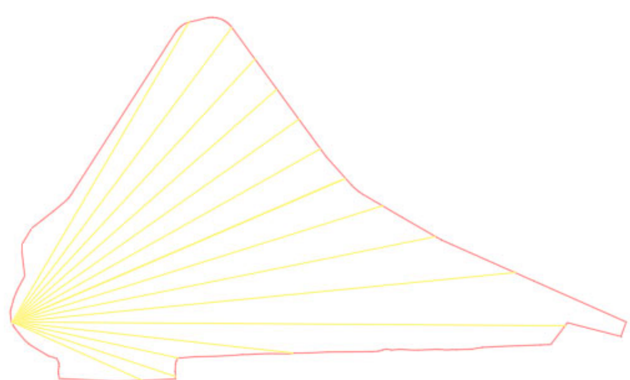
3.2) North Pond Wave-Action Freeboard.

The wave action freeboard is calculated for the 100-year storm based on the method described in the NRCS "Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines" TR-56 (2014). The fetch lengths are estimated in ArcGIS based on the 100-year WSE contour.

Maximum Fetch Length

Angle (a) (degrees)	Fetch Length (x) (feet)	cos (a)	x cos (a)
-42	227	0.74	168.69
-36	239	0.81	193.36
-30	232	0.87	200.92
-24	229	0.91	209.20
-18	228	0.95	216.84
-12	229	0.98	224.00
-6	235	0.99	233.71
0	251	1.00	251.00
6	279	0.99	277.47
12	326	0.98	318.88
18	357	0.95	339.53
24	178	0.91	162.61
30	109	0.87	94.40
36	111	0.81	89.80
42	91	0.74	67.63

Schematic of pond **100 year contour 783.30 ft**



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Maximum Effective Fetch (F_e)

Σ [cos (a)] = 14

Σ [x cos (a)] = 3,048

$$F_e = \frac{\sum[x\cos(a)]}{\sum[\cos(a)]}$$ = 226 feet; 0.04 miles

Wave Run-up (R)
(assumes fetch limited - duration of wind is adequate to achieve max H_s)

10 m Wind Velocity (V_w) = 50.0 mph — Velocity prescribed in GASDP Eng. Guidelines

Significant Wave Height (H_s) = 0.33 feet — $H_s = 0.0232 \times V_w^{1.06} \times F_e^{0.47}$ — TR-56 eq. 3

Deep Water Wave length (L) = 6.6 feet — $L = 1.24 \times V_w^{0.88} \times F_e^{0.56}$ — TR-56 eq. 10

Embankment Slope = 3 :1 (h:v)

Embankment Angle (θ) = 18.43 degrees

Surf Parameter (δ) = 1.49 — $\delta = \frac{\tan(\theta)}{\sqrt{\frac{H_s}{L}}}$ — TR-56 eq. 9

Wave Run-up (R) = 0.79 feet — $R = H_s \times \left(\frac{1.286 \times \delta}{1 + 0.247 \times \delta}\right) \times 1.7$ — TR-56 eq. 8

Wind Setup (S)

Average Depth (D) = 5.5 feet — $D = \frac{depth\ at\ toe}{2}$

Maximum Fetch Distance (F) = 0.068 miles

Wave Setup (S) = 0.02 feet — $S = \frac{V_w^2 \times F}{1400 \times D}$ — TR-56 eq. 6

Total Wave Action Height for Freeboard (FB)

Wave-Action Height (FB) = 0.8 feet — $FB = R + S$

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4) PROPOSED RISER STRUCTURE

The riser structure was designed to convey outflows below the Pre-Development discharges for the 2 and 25-year storms. Pre-Development discharges are presented in Table 5-A, also refer to Calculation Sheet C for detailed analysis. Table 5-B shows the proposed dimensions and model inputs for the riser structure and outflow pipe. The rating curve for the proposed riser is presented in Table 5-C.

Table 5-A. Pre-Development Runoff.

Storm event (years)	Pre-Development runoff (cfs)
2	20.16
25	49.22
100	70.56

Table 5-B. Proposed Riser Dimensions

Riser Component	Golder's Design	Model Inputs	
Top Weir	For the top weir we propose a **4'x10'** (plan section) placed at **782.5 FT-MSL**	Discharge Coeff.	3.33
		Contraction Type	Both Ends
Middle Step Weir	For the middle weir we propose a **1'x1.5'** (side section) placed at **781 FT-MSL**	Discharge Coeff.	3.33
		Contraction Type	Both Ends
Low Level Weir/Orifice	For the low level weir/orifice we propose **three circular sections of 9" in diameter** and placed at **774 FT-MSL**	Orifice Type	Side
		Orifice Coeff.	0.614
First Section Pipe	The first section pipe is a **Ø36" HDPE HDR17** connected to the riser structure **at 770.9 FT-MSL** (invert elev.) and to a downstream manhole at **769.3 FT-MSL** (invert elev.). **The length of this pipe is 104 feet with a 1.5% slope.**	Length	104 feet
		Inlet Invert elev.	770.9 ft-msl
		Outlet Invert elev.	769.3 ft-msl
		Manning's n	0.012
		Entrance losses	0.5
		Exit/bend losses	0.5
Second Section Pipe	The second section pipe is a **Ø36" HDPE HDR17** connected to the downstream manhole **at 769.1 FT-MSL** (invert elev.) and discharge freely to a U-Type energy dissipator at **768 FT-MSL** (invert elev.). **The length of this pipe is 68.6 feet with a 1.6% slope.**	Length	68.6 feet
		Inlet Invert elev.	769.1 ft-msl
		Outlet Invert elev.	768 ft-msl
		Manning's n	0.012
		Entrance losses	0.5
		Exit/bend losses	0.5

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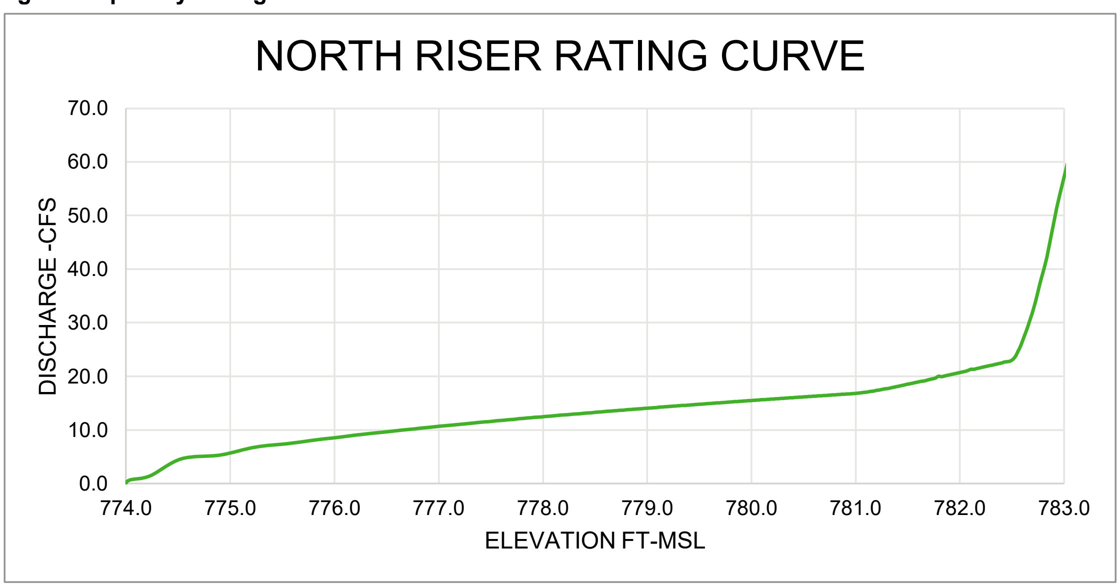
Table 5-C Proposed Riser Rating Curve

Elevation FT-MSL	Discharge (CFS)	Flow Type
783.0	61.5	Top Weir + Middle Step Weir + Low Level Orifice
782.5	23.1	Top Weir + Middle Step Weir + Low Level Orifice
782.0	20.8	Top Weir + Middle Step Weir + Low Level Orifice
781.5	18.5	Middle Step Weir + Low Level Orifice
781.0	16.8	Middle Step Weir + Low Level Orifice
780.5	16.2	Middle Step Weir + Low Level Orifice
780.0	15.5	Low Level Orifice
779.5	14.8	Low Level Orifice
779.0	14.1	Low Level Orifice
778.5	13.3	Low Level Orifice
778.0	12.5	Low Level Orifice
777.5	11.5	Low Level Orifice
777.0	10.7	Low Level Orifice
776.5	9.7	Low Level Orifice
776.0	8.5	Low Level Orifice
775.5	7.1	Low Level Orifice
775.0	6.3	Low Level Orifice
774.5	3.9	Low Level Weir
774.0	0.0	Low Level Weir

Figure 1 Spillway Rating Curve



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5) Model results

The storm events were routed using Autodesk Storm Sanitary Analysis (SSA) 2019. The model results are shown below.

5.1) Basins Runoff

Table 6-A. Basins runoff results for Basin #1

100yr	Peak Runoff (cfs)	55.15	cfs
	Total Runoff (in)	4.97	in
	Total Infiltration (in)	2.57	in
25yr	Peak Runoff (cfs)	40.08	cfs
	Total Runoff (in)	3.60	in
	Total Infiltration (in)	2.42	in
2yr	Peak Runoff (cfs)	18.72	cfs
	Total Runoff (in)	1.69	in
	Total Infiltration (in)	2.06	in

Table 6-B. Basins runoff results for Basin #2

100yr	Peak Runoff (cfs)	57.48	cfs
	Total Runoff (in)	6.24	in
	Total Infiltration (in)	1.3	in
25yr	Peak Runoff (cfs)	44.53	cfs
	Total Runoff (in)	4.76	in
	Total Infiltration (in)	1.27	in
2yr	Peak Runoff (cfs)	24.97	cfs
	Total Runoff (in)	2.59	in
	Total Infiltration (in)	1.16	in

Table 6-C. Basins runoff results for Basin #3

100yr	Peak Runoff (cfs)	71.47	cfs
	Total Runoff (in)	6.94	in
	Total Infiltration (in)	0.60	in
25yr	Peak Runoff (cfs)	56.77	cfs
	Total Runoff (in)	5.43	in
	Total Infiltration (in)	0.59	in
2yr	Peak Runoff (cfs)	34.15	cfs
	Total Runoff (in)	3.19	in
	Total Infiltration (in)	0.57	in

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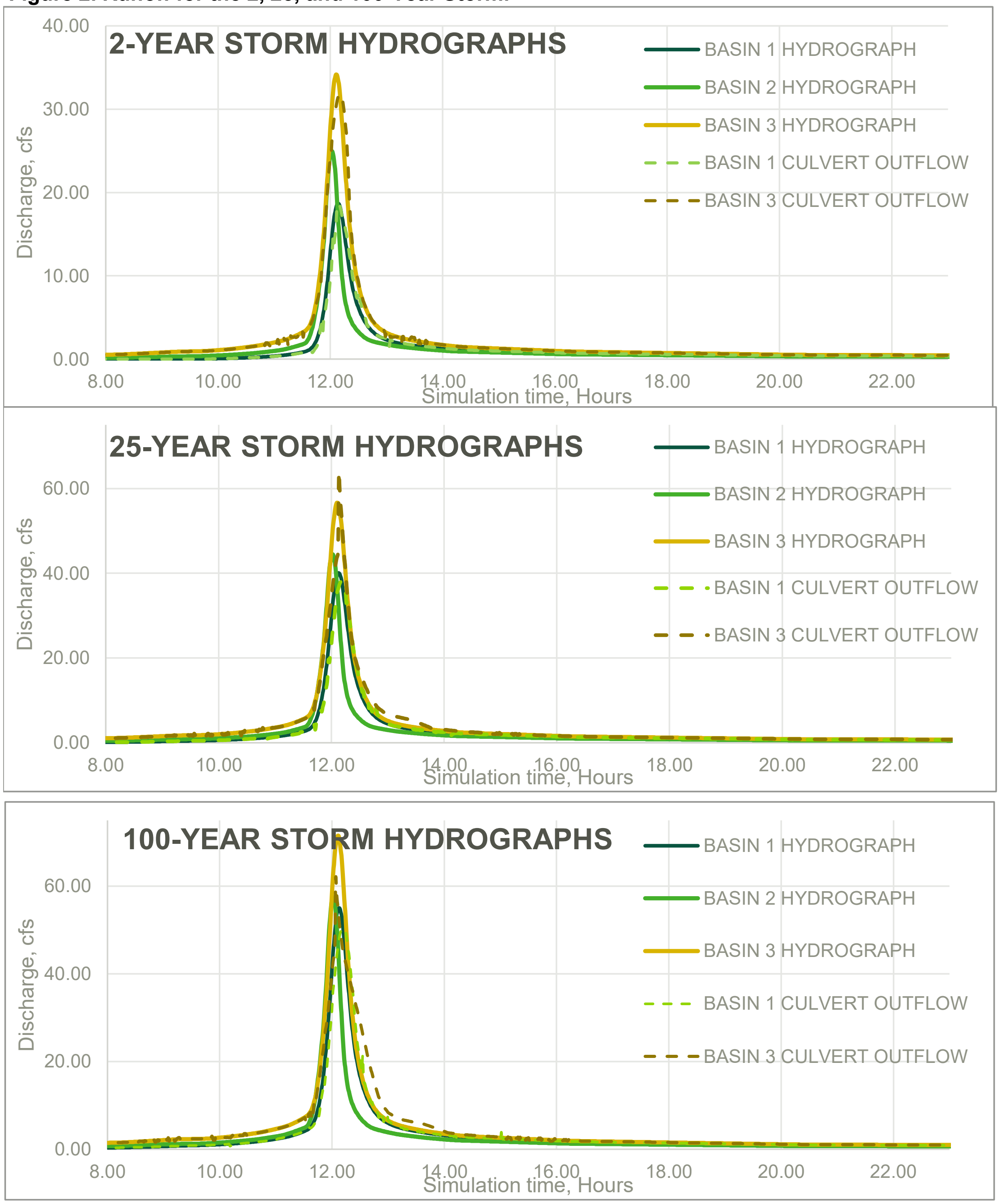
Figure 2. Runoff for the 2, 25, and 100-Year Storm.



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5.2) Pond Stage

The stage elevations were measured from the upstream slope based on the available topography.

Table 7. Reservoir Stage Result for Existing Conditions

100yr	Maximum WSE (ft)	783.30	ft-msl
	Available Freeboard (ft)	1.70	ft
	Peak Inflow (cfs)	159.91	cfs
25yr	Maximum WSE (ft)	782.81	ft-msl
	Available Freeboard (ft)	2.19	ft
	Peak Inflow (cfs)	131.52	cfs
2yr	Maximum WSE (ft)	780.87	ft-msl
	Available Freeboard (ft)	4.13	ft
	Peak Inflow (cfs)	68.85	cfs

Bottom of pond	774
Top of Pond	785

Figure 3 Pond Stage During the 2,25, and 100-Year Storm

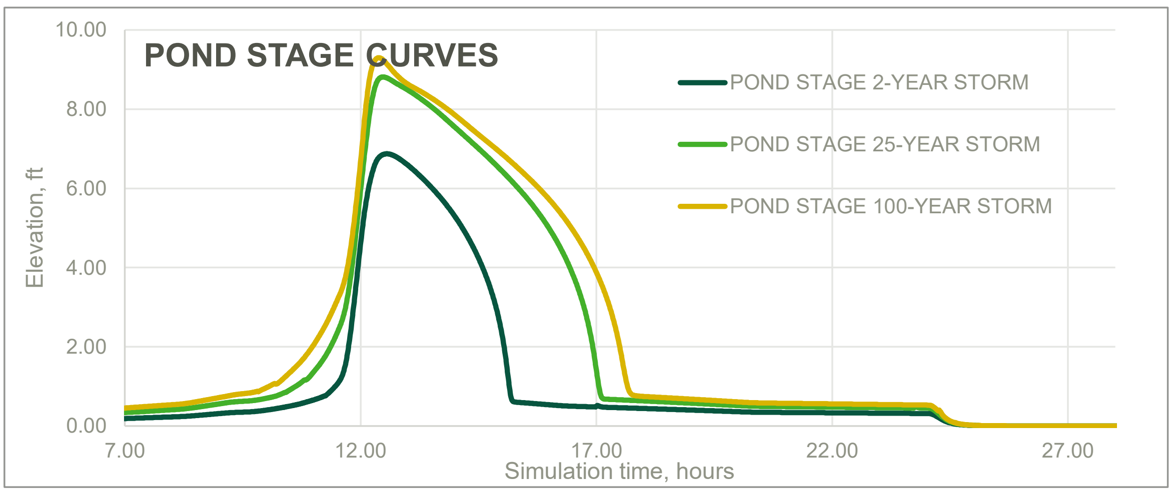

Table 8. Outflow for the 2,25,and 100-Year Storm Event.

100yrs	**Pre-Development**	**70.56**	cfs
	North Pond Peak Flow	77.32	cfs
	Max Velocity	14.05	ft/sec
25yr	**Pre-Development**	**49.22**	cfs
	Peak Flow	39.44	cfs
	Max Velocity	9.75	ft/sec
2yr	**Pre-Development**	**20.16**	cfs
	Peak Flow	16.65	cfs
	Max Velocity	8.17	ft/sec

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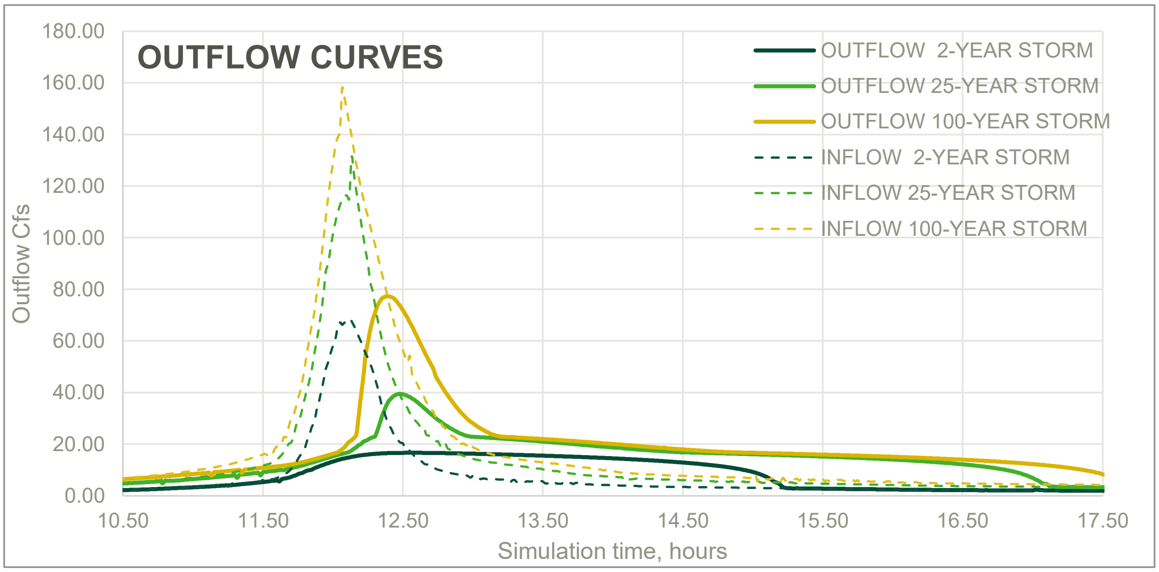
Figure 4. Outflow Curves



6) References

GASDP(2015), *Engineer Guidelines.*

NOAA(1978), *Probable Maximum Precipitation Estimates, United States East of the 105th Meridian* .

NRCS(1986), *Technical Release No. 55 (Urban Hydrology for Small Watersheds).*

NRCS(1993), *National Engineering Handbook Part 630 (Hydrology)* .

Autodesk(2013), *Autodesk Storm and Sanitary Analysis User's Guide.*

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OBJECTIVE

This calculations sheet covers the hydrologic and hydraulic analysis for the proposed conditions of the AP-1 South Stormwater Attenuation Pond at Plant McDonough-Atkinson (Plant McDonough) located in Cobb County, Georgia.

METHOD

For this analysis, we used The National Engineering Handbook (NEH) part 630 (Hydrology) and the NRCS Technical Release No. 55 (Urban Hydrology for Small Watersheds).

CALCULATIONS

1) RAINFALL

The design storm precipitations are from the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server (PFDS). The table below shows the precipitation events considered for this design.

Agency	Event	Depth (in)	Distribution	ARC	Sourse
FEMA	100-year, 24 hour	7.54	Type II	II	NOAA Atlas 14
FEMA	25-year, 24 hour	6.02	Type II	II	NOAA Atlas 14
FEMA	2-year, 24 hour	3.75	Type II	II	NOAA Atlas 14

2) WATERSHED

The watershed draining to the AP-1 South Pond was delineated using the topography and contours elevations provided by Georgia Land Department and Metro Engineering and Surveying Co, Inc. The watershed was divided into three basins. Two of the basins have a culvert structure that conveys runoff to the South Pond (refer to attached basin maps). The tables below present a summary of the hydrologic properties of each subbasin.

Table 1-A. Basin #1

Area and delineation	1.41	acres
Time of Concentration	12.12	minutes
CN Computation	95	

Table 1-B. Basin #2

Area and delineation	1.79	acres
Time of Concentration	13.12	minutes
CN Computation	95	

Table 1-C. Basin #3

Area and delineation	9.42	acres
Time of Concentration	15.80	minutes
CN Computation	95	

2.1) Curve Number

The curve number (ARC II) was chosen based on the recommended curve numbers given in Table 9-1, 9-2, and 9-5 of the National Engineering Handbook (NEH) for each land use. The land use areas were delineated using ArcMap. The soil type on the watershed is considered type B (Moderately Low Runoff Potential) and D (high runoff potential). The land cover present in the watershed is listed below.

Gravel
Closure Turf

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Table 2.A- Curve Number Sub-basin #1

Soil Group	Land Use	CN	Area	CNxArea
D	Closure Turf	95	1.2	114
	Gravel	95	0.2	19
TOTALS			1.4	133
			CN	**95**

Table 2.B- Curve Number Sub-basin #2

Soil Group	Land Use	CN	Area	CNxArea
D	Closure Turf	95	1.79	170.05
TOTALS			1.79	170.05
			CN	**95**

Table 2.C- Curve Number Sub-basin #3

Soil Group	Land Use	CN	Area	CNxArea
D	Closure Turf	95	9.42	894.9
TOTALS			9.42	894.9
			CN	**95**

2.2) Time of Concentration

The time of concentration is calculated within Autodesk Storm and Sanitary Analysis 2019 (SSA) using the Velocity Method described in the NRCS technical release No. 55. The flow length and slope were estimated using the ArcMap Spatial Analysis tools and Autocad Civil 3D Watershed Analysis tools. Sheet, Concentrated, and Channel flows properties were determined using imagery information and the provided topography. Tables 3A - 3C exhibit the flow properties considered and the time of concentration results.

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Table 3-A. Time of Concentration Sub-basin #1

Flow Type	Flow Properties	segment 1	segment 2	segment 3
Sheet Flow	Flow Length (ft)	100	-	-
	Flow Slope (%)	3.00	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.22 (Closure Turf)	-	-
	Computed Flow Time (min)	10.48	-	-
Shallow Concentrated Flow	Flow Length (ft)	274	-	-
	Flow Slope (%)	3.00	-	-
	Surface Type	Unpaved	-	-
	Flow Velocity (ft/sec)	2.79	-	-
	Computed Flow Time (min)	1.64	-	-
Channel Flow	Flow Length (ft)	-	-	-
	Channel Slope (%)	-	-	-
	Cross Section Area (ft2)	-	-	-
	Manning's roughness	-	-	-
	Wetted perimeter (ft)	-	-	-
	Computed Flow Time (min)	-	-	-

TOTAL	**12.12 min**

Table 3-B. Time of Concentration Sub-basin #2

Flow Type	Flow Properties	segment 1	segment 2	segment 3
Sheet Flow	Flow Length (ft)	100.0	-	-
	Flow Slope (%)	3	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.22 (Closure Turf)	-	-
	Computed Flow Time (min)	10.48	-	-
Shallow Concentrated Flow	Flow Length (ft)	280	-	-
	Flow Slope (%)	3.0	-	-
	Surface Type	Unpaved	-	-
	Flow Velocity (ft/sec)	2.79	-	-
	Computed Flow Time (min)	1.67	-	-
Channel Flow	Flow Length (ft)	371	-	-
	Channel Slope (%)	1.00	-	-
	Cross Section Area (ft2)	8.00	-	-
	Manning's roughness	0.02 (Closure Turf)	-	-
	Wetted perimeter (ft)	10.00	-	-
	Computed Flow Time (min)	0.96	-	-

TOTAL	**13.12 min**

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Table 3-C. Time of Concentration Sub-basin #3

Flow Type	Flow Properties	segment 1	segment 2	segment 3
Sheet Flow	Flow Length (ft)	100.0	-	-
	Flow Slope (%)	3.0	-	-
	2yr-24hr rainfall (in)	3.73	-	-
	Manning's roughness	0.22 (Closure Turf)	-	-
	Computed Flow Time (min)	10.48	-	-
Shallow Concentr-ated Flow	Flow Length (ft)	220	96	270
	Flow Slope (%)	3.0	16.1	1.0
	Surface Type	Unpaved	Unpaved	Unpaved
	Flow Velocity (ft/sec)	2.79	6.47	1.61
	Computed Flow Time (min)	1.31	0.25	2.80
Channel Flow	Flow Length (ft)	370	-	-
	Channel Slope (%)	1.00	-	-
	Cross Section Area (ft2)	8.00	-	-
	Manning's roughness	0.02 (Closure Turf)	-	-
	Wetted perimeter (ft)	10.00	-	-
	Computed Flow Time (min)	0.96	-	-

TOTAL	**15.8 min**

3) SOUTH POND

3.1) Stage-Storage

The stage-storage curve was computed within Autocad Civil 3D using the Average Area Method. Surface information was obtained from Golder's South Pond gradings.

Table 4 Stage-storage relationship for the proposed South Pond

Elev. (ft)	Stage (ft)	Contour Area (Ac)	Incremental Vol AC-ft	Cumulative Vol. AC-ft	
777	0.00	**0.01**	**N/A**	**0.00**	**Bottom of Pond**
778	1.00	0.04	0.02	0.02	
779	2.00	0.08	0.06	0.08	
780	3.00	0.13	0.11	0.19	
781	4.00	0.23	0.18	0.37	
782	5.00	0.34	0.28	0.65	
783	6.00	0.47	0.40	1.06	
784	7.00	0.61	0.54	1.60	
785	8.00	0.79	0.70	2.30	**Auxiliary Spillway Crest**

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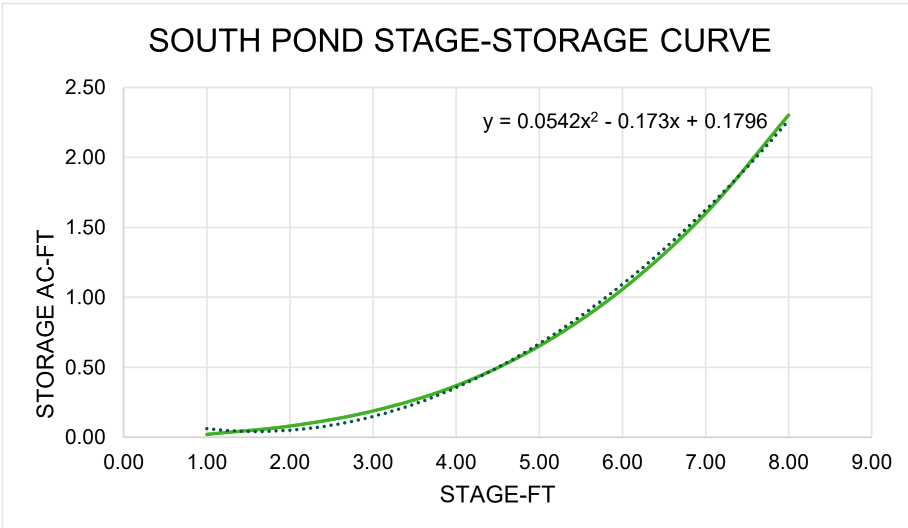

3.2) South Pond Wave-Action Freeboard.

The wave action freeboard is calculated for the 100-year storm based on the method described in the NRCS "Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines" TR-56 (2014). The fetch lengths are estimated in ArcGIS based on the 100-year WSE contour.

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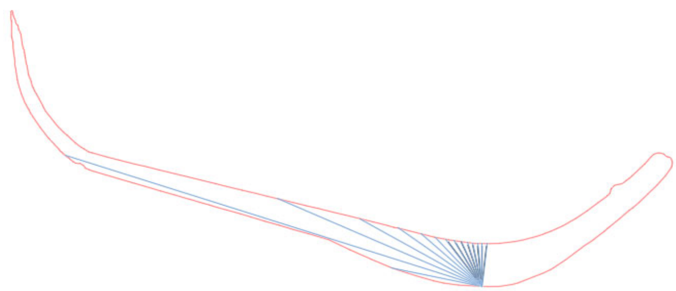
Maximum Fetch Length

Schematic of pond **100 year contour 783.2 ft**

Angle (a) (degrees)	Fetch Length (x) (feet)	cos (a)	x cos (a)
-42	91.00	0.74	67.63
-36	427.00	0.81	345.45
-30	218.00	0.87	188.79
-24	137.00	0.91	125.16
-18	101.00	0.95	96.06
-12	80.00	0.98	78.25
-6	67.00	0.99	66.63
0	59.00	1.00	59.00
6	53.00	0.99	52.71
12	50.00	0.98	48.91
18	47.00	0.95	44.70
24	45.00	0.91	41.11
30	44.00	0.87	38.11
36	44.00	0.81	35.60
42	44.00	0.74	32.70



Maximum Effective Fetch (F_e)

Σ [cos (a)] = 14
Σ [x cos (a)] = 1,321

$$F_e = \frac{\sum[x\cos(a)]}{\sum[\cos(a)]}$$ = 98 feet, 0.02 miles

Wave Run-up (R)
(assumes fetch limited - duration of wind is adequate to achieve max H_s)

10 m Wind Velocity (V_w) = 50.0 mph — Velocity prescribed in GASDP Eng. Guidelines

Significant Wave Height (H_s) = 0.22 feet — $H_s = 0.0232 \times V_w^{1.06} \times F_e^{0.47}$ — TR-56 eq. 3

Deep Water Wave length (L) = 4.2 feet — $L = 1.24 \times V_w^{0.88} \times F_e^{0.56}$ — TR-56 eq. 10

Embankment Slope = 3 :1 (h:v)

Embankment Angle (θ) = 18.43 degrees

Surf Parameter (δ) = 1.43 — $\delta = \dfrac{\tan(\theta)}{\sqrt{\frac{H_s}{L}}}$ — TR-56 eq. 9

Wave Run-up (R) = 0.52 feet — $R = H_s \times \left(\dfrac{1.286 \times \delta}{1 + 0.247 \times \delta}\right) \times 1.7$ — TR-56 eq. 8

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Wind Setup (S)

Average Depth (D) = 4.0 feet $D = \frac{depth\ at\ toe}{2}$

Maximum Fetch Distance (F) = 0.081 miles

Wave Setup (S) = 0.036 feet $S = \frac{V_w^2 \times F}{1400 \times D}$ TR-56 eq. 6

Total Wave Action Height for Freeboard (FB)

Wave-Action Height (FB) = 0.6 feet $FB = R + S$

4) PROPOSED RISER STRUCTURE

The riser structure was designed to convey outflows below the Pre-Development discharges for the 2 and 25 year storms. Pre-Development discharges are presented in Table 5-A, also refer to Calculation Sheet C for detailed analysis. Table 5-B shows the proposed dimensions and model inputs for the riser structure and outflow pipe. The rating curve for the proposed riser is presented in Table 5-C.

Table 5-A. Pre-Development Runoff.

Storm event (years)	Pre-Development runoff (cfs)
2	29.26
25	73.27
100	105.67

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Table 5-B. Proposed Riser Dimensions

Riser Component	Golder's Design	Model Imputs	
Top Weir	For the top weir we propose a **4'x4'** (plan section) placed at **782 FT-MSL**	Discharge Coeff.	3.33
		Contraction Type	Both Ends
Middle Step Weir	For the middle weir we propose a **1.5'x1.5'** (side section) placed at **780.5 FT-MSL**	Discharge Coeff.	3.33
		Contraction Type	Both Ends
Low Level Weir/Orifice	For the low level weir/orifice we propose **three circular sections of 9"** in diameter and placed at **777 FT-MSL**	Orifice Type	Side
		Orifice Coeff.	0.614
First Section Pipe	The first section pipe is a **Ø30" HDPE HDR 17** connected to the riser structure at **774.2 FT-MSL** (invert elev.) and to a downstream manhole #1 at **767.5 FT-MSL** (invert elev.). **The length of this pipe is 79.4 feet with a 8.4% slope.**	Length	79.4 feet
		Inlet Invert elev.	774.2 ft-msl
		Outlet Invert elev.	767.5 ft-msl
		Mnning's n	0.012
		Entrance losses	0.5
		Exit/bend losses	0.5
Second Section Pipe	The second section pipe is a **Ø30" HDPE HDR 17** connected to the downstream manhole #1 at **759.4 FT-MSL** (invert elev.) and to the downstream manhole #2 at **757.7 FT-MSL** (invert elev.). **The length of this pipe is 21.1 feet with a 8% slope.**	Length	21.1 feet
		Inlet Invert elev.	759.4 ft-msl
		Outlet Invert elev.	757.7 ft-msl
		Mnning's n	0.012
		Entrance losses	0.5
		Exit/bend losses	0.5
Third Section Pipe	The Third section pipe is a **Ø30" HDPE HDR17** connected to the downstream manhole #2 at **754.6 FT-MSL** (invert elev.) and discharge freely to a type U energy dissipator at **754.5 FT-MSL** (invert elev.). **The length of this pipe is 7 feet with a 1.4% slope.**	Length	7 feet
		Inlet Invert elev.	754.6 ft-msl
		Outlet Invert elev.	754.5 ft-msl
		Mnning's n	0.012
		Entrance losses	0.5
		Exit/bend losses	0.5

Table 5-C Proposed Riser Rating Curve

Elevation FT-MSL	Discharge (CFS)	Flow Type
782.0	50.6	Top Weir+ Middle Step Weir + Low Level Orifice
781.5	18.8	Top Weir+ Middle Step Weir + Low Level Orifice
781.0	14.4	Top Weir+ Middle Step Weir + Low Level Orifice
780.5	11.7	Top Weir+ Middle Step Weir + Low Level Orifice
780.0	10.8	Middle Step Weir + Low Level Orifice
779.5	9.7	Middle Step Weir + Low Level Orifice
779.0	8.4	Middle Step Weir + Low Level Orifice
778.5	7.2	Low Level Orifice
778.0	5.7	Low Level Orifice
777.5	2.8	Low Level Weir
777.0	0.0	Low Level Weir

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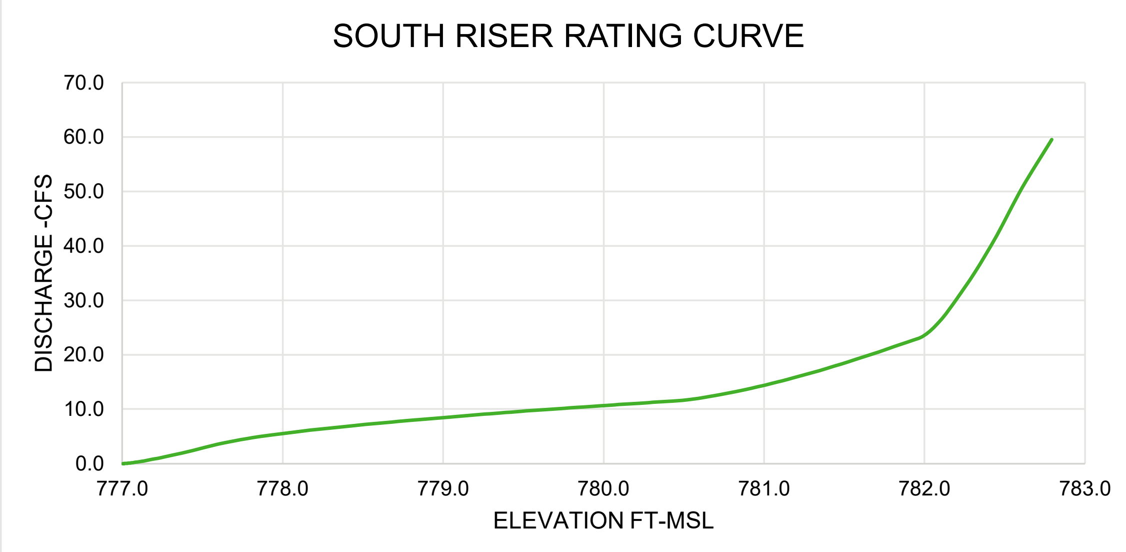
Figure 1 Spillway Rating Curve



5) Model results

The storm events were routed using Autodesk Storm Sanitary Analysis (SSA) 2019. The model results are shown below.

5.1) Basins Runoff
Table 6-A. Basins runoff results for Basin #1

100yr	Peak Runoff (cfs)	11.53	cfs
	Total Runoff (in)	6.94	in
	Total Infiltration (in)	0.6	in
25yr	Peak Runoff (cfs)	9.13	cfs
	Total Runoff (in)	5.43	in
	Total Infiltration (in)	0.59	in
2yr	Peak Runoff (cfs)	5.52	cfs
	Total Runoff (in)	3.19	in
	Total Infiltration (in)	0.57	in

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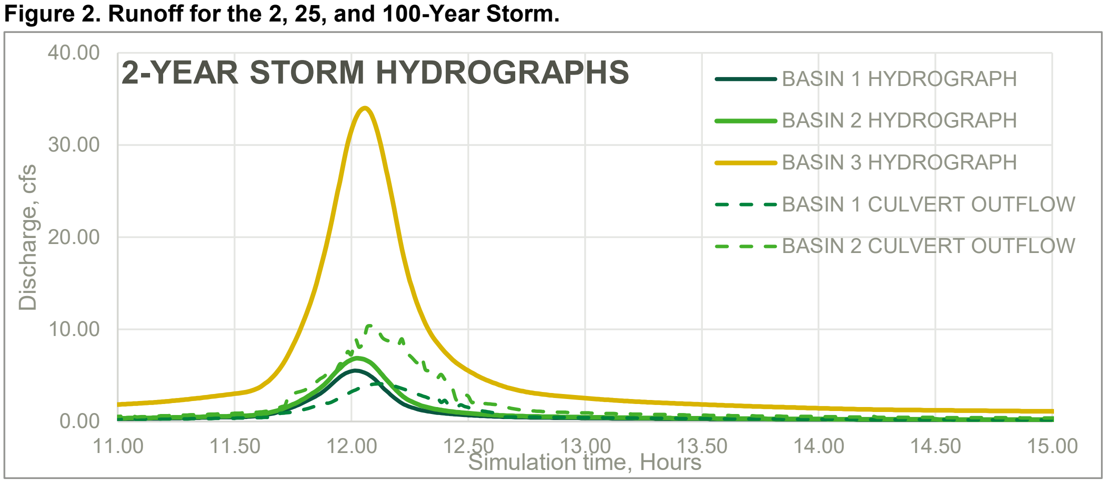
Table 6-B. Basins runoff results for Basin #2

100yr	Peak Runoff (cfs)	14.36	cfs
	Total Runoff (in)	6.94	in
	Total Infiltration (in)	0.6	in
25yr	Peak Runoff (cfs)	11.37	cfs
	Total Runoff (in)	5.43	in
	Total Infiltration (in)	0.59	in
2yr	Peak Runoff (cfs)	6.87	cfs
	Total Runoff (in)	3.19	in
	Total Infiltration (in)	0.57	in

Table 6-C. Basins runoff results for Basin #3

100yr	Peak Runoff (cfs)	71.24	cfs
	Total Runoff (in)	6.94	in
	Total Infiltration (in)	0.6	in
25yr	Peak Runoff (cfs)	56.33	cfs
	Total Runoff (in)	5.43	in
	Total Infiltration (in)	0.59	in
2yr	Peak Runoff (cfs)	34.04	cfs
	Total Runoff (in)	3.19	in
	Total Infiltration (in)	0.57	in

Figure 2. Runoff for the 2, 25, and 100-Year Storm.



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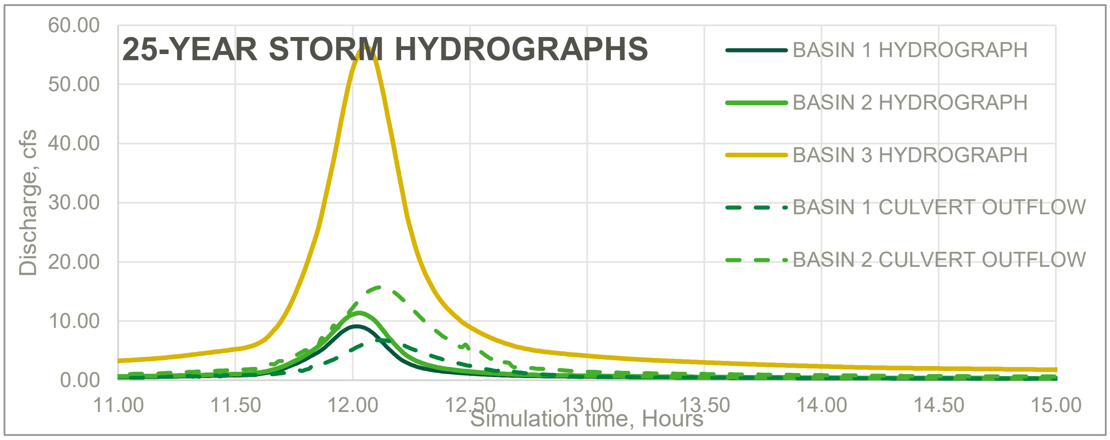

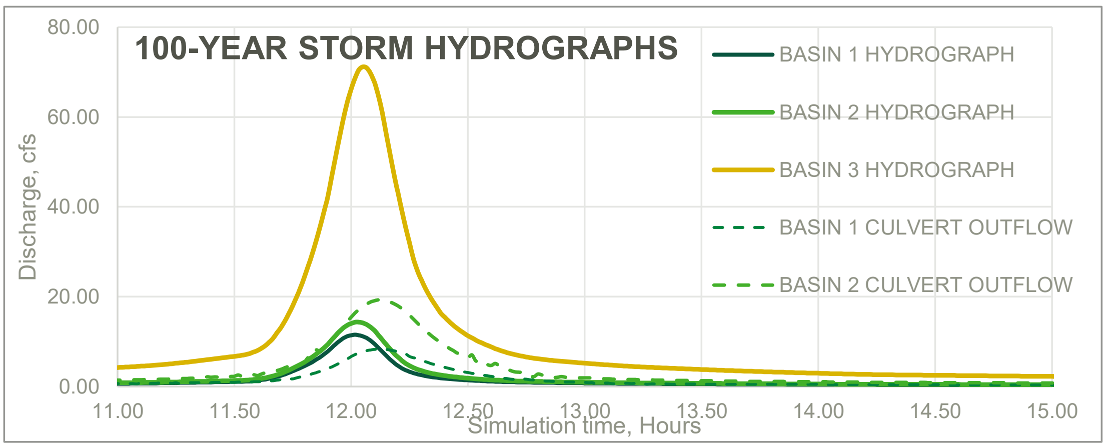

5.2) Pond Stage

The stage elevations were measured from the upstream slope based on the available topography.

Table 7. Reservoir Stage result for existing conditions

100yr	Maximum WSE (ft)	783.18	ft-msl
	Available Freeboard (ft)	1.82	ft
	Peak Inflow (cfs)	89.31	cfs
25yr	Maximum WSE (ft)	782.80	ft-msl
	Available Freeboard (ft)	2.20	ft
	Peak Inflow (cfs)	71.15	cfs
2yr	Maximum WSE (ft)	782.08	ft-msl
	Available Freeboard (ft)	2.92	ft
	Peak Inflow (cfs)	44.19	cfs

Bottom of pond	777
Top of Pond	785

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Figure 3 Pond Stage During the 2,25, and 100-Year Storm

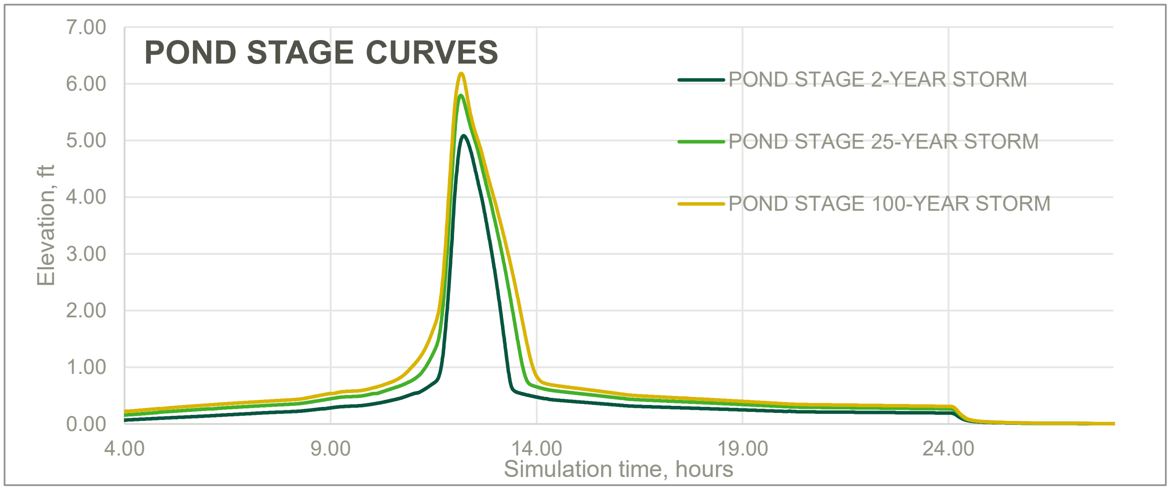
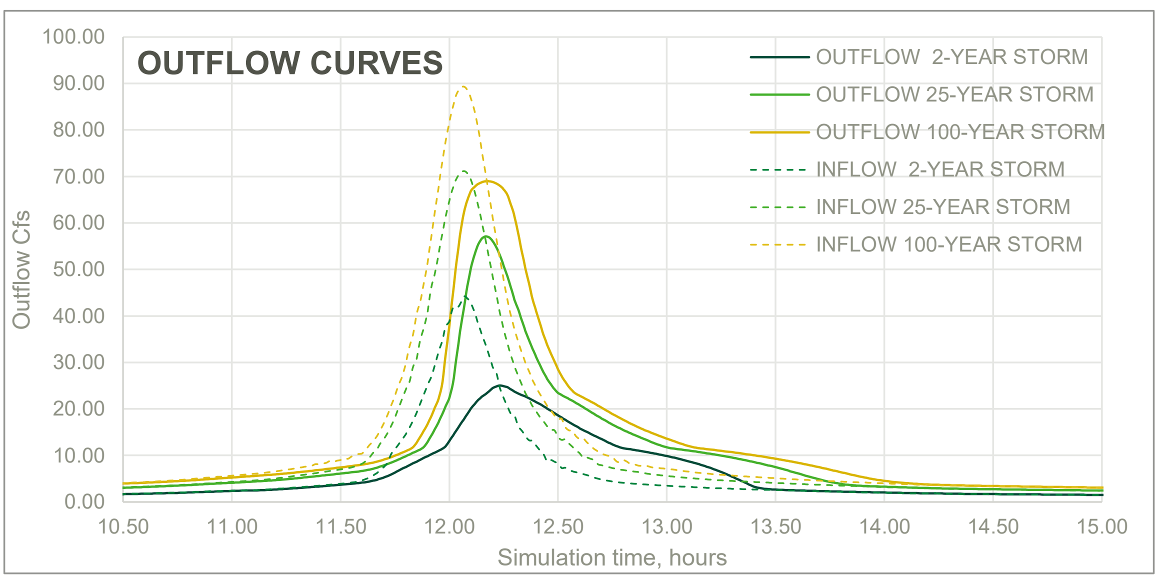


Table 8. Outflow for the 2,25,and 100-Year Storm Event.

100yrs	**Pre-Development**	**105.67**	cfs
	Peak Flow	68.97	cfs
	Max Velocity	18.05	ft/sec
25yr	**Pre-Development**	**73.3**	cfs
	Peak Flow	57.09	cfs
	Max Velocity	14.94	ft/sec
2yr	**Pre-Development**	**29.26**	cfs
	Peak Flow	25.04	cfs
	Max Velocity	7.82	ft/sec

Figure 4. Outflow Curves



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6) References

GASDP(2015), *Engineer Guidelines.*

NOAA(1978), *Probable Maximum Precipitation Estimates, United States East of the 105th Meridian* .

NRCS(1986), *Technical Release No. 55 (Urban Hydrology for Small Watersheds).*

NRCS(1993), *National Engineering Handbook Part 630 (Hydrology)* .

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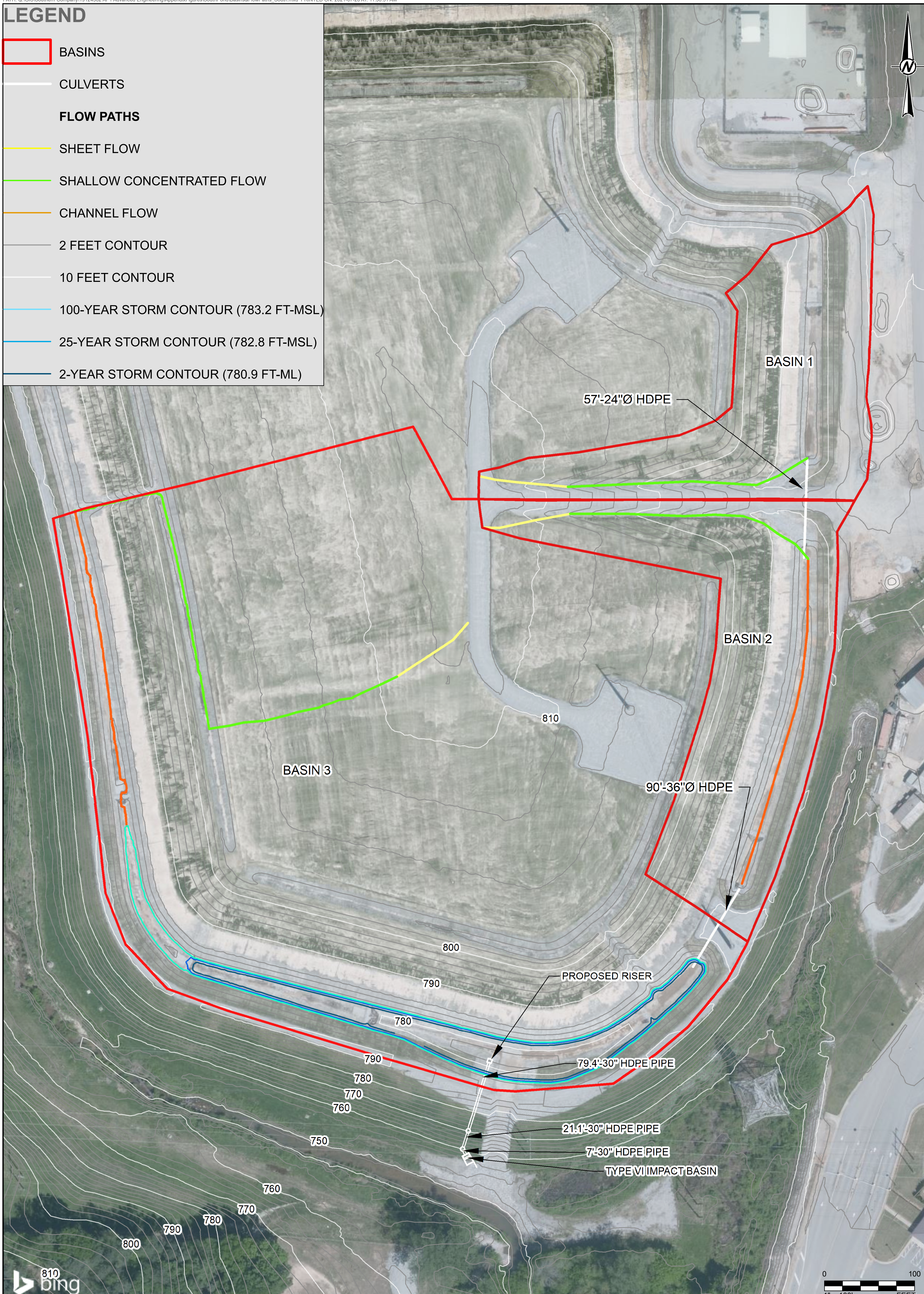

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CLIENT

SOUTHERN COMPANY / GEORGIA POWER

GOLDER
MEMBER OF WSP

DATE	2021-7-20
PREPARED	EC
REVIEW	JDG
APPROVED	GLH

PROJECT

AP1 CLOSURE DESIGN

TITLE

SOUTH POND BASINS AND FLOW PATHS

PROJECT No. 19124362

FIGURE 3

REFERENCES:

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PROJECT	AP1 CLOSURE DESIGN
TITLE	SOUTH POND BASINS AND LAND UASE
PROJECT No.	19124362
DATE	2021-04-15
PREPARED	EC
REVIEW	JDG
APPROVED	GLH
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FIGURE	4

GOLDER MEMBER OF WSP

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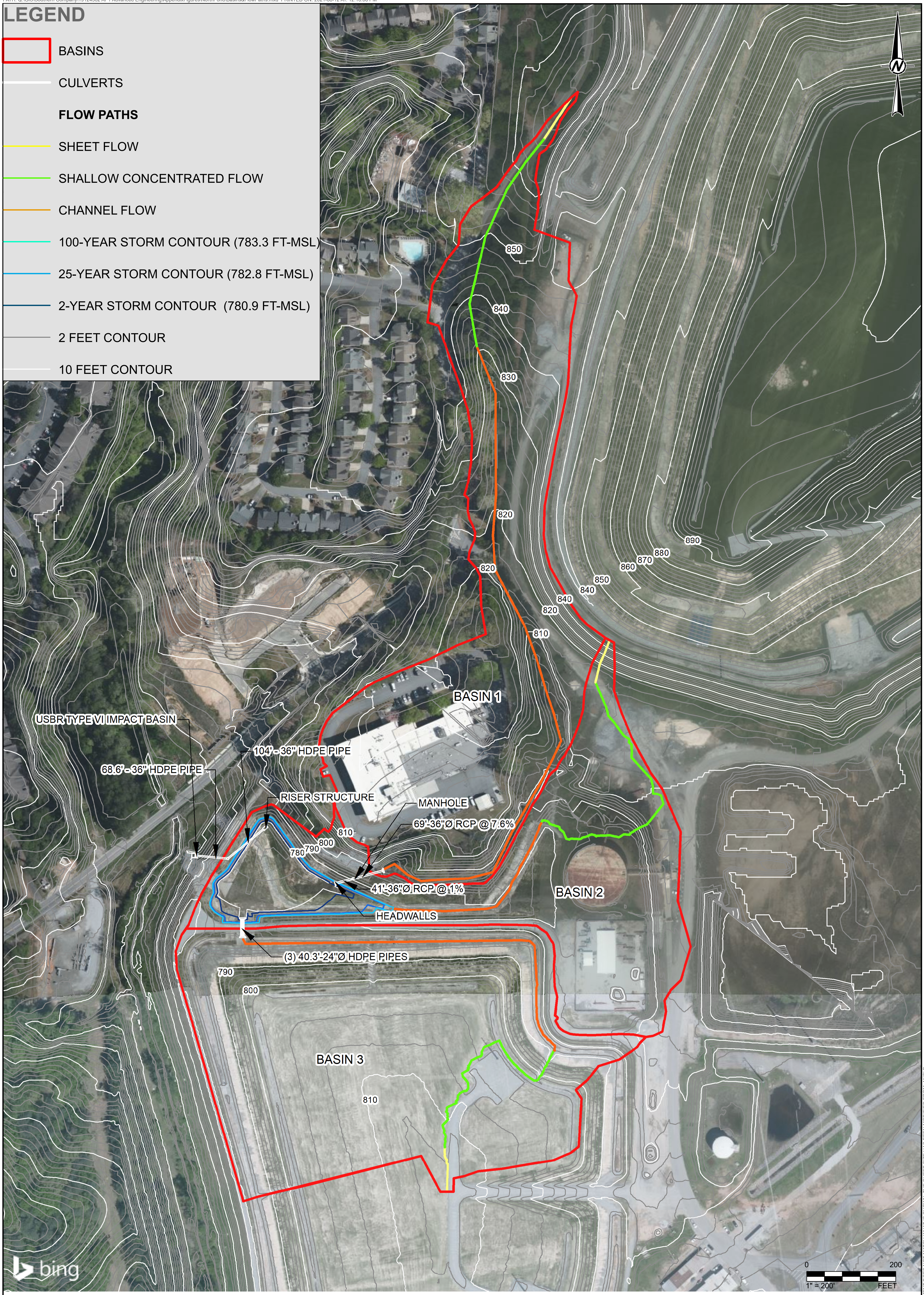

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NORTH POND BASINS AND FLOW PATHS

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FIGURE 1

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REV. 0

CLIENT
SOUTHERN COMPANY / GEORGIA POWER

GOLDER
MEMBER OF WSP

DATE	2021-08-12
PREPARED	EC
REVIEW	JDG
APPROVED	GLH

PROJECT
AP1 CLOSURE DESIGN

TITLE
NORTH POND BASINS AND LAND UASE

PROJECT No. 19124362

FIGURE 2

REFERENCES:

IMAGERY: 2021 MICROSOFT CORPORATION 2021 MAXAR CNES (2021) DISTRIBUTION AIRBUS DS

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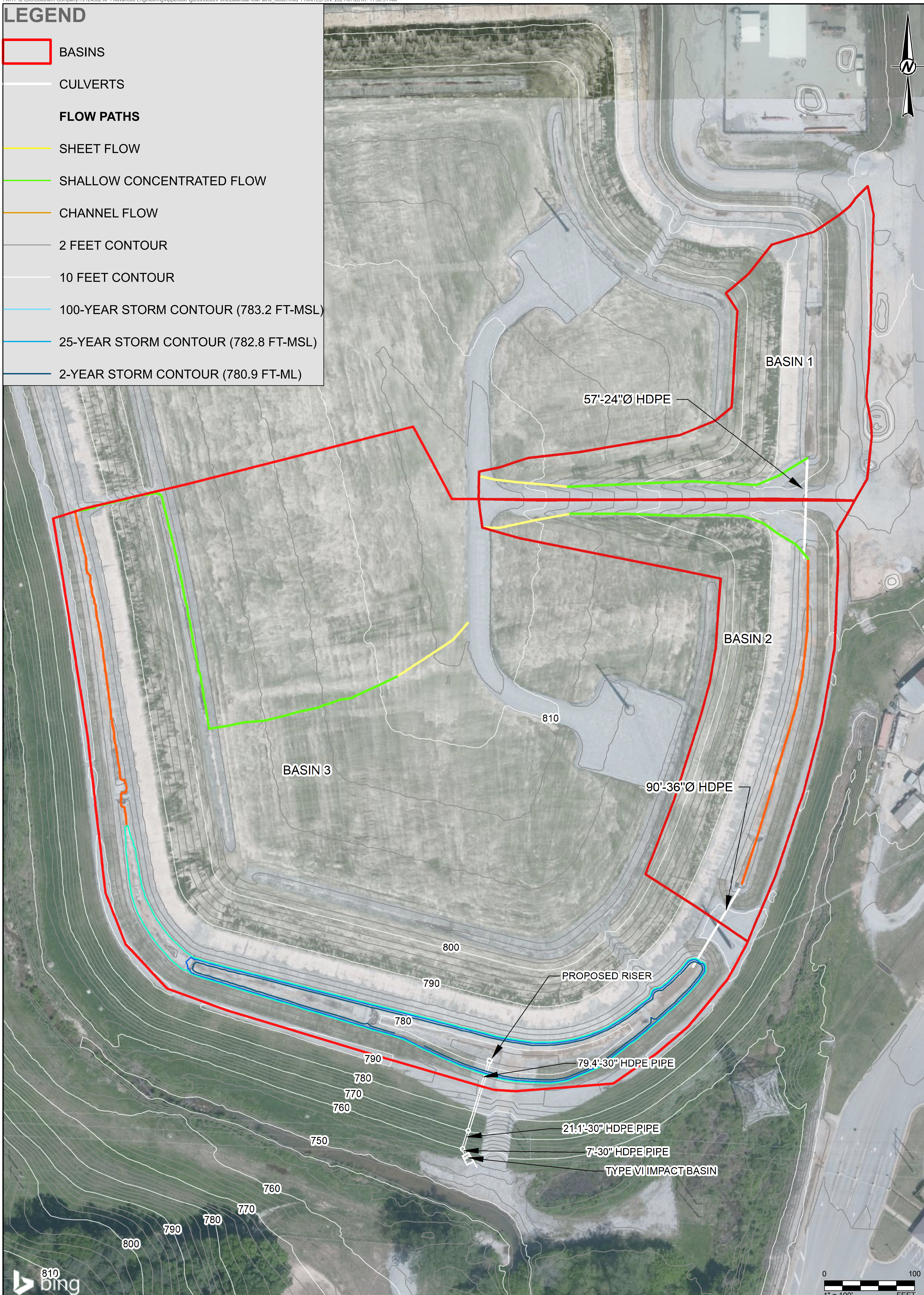

REV. 1

CLIENT

SOUTHERN COMPANY / GEORGIA POWER

GOLDER MEMBER OF WSP

DATE	2021-7-20
PREPARED	EC
REVIEW	JDG
APPROVED	GLH

PROJECT

AP1 CLOSURE DESIGN

TITLE

SOUTH POND BASINS AND FLOW PATHS

PROJECT No. 19124362

FIGURE 3

REFERENCES:

IMAGERY: 2021 MICROSOFT CORPORATION 2021 MAXAR CNES (2021) DISTRIBUTION AIRBUS DS

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REV. 0

CLIENT
SOUTHERN COMPANY / GEORGIA POWER

GOLDER
MEMBER OF WSP

DATE	2021-04-15
PREPARED	EC
REVIEW	JDG
APPROVED	GLH

PROJECT
AP1 CLOSURE DESIGN

TITLE
SOUTH POND BASINS AND LAND UASE

PROJECT No. 19124362

FIGURE 4

REFERENCES:

IMAGERY: 2021 MICROSOFT CORPORATION 2021 MAXAR CNES (2021) DISTRIBUTION AIRBUS DS

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CLIENT	SOUTHER COMPANY / GEORGIA POWER
PROJECT	AP1 CLOSURE DESIGN
DATE	2021-04-15
PREPARED	EC
REVIEW	JDG
APPROVED	GLH
TITLE	SOUTH & NORTH POND BASINS AND FLOW PATHS FOR THE PREDEVELOPMENT CONDITIONS
PROJECT No.	19124362
REV.	0
FIGURE	5

GOLDER
MEMBER OF WSP

REFERENCES:

TERRAIN INFORMATION WAS MAPPED, EDITED, ADN PUBLISHED BY THE GEOLOGICAL SURVEY FROM TOPOGRAPHY AND AERIAL PHOTOGRAPHS TAKEN IN 1952.

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APPENDIX J

AP-1 North Outfall Spillway Armoring Design Retrofit

CALCULATION

DATE August 2020

Project No. 1777449

TO Mr. Morgan French, SCS, Ms. Virginia Pantano, GPC

FROM Greg Hebeler, PE; James Grimes, PE

EMAIL ghebeler@golder.com

STABILITY ANALYSIS OF AP-1 NORTH OUTFALL RIPRAP CHUTE SPILLWAY RETROFIT

This memorandum documents the engineering analyses completed by Golder Associates Inc. (Golder) to evaluate the required riprap details (particle size, placement geometry, etc.) to provide stable armoring of the Plant McDonough CCR Unit Ash Pond 1 (AP-1) north spillway outfall retrofit. The AP-1 north spillway retrofit consists of repairing a section of grouted riprap which had become damaged during high flow conditions over the spillway. The grouted riprap is replaced with an ungrouted and regraded riprap armored channel with slightly modified grading in the retrofit design. The location of the retrofit and north outfall basin are shown in Figure 1.

Figure 1: Site Location and Watershed

Golder conducted a hydrologic analysis of the AP-1 north outfall utilizing the Autodesk Storm and Sanitary Analysis (SSA) Program and SCS methodology. Golder delineated a stormwater basin and estimated a time of concentration based on the final AP-1 closure topography and LiDAR generated in March 2018 by the site closure contractor Cooper Barnett & Page (CBP). The curve number is based on the final design and construction of the ClosureTurf system which serves as the lining and ground cover system over the basin. The basin is analyzed for the 100-year, 24 hour storm event using the SCS Type II storm distribution, using storm depths provided by the NOAA Atlas 14. The site hydrology parameters are summarized in Table 1.

Table 1: Hydrology Parameters

Parameter	AP-1 North Outfall Basin
Basin Size (acres)	17.5
Time of Concentration (minutes)	14.4
Curve Number	95
100-year, 24 hour storm depth (inches)	7.5

The AP-1 outlet structure consists of a trapezoidal overflow weir with a bottom width of 15' and 10:1 (H:V) side slopes. As the primary stormwater outlet, the weir invert matches the bottom invert of the AP-1 unit cover system within the contributing portion of the closure. A rating curve for the outflow structure is generated within the SSA model. During a 100-year, 24 hour storm the discharge over the spillway structure is estimated to be 135 cubic feet per second (cfs).

Golder sized the riprap size of the flexible spillway armoring retrofit according to the Agricultural Research Service (ARS) Rock Chute design technique presented in Technical Supplement 14C of the National Engineering Handbook Part 654. Golder selected Georgia Department of Transportation (GDOT) Type I riprap which provides a median stone size of 12 inches and therefore stable armoring according to the ARS Rock Chute Design method.

$$Equation\ TS14C-17{:}\ D_{50}=SF*12(0.233qS^{0.58})^{0.529}$$

Peak Flow Rate	*Q*	*135*	*cfs*
Spillway Base Width	*b*	*20*	*ft*
Unit Discharge	*q*	*6.75*	*cfs/ft*
Spillway Slope	*S*	*0.20*	*ft/ft*
Safety Factor	*SF*	*1.25*	
Minimum Median Stone Size	D_{50}	*11.6*	*in*

APPENDIX K

Technical Memorandum:

Phase 2 & 3 Laboratory Bench Scale Testing for Plant McDonough CCR Unit AP-1 AEM Perimeter Subsurface Barrier Wall

TECHNICAL MEMORANDUM

DATE December 13, 2023 **Reference No.** 31406440.006

TO Ms. Alex Wild, PE; Ms. Lauren Petty, PG – Georgia Power Company

CC Mr. Stephen Benda – GPC; Mr. Brian Goldsmith, Mr. Nortey Yeboah, Mr. Damon Woodson – SCS

FROM Fernando Famania, PEng., PMP, Brian Johnson, PE, Greg Hebeler, PE; Lizmarie Steel, PE **EMAIL lizmarie.steel@wsp.com**

PHASE 2 & 3 LABORATORY BENCH SCALE TESTING FOR PLANT MCDONOUGH CCR UNIT AP-1 ADVANCED ENGINEERING METHOD PERIMETER SUBSURFACE BARRIER WALL

1.0 BACKGROUND

WSP USA Inc. (WSP) was retained by Georgia Power Company (Georgia Power) for the assessment of advanced engineering methods (AEM) and advanced engineering technologies (AET) to supplement the ongoing closure of the Plant McDonough-Atkinson (Plant McDonough or Plant) CCR Unit AP-1 (AP-1).

In 2021, WSP (as Golder) completed the conceptual design for the selected AEMs and Stormwater Improvements, as presented in the Closure Drawings as part of the Plant McDonough CCR Unit AP-1 Permit Application – Part A Section 9. This conceptual design included Phase 1 laboratory testing, which included bench scale testing of potential soil/additive mixes for the designed fully encompassing perimeter barrier wall around AP-1. Georgia Environmental Protection Division (GA EPD) later provided comments on the AP-1 permit submittal documents asking for laboratory verification demonstration of test mixes meeting the target hydraulic conductivity (a.k.a., permeability) and strength Performance Requirements prior to permit issuance. As a result, additional laboratory testing was completed successfully producing samples which exceed the hydraulic conductivity and strength Performance Requirements of the design submitted in the permit application to EPD, using materials previously demonstrated to meet similar requirements at other sites.

WSP recently completed a (Phase 2 & 3) supplemental bench scale testing program that consisted of mixing site soils with different percentages of bentonite, Portland cement, and Ground Granulated Blast Furnace Slag (GGBFS) and water to evaluate potential mixes to achieve the Performance Requirements. The latest phase of supplemental mixes (Phase 3) included the addition of GGBFS to the mixes to further reduce the permeability of the mixes. GGBFS has been shown in other mixes to consistently produce permeabilities ≤ 1×10^{-7} cm/sec at many sites largely independent of site soil characteristics.

The testing described herein demonstrates that the proposed barrier wall can be constructed by mixing specific percentages of site soils with additives to produce an engineered mixture consisting of tap water, bentonite, and common construction industry binders (i.e., Portland cement and GGBFS), to produce a hydraulic barrier wall capable of meeting the design Performance Requirements established in the CQA plan in the proposed permit application. The Performance Requirements established for the barrier wall include a hydraulic conductivity of less than or equal to (≤) 1 x 10^{-7} centimeters/second (cm/sec) and an Unconfined Compressive Strength (UCS) of greater than or equal to (≥) 10 pounds per square inch (psi).

2.0 SCOPE OF WORK

The objectives of the bench scale verification program were to demonstrate that the Performance Requirements, as determined through the engineering design, can be achieved; and provide one or more preliminary mix designs that meet the Performance Requirements outlined in the barrier wall design documents and herein.

The binder used for Phase 3 was a lime-based Portland Cement (PC-L) combined with GGBFS. The ratio of GGBFS to PC-L (GGBFS:PC-L) is a key factor in the material's ability to achieve specific performance requirements. The selected binder was a manufactured pre-mixed material consisting of 75% GGBFS and 25% PC-L with a commercial name of TerraFlow 75, a sustainable alternative to standard Portland cement.

Various percentages of mix materials (e.g., site soils/water/Portland cement/bentonite/GGBFS) were created in order to achieve appropriate consistency, and to align with the design installation techniques of in-situ soil/additive mixing by use of a cutter-head, jet grouting, and/or other similar in-situ mixing techniques. Mixtures were prepared and cured in a controlled environment and consisted of combining a given mass of soil with a specified amount of bentonite in slurry form (bentonite to water ratio of 10%) hydrated for a minimum of 12 hours. Once the soil and the bentonite slurry were thoroughly mixed, the binder (either Portland cement or Terraflow75) was added in a slurry form at a Water to Cement ratio (W/C) of 0.6. The density of the bentonite and cement slurry was tested before being added to the soil. The density was verified using a mud balance device as per ASTM D4380 – Standard Test Method for Determining Density of Construction Slurries.

Samples were prepared and tested for hydraulic conductivity and UCS.

3.0 SITE SOIL CHARACTERIZATION

All site soil samples were characterized prior to being shipped to the mix testing laboratory for this bench scale program for verification of consistent gradation. All soil samples were then combined to create one representative soil type for use in all mix studies. The natural water content of the soil used for calculating the proportions of each slurry was between 8% and 10%.

The representative soil was evaluated for particle size distribution, with 3.9% classified as gravel, 52.8% as sand and 43.3% as fines (particles passing a #200 sieve / smaller than 75 microns). The particle size distribution report of the representative soil was tested in accordance with ASTM C136 – Particle Size Distribution. Report is included in Attachment 1.

4.0 BENCH SCALE MIX PREPARATION

The grout character and proportions are primary drivers that increase strength and reduce permeability at small scales in soil-grout mixtures, are assessed during bench-scale testing. Homogeneity is a driver for larger scale variability and driven by means and methods employed during construction. In bench-scale testing small batches of soil-grout are made to form samples for testing, the goal is to assess soil-grout mixtures that are homogeneous such that variability between samples is minimized resulting in a need to test less samples. Bench-scale testing demonstrates proof-of-concept, while Contractor testing of their mix is performed with equipment, techniques and available reagents that simulate the means and methods to be employed and often produce more variable results, as seen during construction. It is generally expected that field-scale mixes will have higher dosing rates of grouts/reagents to account for this method driven variability. Thus, WSP recommends that bench-scale testing, as presented here, is done first to prove the concept works; then the Contractor performs testing with their means and methods and available reagents to demonstrate they work together; and finally a field-scale pilot is performed to assess variability due to scale effects and site material variability. This testing is then followed by full-scale construction. The field-scale pilot (which includes intensive sampling and testing) will identify any final modifications needed to the proposed means, methods, reagents, and mix proportions to meet the required combination of strength and permeability.

The laboratory mixing program included the following steps:

- Collection and processing of individual representative soil samples from the site.
- Procurement of additives (i.e., GGBFS/PC-L, PC, and B), from manufacturer.
- Delivery of the representative soil to the WSP mixing/testing facility in Burnaby, BC, Canada for completion of the bench scale program.
- Combining soil samples and mixing into one soil sample, followed by particle size distribution analysis of the combined soil.
- Planning and discussion for number of mixes, mix designs, and testing program.
- Weighing out a specific mass of representative soil per mix.
- Mixing and adding bentonite slurry to the soil. The bentonite slurry was prepared at 10% Bentonite to Water ratio and hydrated for a minimum of 12 hours before adding it to the soil and mixing thoroughly using a hand held paddle mixer.
- Mixing binder(s) with water to form “binder slurry” based on the mix design. The binder (Cement or TerraFlow75) was prepared at a Water to Cement ratio (W/C) of 0.6. Once the binder slurry was mixed for about five minutes, it was weighed and added to the bucket with soil and bentonite slurry.
- Combining the binder slurry with the representative soil, by weight and mechanical mixing together in a 5-gallon bucket with an electric drill with a mud mixer attachment. The slurry was mixed for approximately five minutes at the maximum paddle mixing speed.
- Casting the soil-cement mixtures in cylinders, curing in a humid environment, and extruding each sample for Unconfined Compressive Strength (UCS) and hydraulic conductivity testing.

The bench scale program was completed in general accordance with Standard Practice ASTM D1632 – “Making and Curing Soil-Cement and Flexure Test Specimens in the Laboratory”. The size of the plastic cylinders and the sequence for adding the different materials based on project experience differed from this document based on practicability and project experience. This bench scale program followed the same sequence as the field using larger equipment. It's important to note that ASTM D1632 is not intended to represent or replace the standard of care by which the adequacy of a given professional service must be judged, nor should this document be applied without consideration of a project’s many unique aspects. As mentioned above, the bentonite slurry was mixed into the soil first, then the cement slurry was added to the same bucket based on the required weight replicating the sequence of material being added to the in-situ soil in the field. The mixture was blended using the same drill and mixing attachment as the one used for cement slurry until the material appeared to be well mixed, with enough working consistency and without the presence of unmixed material. Mixes 1 to 4 required additional water to produce a material which was adequately mixed and workable.

Once the mix was visually uniform in color and texture it was cast in 3-inch-diameter by 6-inch-high cylindrical plastic molds for curing. Soil-cement mixes were placed in the molds using a concrete scoop and a putty knife. The soil-cement mix was scooped into the molds in three to four lifts based on the consistency of the material. Rodding and tapping the mold against a firm surface after each lift were completed to ensure air bubbles and voids were not present in the specimen, as required by ASTM D1632. Once the cylinders were filled, the top of each specimen was smoothed with a putty knife.

Cylinders were labelled on each lid to identify date cast, mix design number, and a letter in alphabetical order to have a unique ID per cylinder. A lid was placed on each specimen to avoid loss in moisture and the specimens were placed inside a curing room at a temperature of 69.8 – 77.0 degrees (°F) and relative humidity at 95% or higher. This temperature is based on Standard ASTM C192M-18 Making and Curing Concrete Test Specimens in the Laboratory.

The specimens were extruded from the cylinders and inspected for damage or uneven surfaces. If the ends of the specimen were uneven the surface of each end was trimmed to provide a smooth, flat surface perpendicular to the axis of the specimen for the purpose of even load distribution during testing.

All UCS tests were conducted using a Model Sigma-1 UCS machine with a calibrated load cell capacity of 10,000 pounds. Each specimen was tested according to ASTM D2166. Following the UCS test, a photograph was taken of the specimen post failure. A sample of the crushed cylinder was taken to test the water content of the specimens.

5.0 PROPOSED MIX DESIGN & RESULTS

5.1 Materials

The Portland cement binder used for Phase 2 was Type I General Use (Type GU), which is the Canadian equivalent to Type I cement in the United States (US). Phase 3 used a 25% lime-based Portland Cement with Lime (PC-L) combined with 75% GGBFS (manufactured by Lafarge Canada, commercially known as TerraFlow 75), which could be manufactured in the US using similar materials or purchased directly for the project.

The bentonite used for the bench scale program was ACC Premium Gel®, manufactured by American Colloid Company (ACC), Premium Gel is a 200-mesh, high purity, natural sodium bentonite that meets API 13A Section 9 specifications. The water used for the bench scale was tap water. The technical specifications for Type GU Cement, TerraFlow 75, and ACC Premium Gel bentonite are included in Attachment 2.

Water used in the mixes was tap water which provides consistency across the various mix trials and also mimics the typical use of municipal water in commercial in-situ mixing operations. Final checks and proof of compatibility with site water chemistry will be evaluated as part of (or prior to) the wall test section demonstration required in the project specifications to be performed and demonstrate compliance with performance requirements ahead of full scale production wall construction.

5.2 Mix Parameters & Testing

A total of 12 mixes were prepared for Phase 2 and Phase 3 of the laboratory verification program. Phase 2 included Mix 1 to Mix 6 and Phase 3 included Mix 7 to Mix 12. The table below shows the percentages of bentonite and binder added for each mix based on the dry mass of soil. The mass of soil was estimated to prepare enough specimens for the testing required.

Testing was completed based on the following ASTM standards:

- Permeability testing followed ASTM D5084: Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter, and
- UCS testing followed ASTM D2166: Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

Table 1: Phase 2 – UCS and Hydraulic Conductivity Results

Mix No.	Binder Type	Bentonite (% of dry mass)	Bentonite to Water Ratio	Terraflow 75 (% of dry mass)	W/C Ratio	Extra Water (% of dry mass)	UCS (psi)	Hydraulic Conductivity (k_{20} cm/s)
1	Type GU	2	10%	5	0.6	10	78 (7d)	3.67E-07
2	Type GU	2	10%	7	0.6	10	114 (7d)	N/A
3	Type GU	4	10%	5	0.6	3	46 (7d)	N/A
4	Type GU	4	10%	7	0.6	3	66 (7d)	3.27E-07(22d) 4.41E-07(53d)
5	Type GU	6	10%	5	0.6	0	23 (7d)	4.66E-07(24d)
6	Type GU	6	10%	7	0.6	0	39 (7d)	2.60E-07(23d) 2.19E-07(53d)

Table 2: Phase 3 - UCS and Hydraulic Conductivity Results

Mix No.	Binder Type	Bentonite (% of dry mass)	Bentonite to Water Ratio	Terraflow75 (% of dry mass)	W/C Ratio	Extra Water (% of dry mass)	UCS (psi) Curing Time Days (d)	Hydraulic Conductivity (k_{20} cm/s)
7	Terraflow 75	3	10%	5	0.6	0	22 (14d) 40 (28d)	2.19E-07(14d) 1.00E-07(32d)
8	Terraflow 75	3	10%	10	0.6	0	108 (14d) 157 (28d)	3.20E-08(14d) 2.53E-08(28d)
9	Terraflow 75	3	10%	15	0.6	0	232 (14d) 344 (28d)	3.44E-10(37d) 5.95E-10(42d)
10	Terraflow 75	5	10%	5	0.6	0	11 (14d) 19 (28d)	2.52E-07(28d) 2.39E-07(42d)
11	Terraflow 75	5	10%	10	0.6	0	45 (14d) 61 (28d)	3.17E-07(21d) 9.85E-08(46d)

Mix No.	Binder Type	Bentonite (% of dry mass)	Bentonite to Water Ratio	Terraflow75 (% of dry mass)	W/C Ratio	Extra Water (% of dry mass)	UCS (psi) Curing Time Days (d)	Hydraulic Conductivity (k_{20} cm/s)
12	Terraflow 75	5	10%	15	0.6	0	96 (14d) 153 (28d)	5.73E-08(23d) 9.46E-09(52d)

Note: The bentonite slurry was prepared at a bentonite to water ratio of 10% for all mixes with minimum hydration time of 12 hours.

The UCS and permeability test reports can be found in Attachment 3. As seen in the green highlighted results in Table 2, the Phase 3 round of mix trials incorporating GGBFS were successful at demonstrating that a number of mix design combinations can be made that meet or exceed the design requirements as outlined in the CQA Plan.

6.0 CLOSURE

This document summarizes the strength and permeability test results of a supplemental range of mix designs. The presented bench scale testing demonstrated that achievement of the wall design strength and permeability requirements are readily achievable when using mixes that incorporate GGBFS as a mix reagent. These bench scale results are applicable to a range of construction mixing and grouting techniques such as soil mixing, jet grouting and similar soil/grout mixing techniques. As such, wall construction using one of the successful mixes presented in this memo is expected to produce a soil/grout wall that meets the performance requirements of the design as outlined in the CQA Plan in Part A of the Permit Application.

WSP USA Inc.

Gregory L. Hebeler, PE

Technical Fellow, Geotechnical Engineer

Lizmarie Steel, PE

Lead Consultant, Environmental Engineer

Attachments:

Attachment 1: Particle Size Distribution (Representative Site Soil)
Attachment 2: Materials Technical Specifications
Attachment 3: Testing Reports (UCS & Permeability)

ATTACHMENT 1

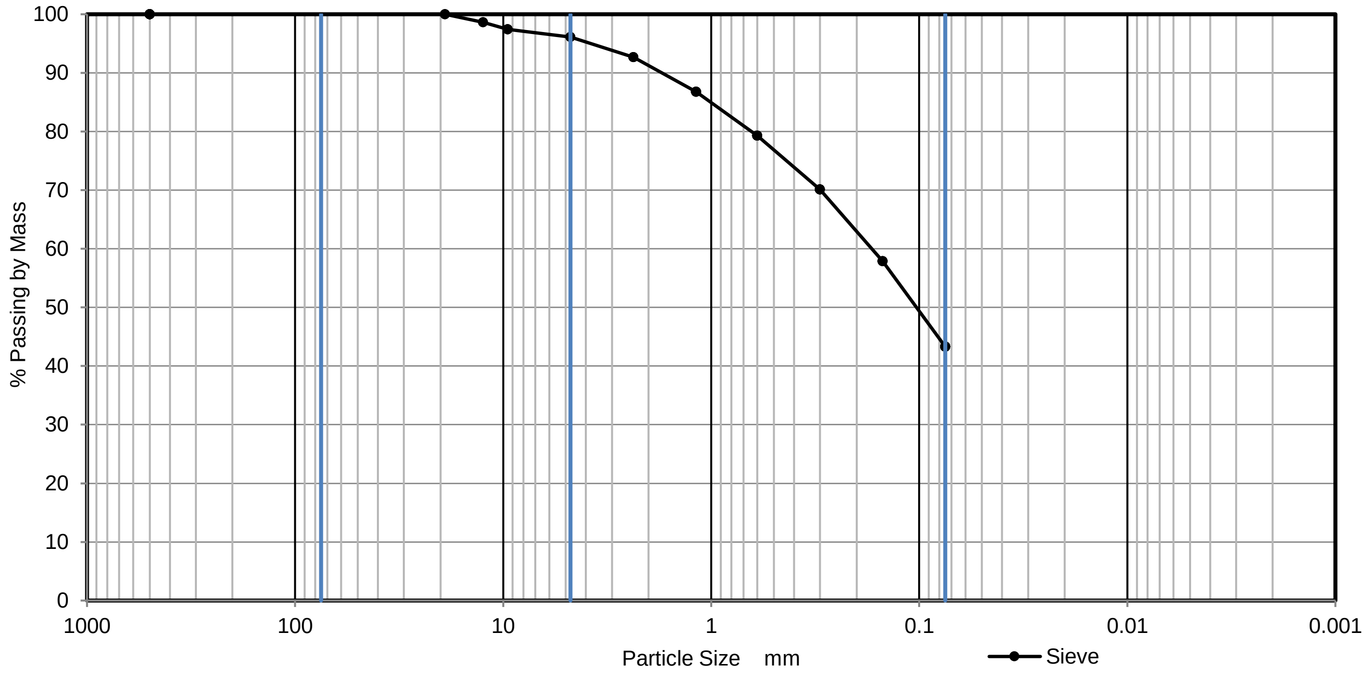
Particle Size Distribution (Representative Site Soil)

PARTICLE SIZE DISTRIBUTION
ASTM C136

Test Request #	B23-021	Project Number:	23591773-2000-2300
Client:	Golder Associates USA Inc.	Project Location:	Cobb County, Georgia, USA
Project Name:	SCS/PLT McDonough AP1	Sample Location:	Soil Sample (Combined)
Source:		Sample No.:	1
Soil Description:		Type:	GS
		Depth (m):	0.00 -

Specimen Reference	NA	Specimen Depth (m):	NA	Date of Test	18 Jan 2023
Specimen Description	NA				

Grain Size Distribution (%)			3.9			52.8		43.3
BOULDERS	COBBLES	GRAVEL		SAND				FINES (Silt, Clay)
		Coarse	Fine	Coarse	Medium	Fine		



Sieve			Material Specification	
Sieve No.	Particle Size mm	% Passing	Lower	Upper
3/4"	19	100.0		
1/2"	12.5	98.6		
3/8"	9.5	97.4		
#4	4.75	96.1		
#8	2.36	92.7		
#16	1.18	86.8		
#30	0.6	79.3		
#50	0.3	70.1		
#100	0.15	57.9		
#200	0.075	43.3		
D60	D30	D10	Cu	Cc
0.17				

Notes:

Disclaimer:

The laboratory testing services reported herein have been performed in accordance with the terms of a contract with WSP's client, and with the recognized standards indicated in this report, or local industry practice. This laboratory testing services report is for the sole use of WSP's client, relates only to the sample(s) tested and does not represent any (actual or implied) interpretation or opinion regarding specification compliance or materials suitability for any specific purpose.

Tested by: VNogra **Date:** 18 Jan 2023 **Checked by:** VNogra **Date:** 23 Jan 2023 **Reviewed by:** SJohn **Date:** 23 Jan 2023

WSP Canada Inc.
Unit 300 - 3811 North Fraser Way, Burnaby, British Columbia, V5J 5J2, Canada

[+1] 604 412 6899

ATTACHMENT 2

Materials Technical Specifications

TECHNICAL DATA

PREMIUM GEL®

API GRADE BENTONITE

NSF®

Certified to NSF/ANSI/CAN 60

DESCRIPTION

PREMIUM GEL is a 200-mesh, high-purity, natural sodium bentonite that meets API 13A Section 9 specifications. PREMIUM GEL is designed to produce viscous slurries in freshwater when mixed at dosages of roughly 4% to 8% solids by weight. PREMIUM GEL is certified to NSF/ANSI/CAN Standard 60, Drinking Water Treatment Chemicals - Health Effects.

RECOMMENDED USE

PREMIUM GEL can be used as a stand-alone drilling fluid or used as the base for a higher-yield fluid with additional extending polymer. Extension up to 150 bbl/ton is possible. PREMIUM GEL can serve as a filtrate-reducing material for use with cement-bentonite formulations. PREMIUM GEL can also be used as a seal for earthen structures, slurry trenching, tunnel boring, and foundation drilling.

PURITY

Natural Wyoming bentonite consisting primarily of sodium montmorillonite with trace amounts of quartz, feldspars, and calcite.

CHARACTERISTICS

- Cools and lubricates drill-bits and cutting surfaces
- Good dispersibility and rapid viscosity development
- Improves rate of penetration versus water alone since cuttings are efficiently removed
- Reduces filtrate loss to the formation and produces thin, tight filter cake for improved borehole stability

MIXING AND APPLICATION

Mixing ratios are based on the use of freshwater and water purity will impact bentonite performance. For best results, acidic and hard makeup water should be pretreated with SODA ASH to a pH of 8.5-9.5. Add PREMIUM GEL slowly through jet/hopper mixer.

PACKAGING

~50 lb (~22.7 kg) bag, 48 or 70 per pallet, ~100 lb (~45.4 kg) 35 per pallet, ~1 ton supersacks, or bulk. All pallets are plastic-wrapped.

SLURRY PROPERTIES - 6.04% SOLIDS		
Property	Typical Value	Specification/Procedure
Viscosity FANN 600 rpm	40 cps	30 cps Min - API 13A Section 9
Yield - 42 gal bbl of 15 cps slurry/ton	90 - 120 bbl/tn	90 Min - API 13A Section 9
Marsh Funnel, seconds/quart	42 seconds	ACC TP-1014
Apparent Viscosity (AV)	15 to 25 cps	ACC TP-2005
Plastic Viscosity (PV)	9 - 10	ACC TP-2005
Yield Point, lb/100 ft^2	15 - 25 lb/100 ft^2	ACC TP-2005
Filtrate, 30 minutes @ 100 psi, mi	13 - 15 ml	15 ml Max - API 13A Section 9
Filter Cake, in	3/32 in	N/A
pH	9.03	ACC TP-1018
GENERAL PROPERTIES		
Property	Typical Value	Specification/Procedure
Moisture %	7.4%	ACC TP-2006
Free Swell	28	ASTM D-5890
Plate Water Absorption	622.80%	ASTM E946-92
Specific Gravity	2.5	Generally Recognized
Bulk Density Non-compacted	53 lbs/ft^3	ACC TP-1005
Bulk Density Compacted	72 lbs/ft^3	ACC TP-1005
Grit % (<75 micron)	3.2%	4.0% Max - API 13A Section 9
Particle Sizing	70% Min passing #200 mesh sieve	ACC TP-1015

MINERALS TECHNOLOGIES

AMERICAN COLLOID COMPANY

PORTLAND CEMENT

PRODUCT NO. 1124-31, -47, -94

DIVISION 3

Cement
03 05 00

PRODUCT DESCRIPTION

QUIKRETE® Portland Cement is a high quality Portland cement meeting ASTM C 150 Type I.

PRODUCT USE

QUIKRETE® Portland Cement is used for making high strength repair mortars, concrete and for any other applications requiring Type I Portland cement. In many locations the product also meets ASTM C 150 Type II. Consult your supplying plant to confirm compliance with ASTM C 150 Type II.

SIZES

• QUIKRETE® Portland Cement
- 31 lb (14 kg) bags
- 47 lb (21.3 kg) bags
- 94 lb (42.6 kg) bags
- 40 kg (88 lb) bags
- 42 kg (93 lb) bags

YIELD

• Yield depends on application. For concrete mixes: Five to six 94 lb (42.6 kg) bags of QUIKRETE® Portland Cement is typically used with appropriate proportions of sand and gravel to produce 1 cu. yd. (0.8 m3) of concrete.

TECHNICAL DATA

QUIKRETE® Portland Cement complies with ASTM C 150 Type I and in many locations also complies with ASTM C 150 Type II. The product is used in a variety of construction materials. Typical mix designs for some applications are listed below:

Concrete Mix
1 Part QUIKRETE® Portland Cement
2 Parts QUIKRETE® All-Purpose Sand (ASTM C-33)
3 Parts QUIKRETE® All-Purpose Gravel (ASTM C-33)

Mortar Mix (Type S, per ASTM C-270)
1 Part QUIKRETE® Portland Cement
1/2 Part QUIKRETE® Hydrated Lime -Type S
3-1/2 to 4-1/2 Parts QUIKRETE® Masonry Sand (ASTM C-144)

Scratch and Brown Coat Stucco Mix (per ASTM C-926)
1 Part QUIKRETE® Portland Cement
1/2 Part QUIKRETE® Hydrated Lime (Type S)
4-1/2 to 6 Parts QUIKRETE® Washed Plaster Sand (ASTM C-897)

INSTALLATION

Installation methods are specific for each type of product.

PRECAUTIONS

The following points apply to all products made from Portland cement:
• Protect from freezing for at least 24-48 hr.
• Use the minimum amount of water necessary to achieve the desired consistency. Adding too much water will weaken the product.
• Keep the product damp for several days to obtain proper curing.

WARRANTY

The QUIKRETE® Companies warrant this product to be of merchantable quality when used or applied in accordance with the instructions herein. The product is not warranted as suitable for any purpose or use other than the general purpose for which it is intended. Liability under this warranty is limited to the replacement of its product (as purchased) found to be defective, or at the shipping companies' option, to refund the purchase price. In the event of a claim under this warranty, notice must be given to The QUIKRETE® Companies in writing. This limited warranty is issued and accepted in lieu of all other express warranties and expressly excludes liability for consequential damages.

The QUIKRETE® Companies
One Securities Centre
3490 Piedmont Rd., NE, Suite 1300, Atlanta, GA 30305
(404) 634-9100 • Fax: (404) 842-1425

ATTACHMENT 3

Testing Reports (UCS & Permeability)

Standard Test Methods for Unconfined Compression Strength of Cohesive Soil

ASTM D2166

Project No.:	23591773-2000-2300	**Borehole:**	M7-3B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	A
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Type:	Soil-cement cylinder	**Classification:**	N/A
Sample Description:	Soil-cement cylinder		
Remarks:	N/A		

Sample Properties

	Initial	Final
Height (cm)	15.26	14.72
Diameter (cm)	7.61	7.75
Area (cm^2)	45.49	47.15
Volume (cm^3)	694	694
Wet weight (g)	1243.2	1242.0
Dry weight (g)	906.8	906.8
Water content (%)	37.1	37.0
Wet density, ρ_{wet} (kg/m^3)	1791	1790
Dry density, ρ_{dry} (kg/m^3)	1307	1307
Saturation (%)	94	94
Void ratio, e (-)	1.07	1.07
Specific Gravity (assumed)	2.70	
Sensitivity	N/A	

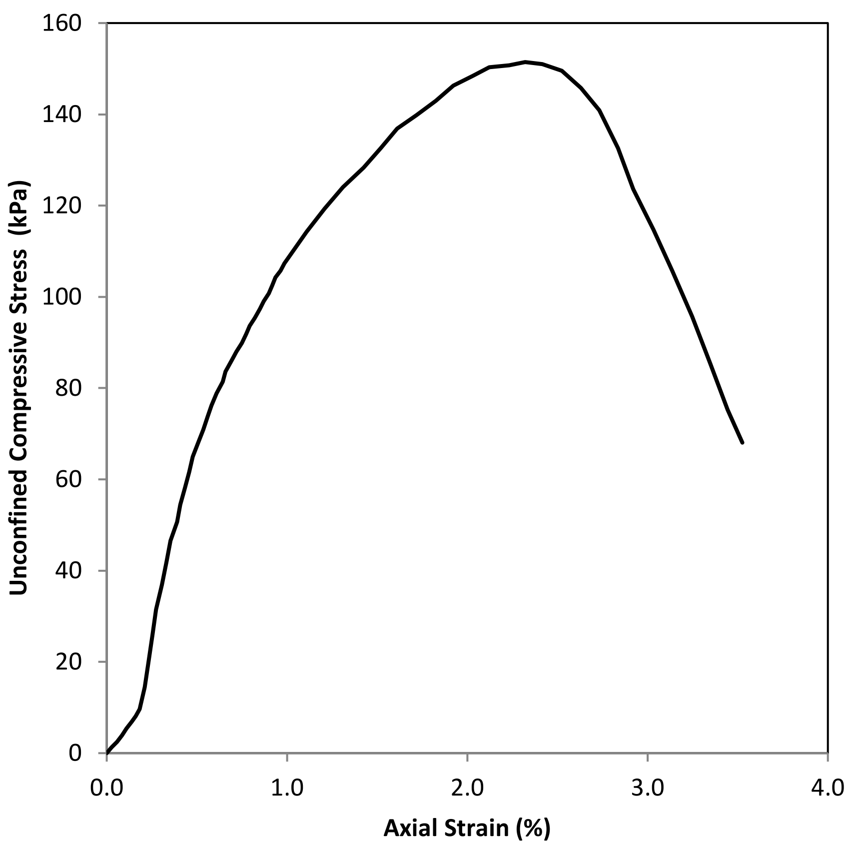
Equipment

Machine	Load Frame 4
Load Cell	400469
Axial DCDT	LP-174
Feed Rate (%/min)	1.00

Test Results

Unconfined Compressive Stress (kPa)	**151**
Strain at Failure, ε_f (%)	**2.32**
Shear Strength (kPa)	76
Initial Height/Diameter Ratio	2.00

Test Comments: N/A



T. Madera	April 3, 2023	D. Lim	April 6, 2023
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.wsp.com

Standard Test Methods for Unconfined Compression Strength of Cohesive Soil

ASTM D2166

Project No.:	23591773-2000-2300	**Borehole:**	M7-3B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	C (28 days)
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Type:	Soil-cement cylinder	**Classification:**	N/A
Sample Description:	Soil-cement cylinder		
Remarks:	N/A		

Sample Properties

	Initial	Final
Height (cm)	15.22	14.62
Diameter (cm)	7.61	7.77
Area (cm^2)	45.54	47.41
Volume (cm^3)	693	693
Wet weight (g)	1246.4	1245.5
Dry weight (g)	927.3	927.3
Water content (%)	34.4	34.3
Wet density, ρ_{wet} (kg/m^3)	1798	1796
Dry density, ρ_{dry} (kg/m^3)	1337	1337
Saturation (%)	91	91
Void ratio, e (-)	1.02	1.02
Specific Gravity (assumed)	2.70	
Sensitivity	N/A	

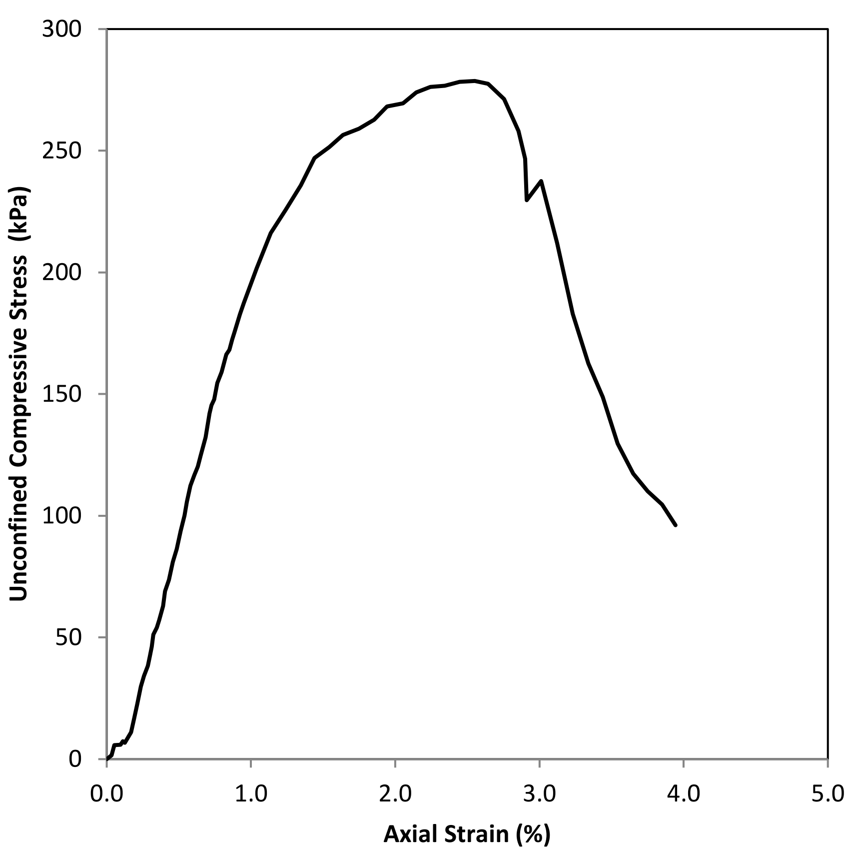
Equipment

Machine	Load Frame
Load Cell	118534
Axial DCDT	LP-267
Feed Rate (%/min)	1.00

Test Results

Unconfined Compressive Stress (kPa)	**279**
Strain at Failure, ε_f (%)	**2.55**
Shear Strength (kPa)	139
Initial Height/Diameter Ratio	2.00

Test Comments: N/A



T. Madera	April 17, 2023	D. Lim	April 19, 2023
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.wsp.com

Standard Test Methods for Unconfined Compression Strength of Cohesive Soil

ASTM D2166

Project No.:	23591773-2000-2300	**Borehole:**	M8-3B-10C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	A
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Type:	Soil-cement cylinder	**Classification:**	N/A
Sample Description:	Soil-cement cylinder		
Remarks:	N/A		

Sample Properties

	Initial	Final
Height (cm)	15.25	14.83
Diameter (cm)	7.63	7.73
Area (cm^2)	45.68	46.97
Volume (cm^3)	697	697
Wet weight (g)	1251.1	1250.2
Dry weight (g)	964.9	964.9
Water content (%)	29.7	29.6
Wet density, ρ_{wet} (kg/m^3)	1796	1795
Dry density, ρ_{dry} (kg/m^3)	1385	1385
Saturation (%)	84	84
Void ratio, e (-)	0.95	0.95
Specific Gravity (assumed)	2.70	
Sensitivity	N/A	

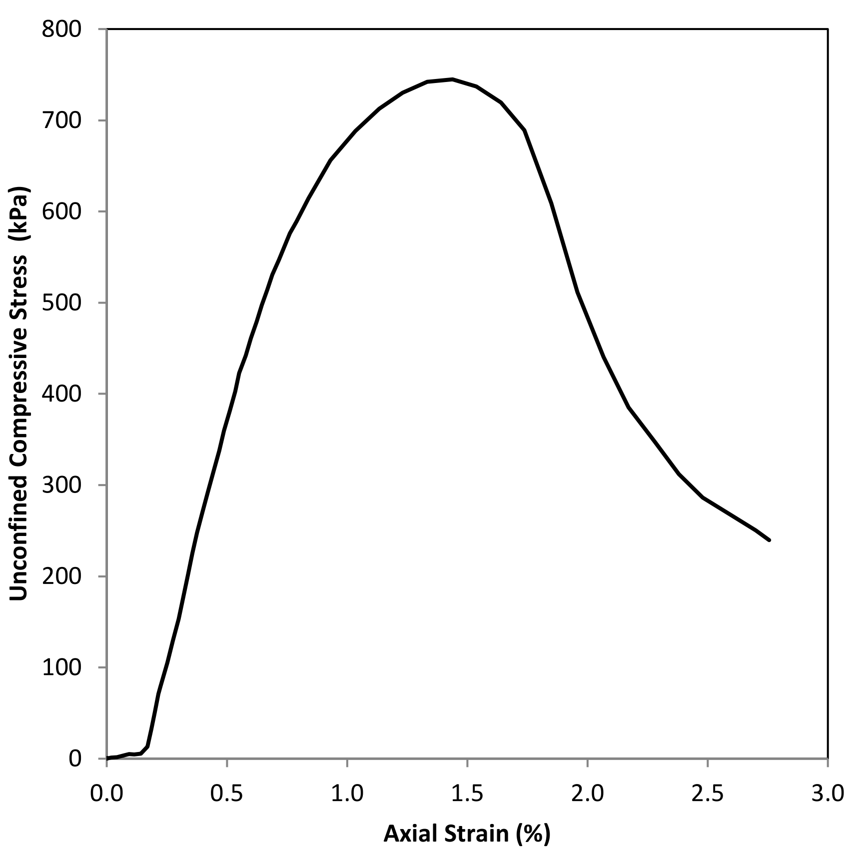
Equipment

Machine	Load Frame
Load Cell	118536
Axial DCDT	LP-946
Feed Rate (%/min)	1.00

Test Results

Unconfined Compressive Stress (kPa)	**745**
Strain at Failure, ε_f (%)	**1.44**
Shear Strength (kPa)	372
Initial Height/Diameter Ratio	2.00

Test Comments: N/A



T. Madera	April 4, 2023	D. Lim	April 10, 2023
TESTED BY	DATE	CHECKED BY	DATE

Standard Test Methods for Unconfined Compression Strength of Cohesive Soil
ASTM D2166

Project No.:	23591773-2000-2300	**Borehole:**	M9-3B-15C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	A
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Type:	Soil-cement cylinder	**Classification:**	N/A
Sample Description:	Soil-cement cylinder		
Remarks:	N/A		

Sample Properties

	Initial	Final
Height (cm)	15.28	14.83
Diameter (cm)	7.63	7.75
Area (cm^2)	45.77	47.15
Volume (cm^3)	699	699
Wet weight (g)	1247.8	1246.9
Dry weight (g)	974.7	974.7
Water content (%)	28.0	27.9
Wet density, ρ_{wet} (kg/m^3)	1784	1784
Dry density, ρ_{dry} (kg/m^3)	1394	1394
Saturation (%)	81	80
Void ratio, e (-)	0.94	0.94
Specific Gravity (assumed)	2.70	
Sensitivity	N/A	

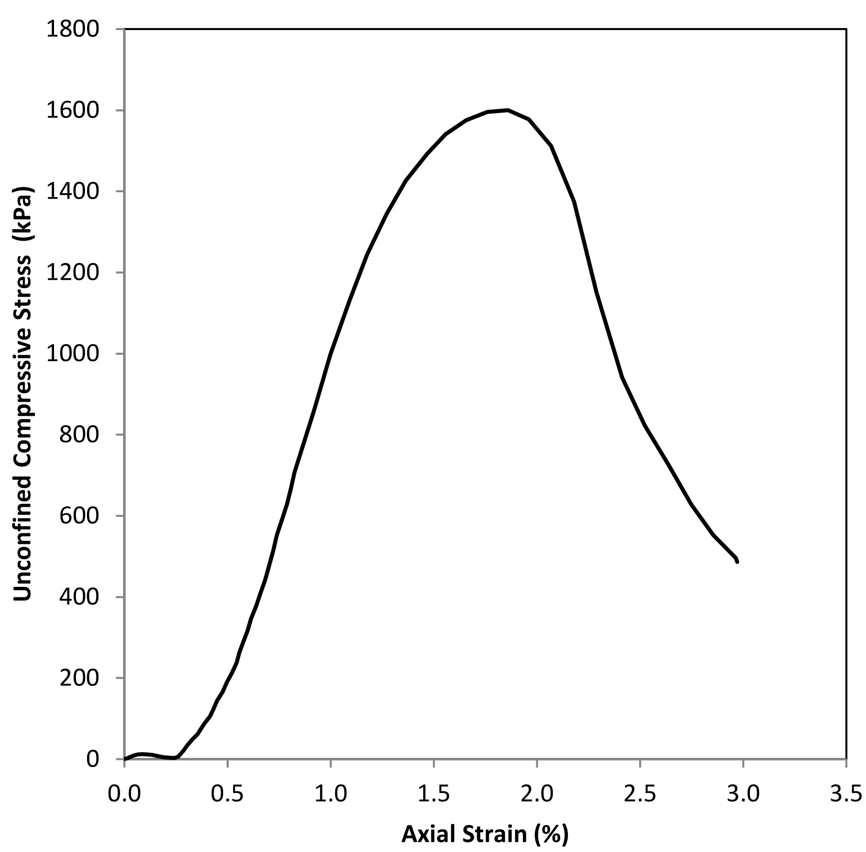
Equipment

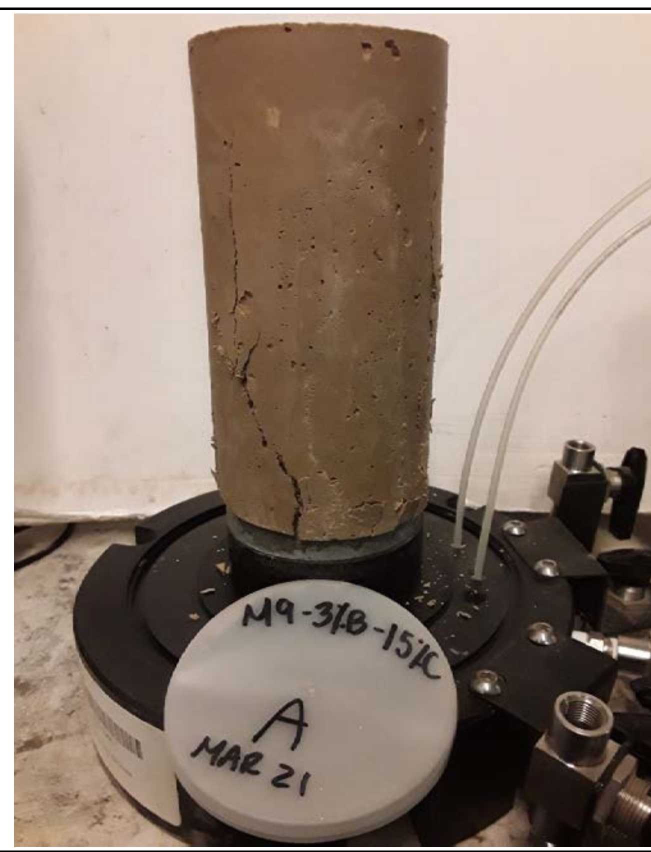
Machine	Load Frame
Load Cell	118536
Axial DCDT	LP-946
Feed Rate (%/min)	1.00

Test Results

Unconfined Compressive Stress (kPa)	**1600**
Strain at Failure, ε_f (%)	**1.86**
Shear Strength (kPa)	800
Initial Height/Diameter Ratio	2.00

Test Comments: N/A





T. Madera	April 4, 2023	D. Lim	April 11, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M1-2B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.67	7.68
Length (cm)	15.15	15.16
Area (cm^2)	46.23	46.29
Volume (cm^3)	700.27	701.97
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	965.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	0.00
B_{value}	0.98
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1251.74	1277.86
Dry Weight (g)	965.10	965.10
w (%)	29.7	32.4
ρ_{dry} (kg/m^3)	1378	1375
S (%)	91.2	99.0
e	0.81	0.82
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-14-23	07:00	0	305.4	--	--	0.0	0.0	22	-	--
		9	304.8	305.4	304.8	0.2	0.4	22	20.10	5.69E-07
		31	304.0	305.4	304.0	0.7	0.7	22	20.05	3.86E-07
		59	302.7	305.4	302.7	1.3	1.4	22	19.96	3.92E-07
		134	299.5	305.4	299.5	2.9	3.0	22	19.75	3.79E-07
		171	298.0	305.4	298.0	3.7	3.7	22	19.65	3.73E-07
		248	294.8	305.4	294.8	5.3	5.3	22	19.44	3.71E-07
		320	291.4	305.4	291.4	7.3	6.7	22	19.22	3.82E-07
		332	290.8	305.4	290.8	7.6	7.0	22	19.18	3.84E-07
		393	289.0	305.4	289.0	8.2	8.2	22	19.06	3.65E-07
		471	285.8	305.4	285.8	9.8	9.8	22	18.85	3.66E-07
		518	283.8	305.4	283.8	10.8	10.8	22	18.72	3.68E-07
		555	282.4	305.4	282.4	11.5	11.5	22	18.62	3.67E-07
		651	278.7	305.4	278.7	13.5	13.2	22	18.38	3.66E-07
		874	270.1	305.4	270.1	17.8	17.5	22	17.81	3.66E-07
		1020	264.7	305.4	264.7	20.5	20.2	22	17.46	3.65E-07
		1100	261.7	305.4	261.7	22.0	21.7	22	17.26	3.65E-07
		1155	259.5	305.4	259.5	23.1	22.8	22	17.11	3.67E-07
		1221	257.3	305.4	257.3	24.2	23.9	22	16.97	3.65E-07
						Trial 1 - Average k_{20} (cm/s)				**3.67E-07**

Remarks

Soil cement

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

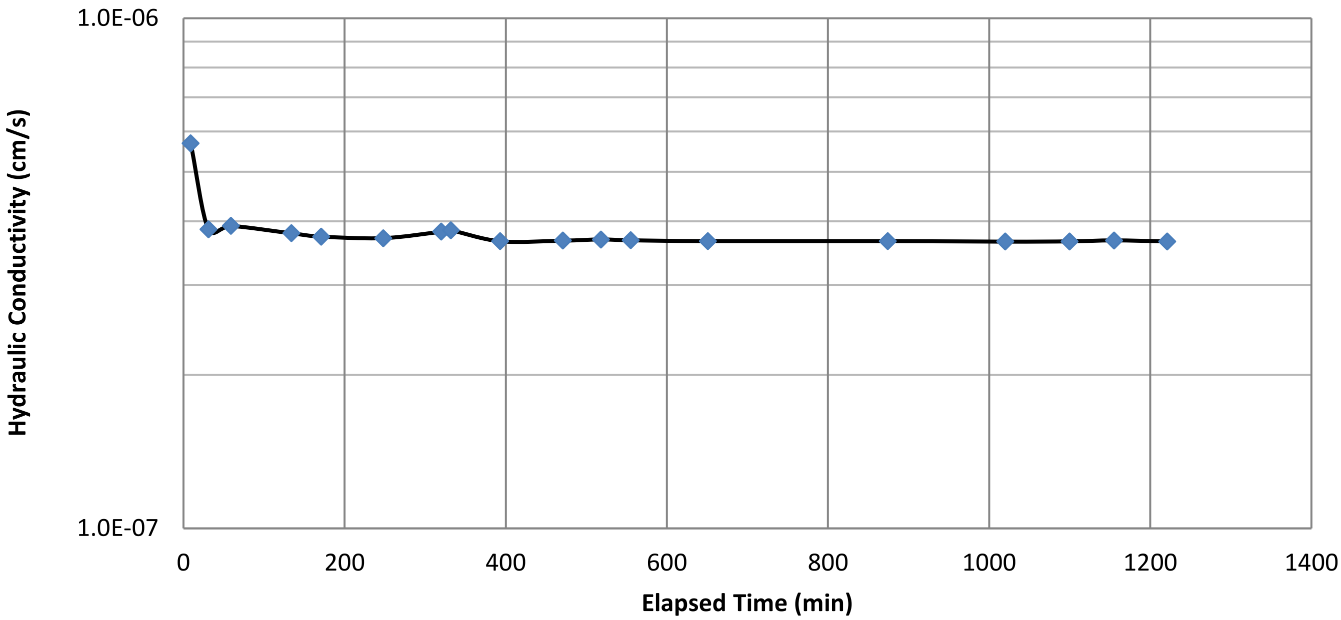
T. Madera	February 14, 2023	D. Lim	February 17, 2023
TESTED BY	DATE	CHECKED BY	DATE

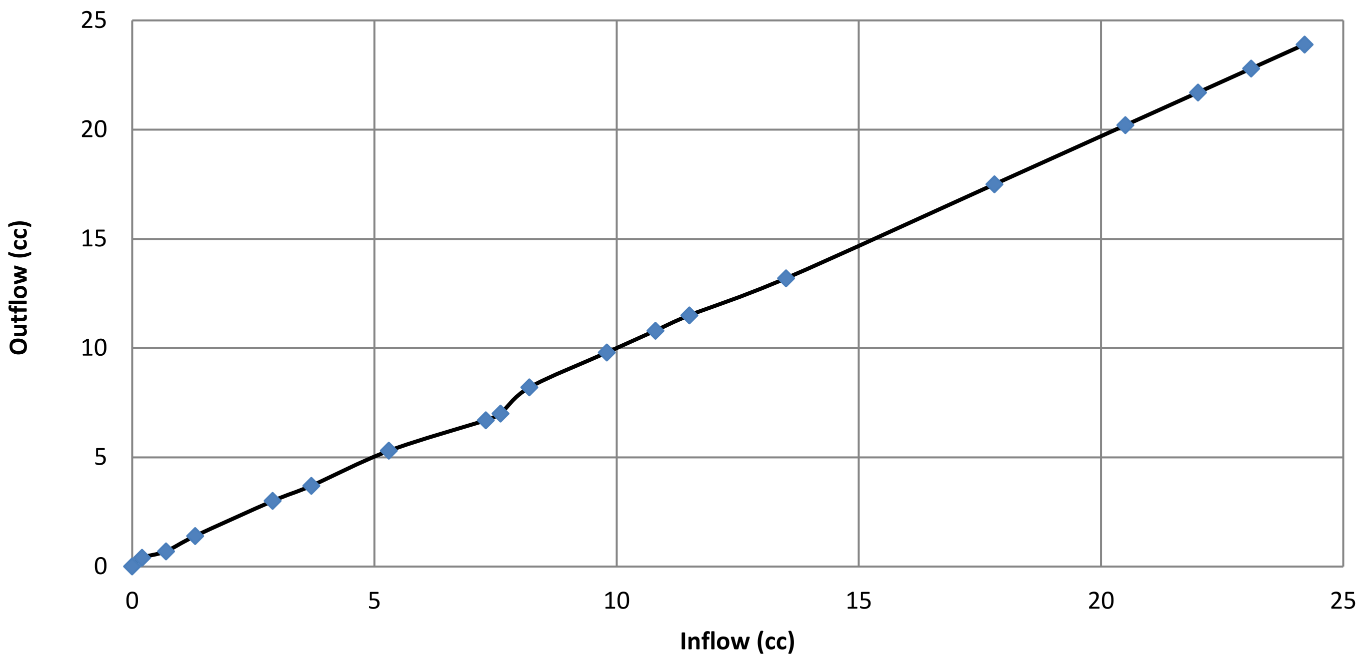
WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M1-2B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	February 14, 2023	D. Lim	February 17, 2023
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.68	7.68
Length (cm)	15.22	15.19
Area (cm^2)	46.28	46.35
Volume (cm^3)	704.47	704.13
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	965.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	-0.87
B_{value}	0.99
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1199.94	1211.00
Dry Weight (g)	856.85	856.85
w (%)	40.0	41.3
ρ_{dry} (kg/m^3)	1216	1217
S (%)	94.8	98.0
e	1.06	1.05
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-7-23	07:00	0	305.6	--	--	0.0	0.0	22	-	--
		8	305.3	305.6	305.3	0.1	0.2	22	20.10	3.20E-07
		54	303.4	305.6	303.4	1.0	1.2	22	19.97	3.48E-07
		215	297.2	305.6	297.2	4.1	4.3	22	19.56	3.38E-07
		309	293.8	305.6	293.8	5.8	6.0	22	19.34	3.32E-07
		359	292.0	305.6	292.0	6.7	6.9	22	19.22	3.30E-07
		414	289.8	305.6	289.8	7.8	8.0	22	19.08	3.34E-07
		484	287.4	305.6	287.4	9.0	9.2	22	18.92	3.30E-07
		537	285.4	305.6	285.4	10.0	10.2	22	18.79	3.32E-07
		1287	260.3	305.6	260.3	22.7	22.6	22	17.13	3.25E-07
		1361	258.1	305.6	258.1	23.8	23.7	22	16.99	3.23E-07
						Trial 1 - Average k_{20} (cm/s)				**3.27E-07**

Remarks

Soil cement

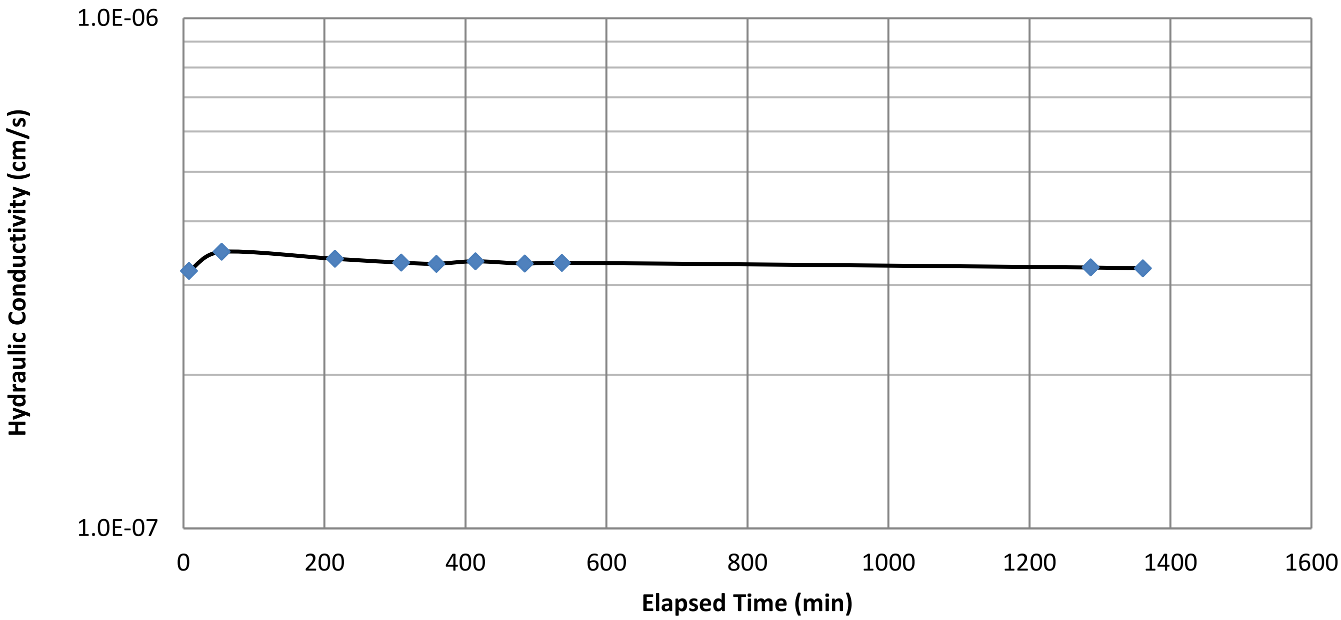
The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

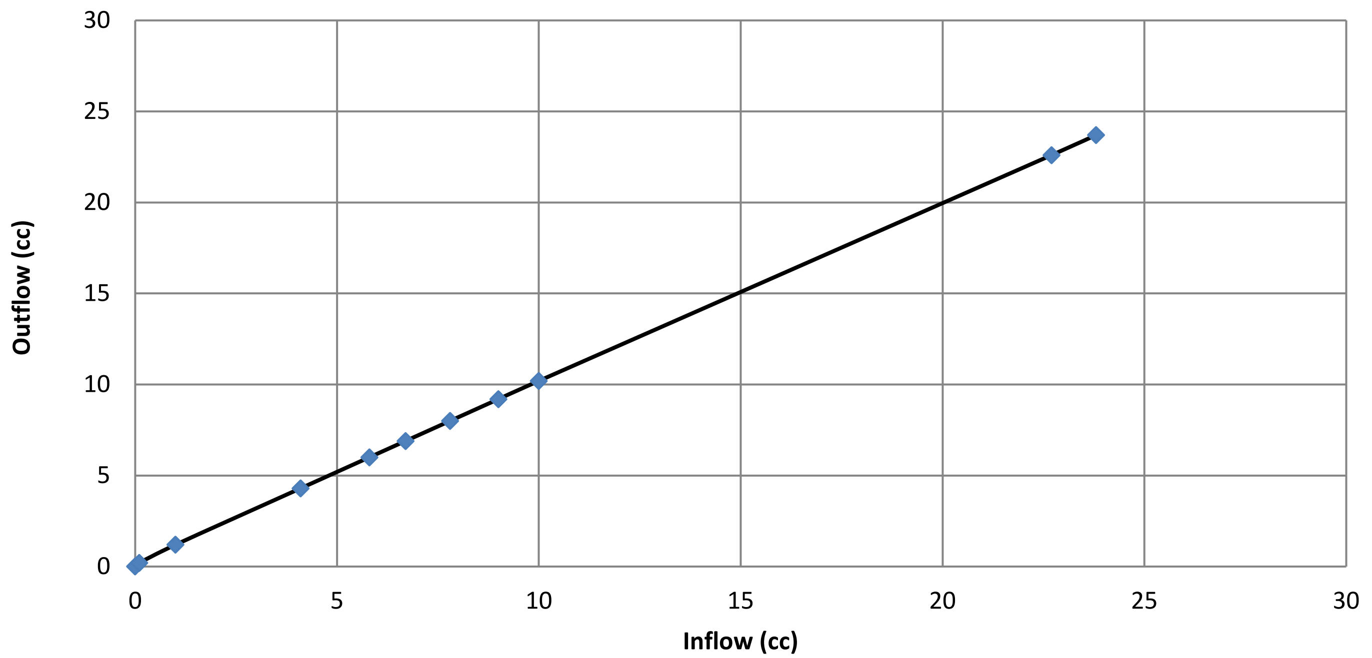
T. Madera	February 7, 2023	D. Lim	February 9, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	February 7, 2023	D. Lim	February 9, 2023
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.68	7.68
Length (cm)	15.22	15.19
Area (cm^2)	46.28	46.35
Volume (cm^3)	704.47	704.13
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	965.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	-0.87
B_{value}	0.99
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1199.94	1211.00
Dry Weight (g)	856.85	856.85
w (%)	40.0	41.3
ρ_{dry} (kg/m^3)	1216	1217
S (%)	94.8	98.0
e	1.06	1.05
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-7-23	07:00	0	305.6	--	--	0.0	0.0	22	-	--
		8	305.3	305.6	305.3	0.1	0.2	22	20.10	3.20E-07
		54	303.4	305.6	303.4	1.0	1.2	22	19.97	3.48E-07
		215	297.2	305.6	297.2	4.1	4.3	22	19.56	3.38E-07
		309	293.8	305.6	293.8	5.8	6.0	22	19.34	3.32E-07
		359	292.0	305.6	292.0	6.7	6.9	22	19.22	3.30E-07
		414	289.8	305.6	289.8	7.8	8.0	22	19.08	3.34E-07
		484	287.4	305.6	287.4	9.0	9.2	22	18.92	3.30E-07
		537	285.4	305.6	285.4	10.0	10.2	22	18.79	3.32E-07
		1287	260.8	305.6	260.8	22.2	22.6	22	17.17	3.21E-07
		1361	258.1	305.6	258.1	23.8	23.7	22	16.99	3.23E-07

Trial 1 - Average k_{20} (cm/s): **3.26E-07**

Remarks

Soil cement

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

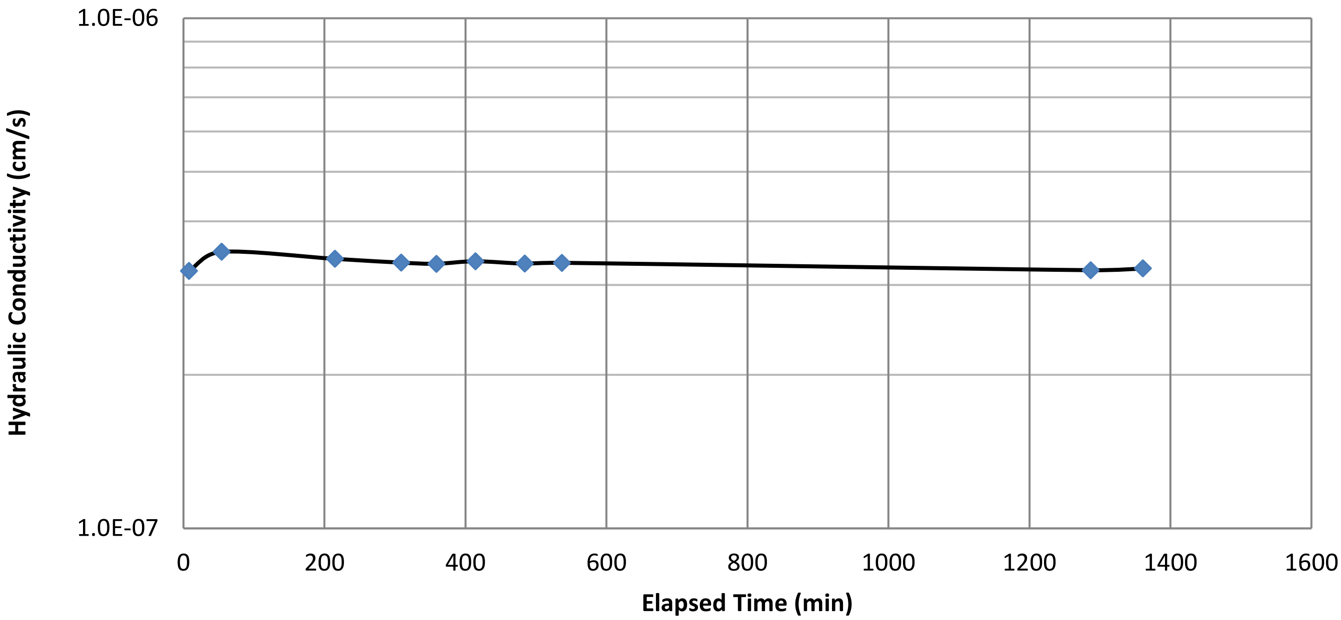
T. Madera	February 7, 2023	INITIAL	DATE
TESTED BY	DATE	CHECKED BY	DATE

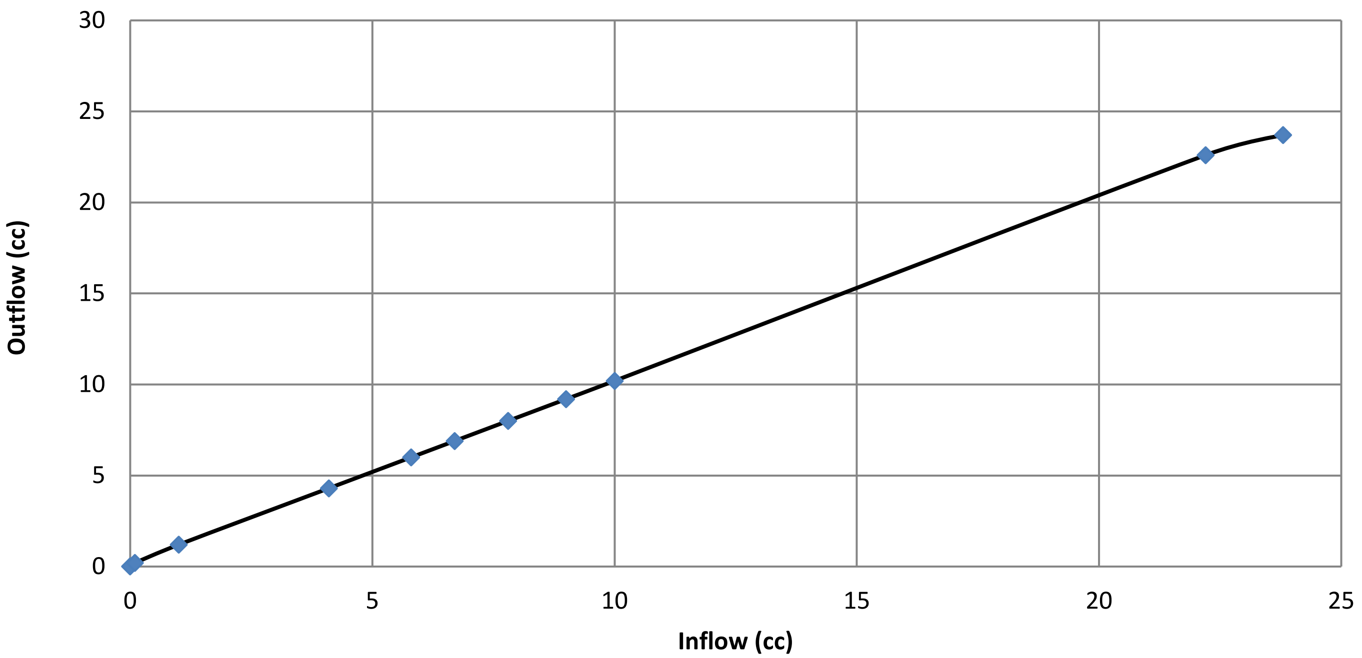
WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	February 7, 2023	INITIAL	DATE
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
300, 3811 North Fraser Way, Burnaby, British Columbia, Canada V5J 5J2
Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	E (56 days)
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.69	7.67
Length (cm)	15.05	15.07
Area (cm^2)	46.40	46.21
Volume (cm^3)	698.34	696.43
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	965.3
Back Pressure (kPa)	896.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	-1.52
B_{value}	0.96
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1193.61	1200.39
Dry Weight (g)	844.38	844.38
w (%)	41.4	42.2
ρ_{dry} (kg/m^3)	1209	1212
S (%)	96.8	99.3
e	1.07	1.06
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
3-10-23	06:19	0	305.0	--	--	0.0	0.0	22	-	--
		3	304.9	305.0	304.9	0.0	0.1	22	20.23	2.83E-07
		33	303.3	305.0	303.3	0.8	0.9	22	20.13	4.39E-07
		54	301.2	305.0	301.2	1.8	2.0	22	19.99	6.02E-07
		151	295.6	305.0	295.6	4.6	4.8	22	19.61	5.37E-07
		212	292.5	305.0	292.5	6.2	6.3	22	19.41	5.11E-07
		265	289.8	305.0	289.8	7.6	7.6	22	19.23	5.00E-07
		334	286.4	305.0	286.4	9.3	9.3	22	19.00	4.88E-07
		437	283.3	305.0	283.3	10.8	10.9	22	18.80	4.38E-07
		483	280.1	305.0	280.1	12.4	12.5	22	18.59	4.57E-07
		541	277.5	305.0	277.5	13.7	13.8	22	18.41	4.53E-07
		614	275.5	305.0	275.5	14.7	14.8	22	18.28	4.29E-07
		664	273.4	305.0	273.4	15.7	15.9	22	18.14	4.27E-07
						Trial 1 - Average k_{20} (cm/s)				**4.41E-07**

Remarks

Soil cement

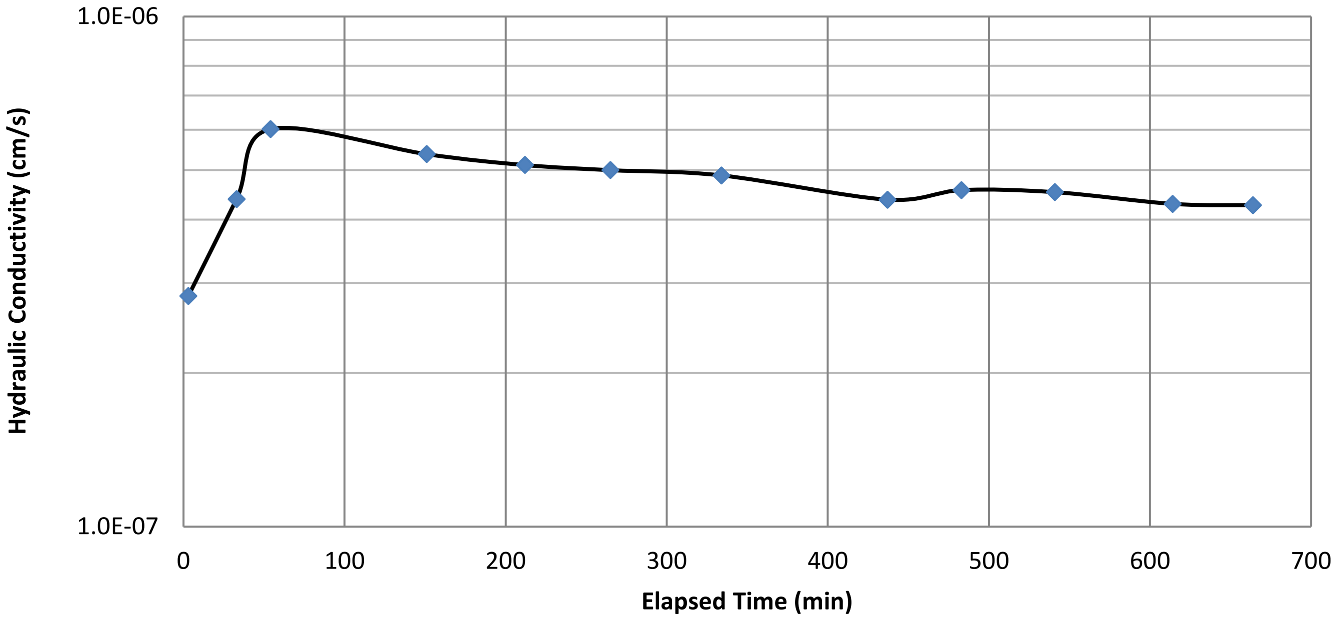
The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

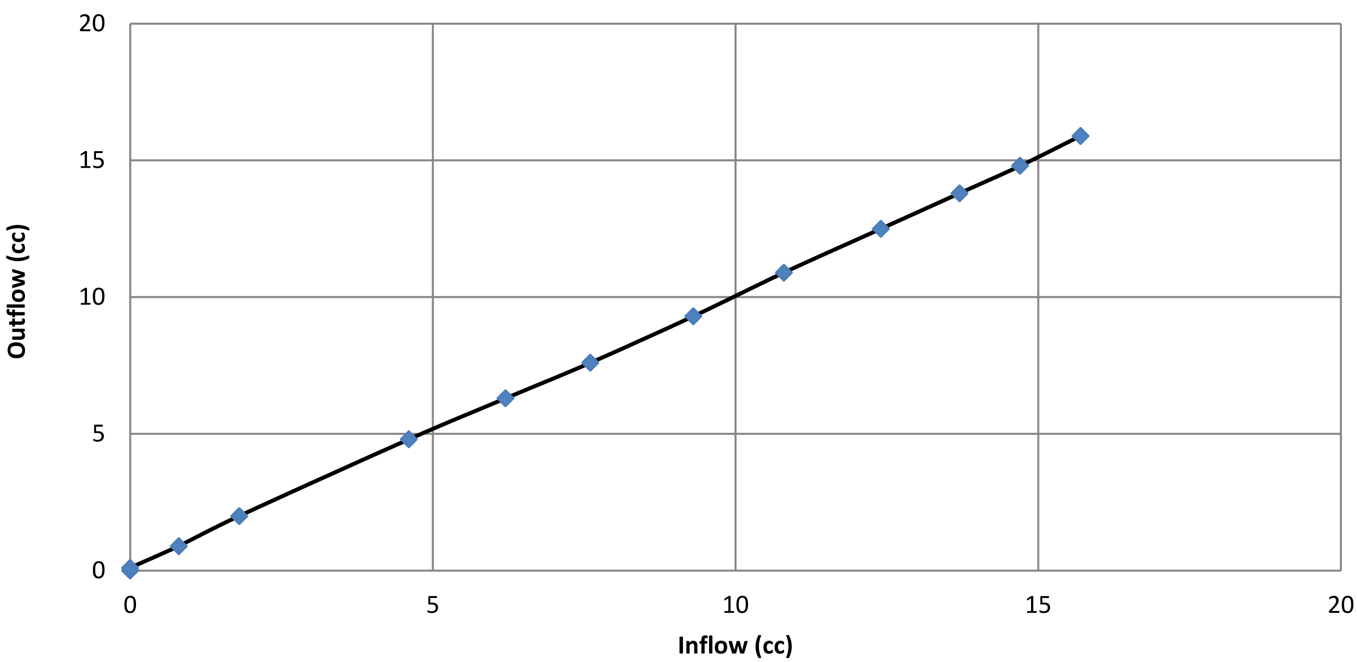
T. Madera	March 10, 2023	D. Lim	March 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M4-4B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	E (56 days)
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	March 10, 2023	D. Lim	March 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

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Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M5-6B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.67	7.67
Length (cm)	15.13	15.13
Area (cm^2)	46.21	46.26
Volume (cm^3)	699.29	699.70
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	979.1
σ'_{3c} (kPa)	55.2
δV_c (cc)	-2.01
B_{value}	0.98
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1124.18	1131.71
Dry Weight (g)	719.43	719.43
w (%)	56.3	57.3
ρ_{dry} (kg/m^3)	1029	1028
S (%)	97.8	99.5
e	1.45	1.45
Gs (assumed)	2.52	2.52

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-9-23	07:00	0	305.2	--	--	0.0	0.0	22	-	--
		9	304.5	305.2	304.5	0.3	0.4	22	20.13	6.63E-07
		39	302.8	305.2	302.8	1.1	1.3	22	20.02	5.26E-07
		57	301.7	305.2	301.7	1.6	1.9	22	19.95	5.26E-07
		120	298.4	305.2	298.4	3.3	3.5	22	19.73	4.88E-07
		171	295.7	305.2	295.7	4.6	4.9	22	19.55	4.80E-07
		229	292.7	305.2	292.7	6.2	6.3	22	19.35	4.74E-07
		288	289.6	305.2	289.6	7.7	7.9	22	19.15	4.73E-07
		354	286.2	305.2	286.2	9.4	9.6	22	18.92	4.72E-07
		381	284.8	305.2	284.8	10.1	10.3	22	18.83	4.72E-07
		447	281.6	305.2	281.6	11.7	11.9	22	18.62	4.68E-07
		518	278.0	305.2	278.0	13.4	13.8	22	18.38	4.68E-07
		600	274.0	305.2	274.0	15.4	15.8	22	18.12	4.67E-07
		670	270.7	305.2	270.7	17.1	17.4	22	17.90	4.65E-07
		737	267.4	305.2	267.4	18.8	19.0	22	17.68	4.66E-07
		794	264.8	305.2	264.8	20.1	20.3	22	17.51	4.64E-07
						Trial 1 - Average k_{20} (cm/s)				**4.66E-07**

Remarks

Soil cement

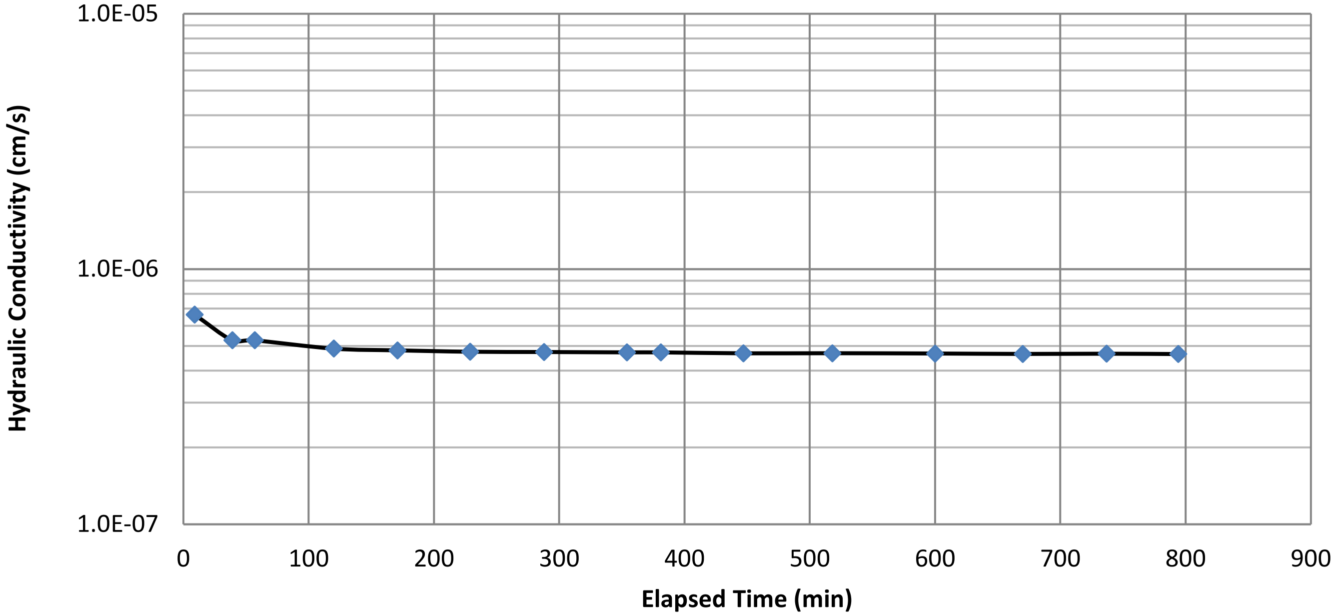
The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

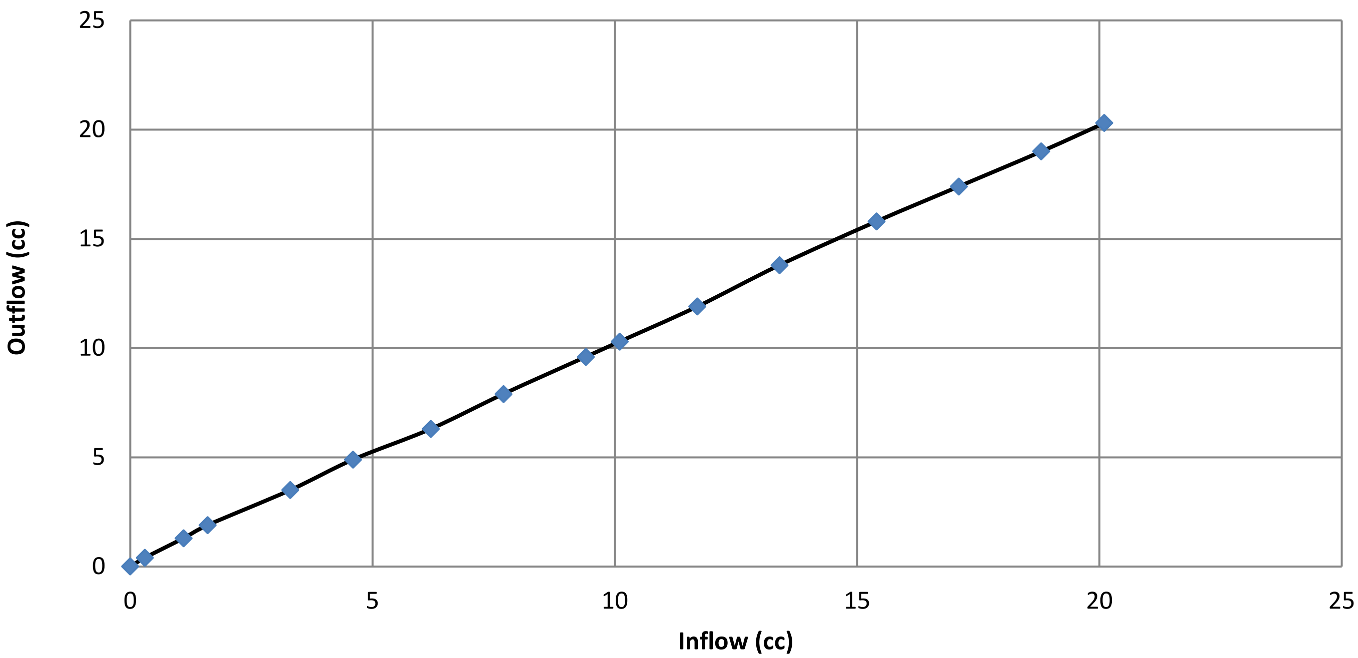
T. Madera	February 9, 2023	D. Lim	February 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M5-6B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	February 9, 2023	D. Lim	February 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

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Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.67	7.67
Length (cm)	15.13	15.13
Area (cm^2)	46.21	46.26
Volume (cm^3)	699.29	699.70
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	979.1
σ'_{3c} (kPa)	55.2
δV_c (cc)	-2.01
B_{value}	0.98
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1124.18	1131.71
Dry Weight (g)	719.43	719.43
w (%)	56.3	57.3
ρ_{dry} (kg/m^3)	1029	1028
S (%)	97.8	99.5
e	1.45	1.45
Gs (assumed)	2.52	2.52

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-9-23	07:00	0	305.2	--	--	0.0	0.0	22	-	--
		9	304.5	305.2	304.5	0.3	0.4	22	20.13	6.63E-07
		39	302.8	305.2	302.8	1.1	1.3	22	20.02	5.26E-07
		57	301.7	305.2	301.7	1.6	1.9	22	19.95	5.26E-07
		120	298.4	305.2	298.4	3.3	3.5	22	19.73	4.88E-07
		171	295.7	305.2	295.7	4.6	4.9	22	19.55	4.80E-07
		229	292.7	305.2	292.7	6.2	6.3	22	19.35	4.74E-07
		288	289.6	305.2	289.6	7.7	7.9	22	19.15	4.73E-07
		354	286.2	305.2	286.2	9.4	9.6	22	18.92	4.72E-07
		381	284.8	305.2	284.8	10.1	10.3	22	18.83	4.72E-07
		447	281.6	305.2	281.6	11.7	11.9	22	18.62	4.68E-07
		518	278.0	305.2	278.0	13.4	13.8	22	18.38	4.68E-07
		600	274.0	305.2	274.0	15.4	15.8	22	18.12	4.67E-07
		670	270.7	305.2	270.7	17.1	17.4	22	17.90	4.65E-07
		737	267.4	305.2	267.4	18.8	19.0	22	17.68	4.66E-07
		794	264.8	305.2	264.8	20.1	20.3	22	17.51	4.64E-07
						Trial 1 - Average k_{20} (cm/s)				**4.66E-07**

Remarks

Soil cement

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	February 9, 2023	D. Lim	February 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

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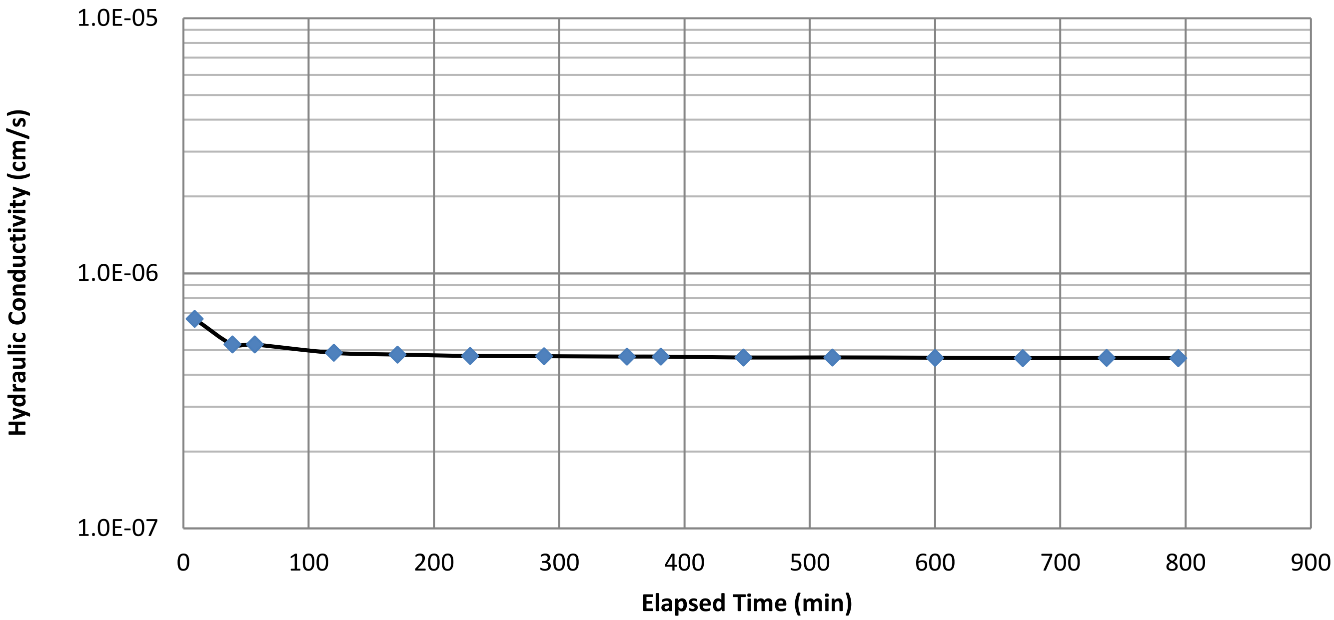
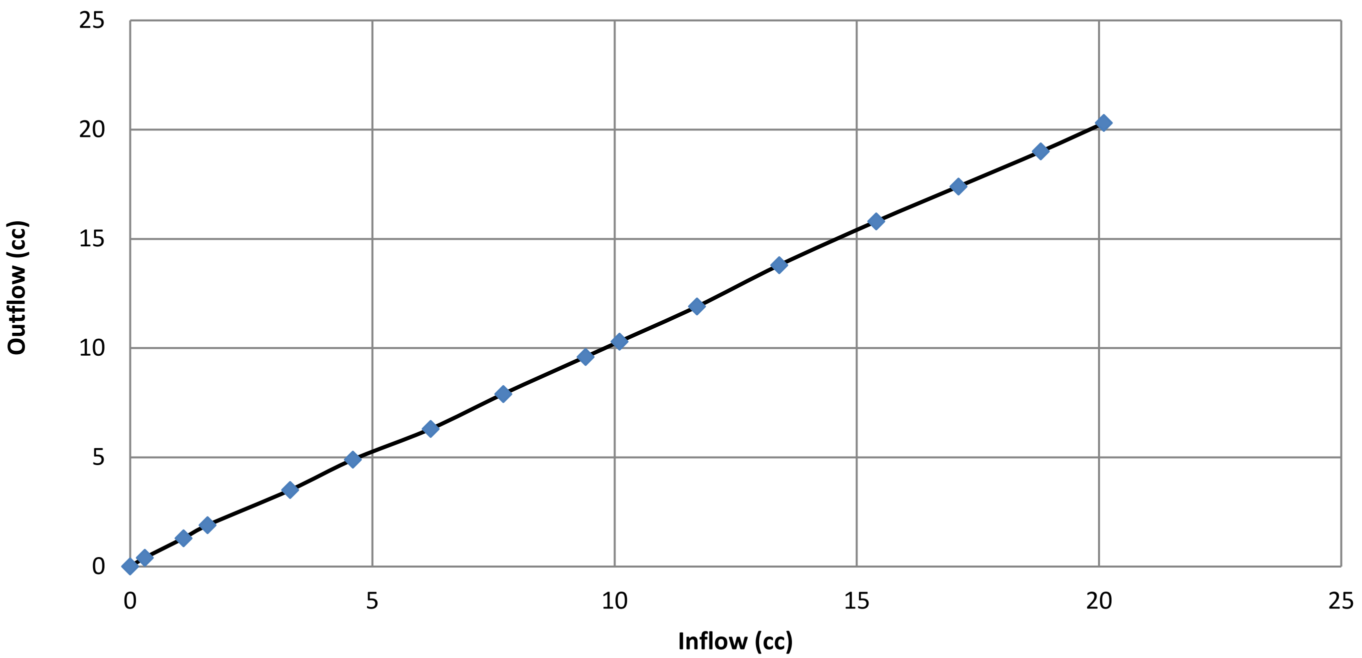
Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	February 9, 2023	D. Lim	February 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

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Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.66	7.66
Length (cm)	15.09	15.09
Area (cm^2)	46.06	46.05
Volume (cm^3)	694.80	694.77
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	1034.2
Back Pressure (kPa)	965.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	-0.80
B_{value}	0.97
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1120.92	1131.29
Dry Weight (g)	730.53	730.53
w (%)	53.4	54.9
ρ_{dry} (kg/m^3)	1051	1051
S (%)	97.0	99.6
e	1.38	1.38
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
2-6-23	07:00	0	305.2	--	--	0.0	0.0	22	-	--
		1	305.1	305.2	305.1	0.1	0.0	22	20.22	8.53E-07
		15	304.5	305.2	304.5	0.5	0.2	22	20.18	3.98E-07
		43	303.6	305.2	303.6	1.0	0.6	22	20.12	3.18E-07
		106	301.7	305.2	301.7	1.9	1.6	22	20.00	2.83E-07
		180	299.4	305.2	299.4	3.1	2.7	22	19.85	2.77E-07
		298	296.0	305.2	296.0	4.8	4.4	22	19.62	2.67E-07
		386	293.4	305.2	293.4	6.1	5.7	22	19.45	2.66E-07
		446	291.8	305.2	291.8	6.9	6.5	22	19.34	2.62E-07
		512	289.5	305.2	289.5	8.0	7.7	22	19.19	2.68E-07
		568	288.1	305.2	288.1	8.7	8.4	22	19.10	2.64E-07
		615	286.7	305.2	286.7	9.4	9.1	22	19.00	2.65E-07
		1378	266.6	305.2	266.6	19.6	19.0	22	17.67	2.55E-07
		1520	263.1	305.2	263.1	21.4	20.7	22	17.44	2.54E-07
						Trial 1 - Average k_{20} (cm/s)				**2.60E-07**

Remarks

Soil cement

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	February 8, 2023	D. Lim	February 9, 2023
TESTED BY	DATE	CHECKED BY	DATE

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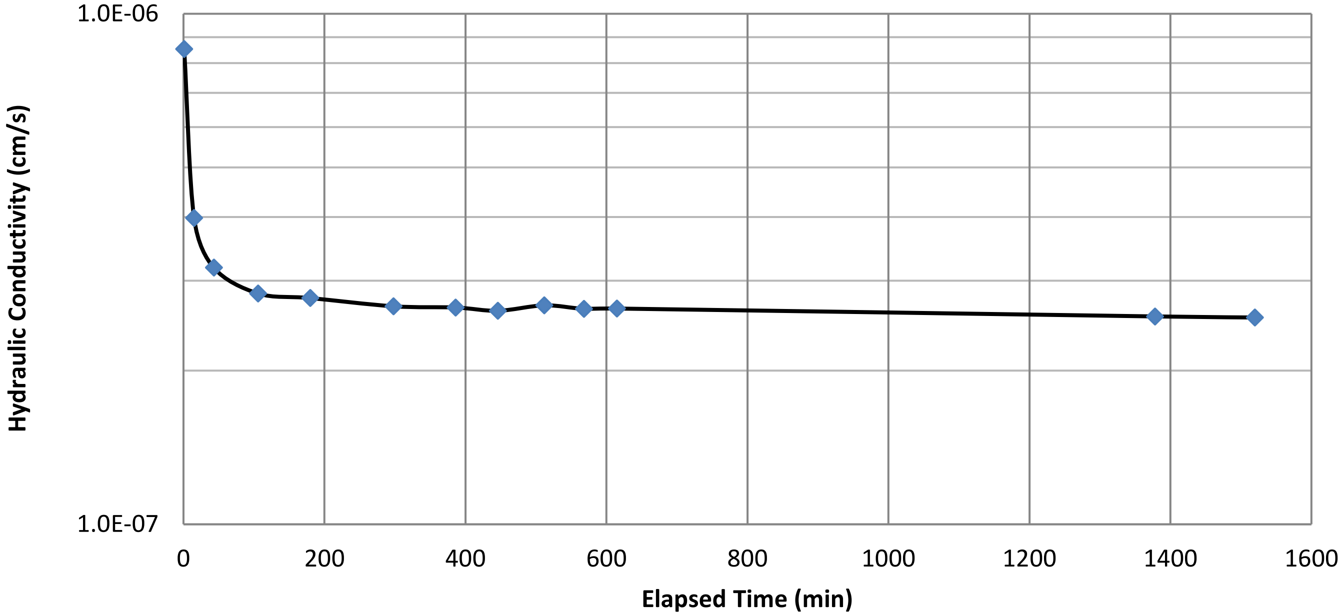
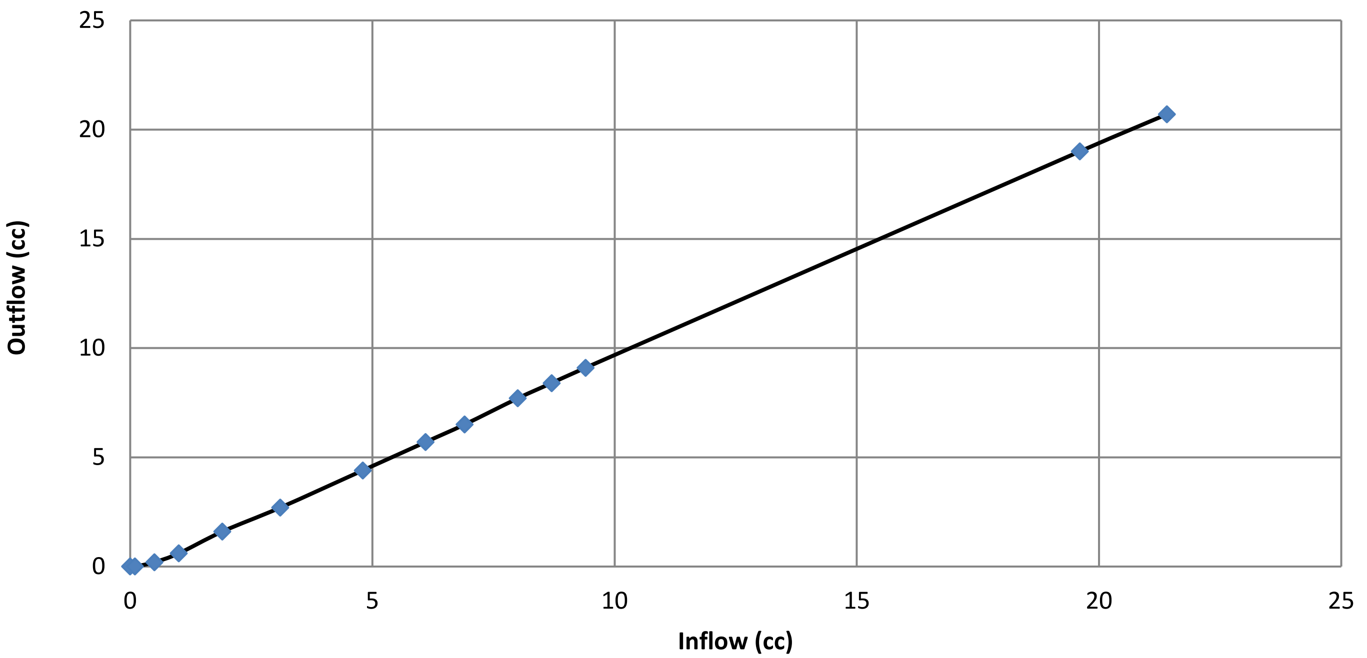
Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	February 8, 2023	D. Lim	February 9, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	E (56 days)
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.67	7.66
Length (cm)	14.97	15.00
Area (cm^2)	46.18	46.03
Volume (cm^3)	691.15	690.45
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	965.3
Back Pressure (kPa)	896.3
σ'_{3c} (kPa)	68.9
δV_c (cc)	-2.30
B_{value}	0.97
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1117.21	1120.06
Dry Weight (g)	723.95	723.95
w (%)	54.3	54.7
ρ_{dry} (kg/m^3)	1047	1049
S (%)	97.9	98.8
e	1.39	1.38
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
3-10-23	00:00	0	305.0	--	--	0.0	0.0	22	-	--
		1	304.8	305.0	304.8	0.1	0.1	22	20.32	1.70E-06
		9	304.6	305.0	304.6	0.2	0.2	22	20.31	3.78E-07
		55	303.3	305.0	303.3	0.8	0.9	22	20.22	2.63E-07
		80	302.6	305.0	302.6	1.2	1.2	22	20.17	2.56E-07
		116	301.8	305.0	301.8	1.6	1.6	22	20.12	2.35E-07
		206	299.4	305.0	299.4	2.8	2.8	22	19.96	2.33E-07
		338	296.1	305.0	296.1	4.5	4.4	22	19.74	2.27E-07
		458	294.4	305.0	294.4	5.4	5.2	22	19.63	2.00E-07
		469	292.8	305.0	292.8	6.1	6.1	22	19.52	2.25E-07
		541	291.0	305.0	291.0	7.0	7.0	22	19.40	2.25E-07
		603	289.4	305.0	289.4	7.8	7.8	22	19.29	2.25E-07
		647	288.4	305.0	288.4	8.3	8.3	22	19.23	2.24E-07
		696	287.2	305.0	287.2	8.9	8.9	22	19.15	2.24E-07
		1592	266.4	305.0	266.4	19.3	19.3	22	17.76	2.20E-07
		1646	265.2	305.0	265.2	19.9	19.9	22	17.68	2.20E-07
		1709	263.8	305.0	263.8	20.6	20.6	22	17.59	2.20E-07
		1777	262.4	305.0	262.4	21.3	21.3	22	17.49	2.19E-07
		1840	261.1	305.0	261.1	22.0	21.9	22	17.41	2.19E-07
						Trial 1 - Average k_{20} (cm/s)				**2.19E-07**

Remarks

Soil cement

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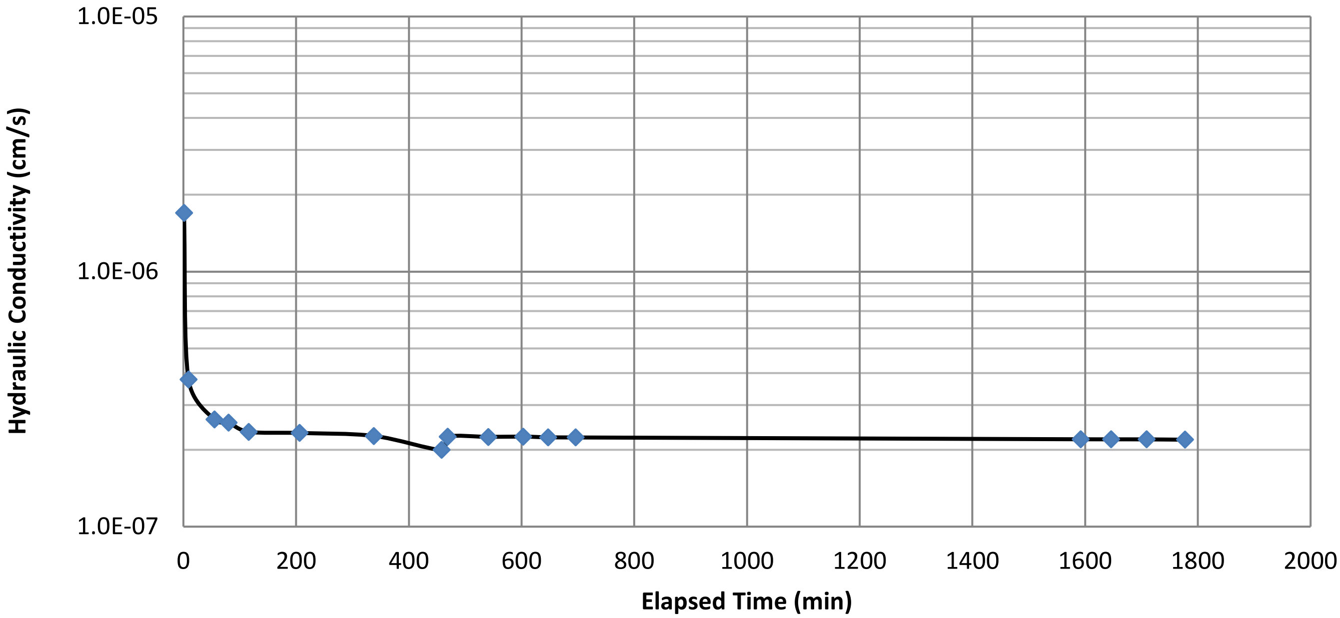
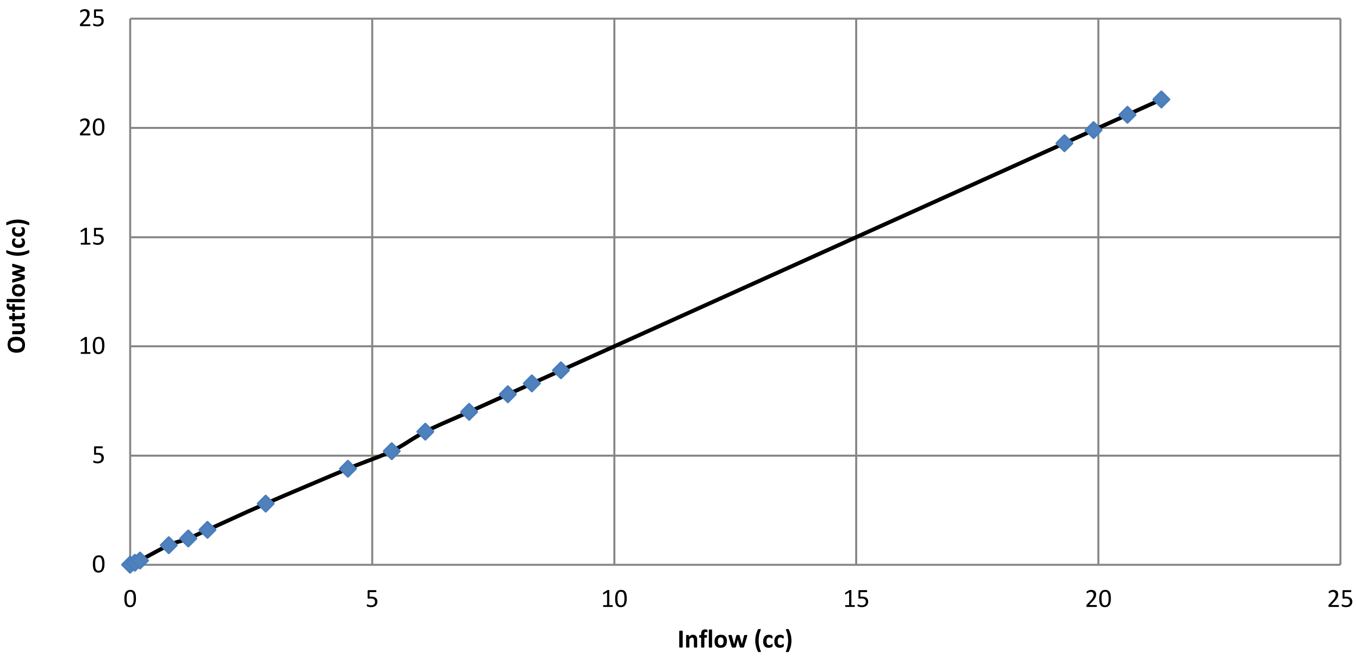
T. Madera	March 10, 2023	D. Lim	March 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M6-6B-7C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	E (56 days)
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	March 10, 2023	D. Lim	March 16, 2023
TESTED BY	DATE	CHECKED BY	DATE

WSP Canada Inc.
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Tel: 604-412-6899 Fax: 604-412-6816 www.golder.com

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M7-3B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	B
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.62	7.58
Length (cm)	15.22	15.14
Area (cm^2)	45.63	45.14
Volume (cm^3)	694.73	683.68
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	930.8
Back Pressure (kPa)	827.4
σ'_{3c} (kPa)	103.4
δV_c (cc)	-8.46
B_{value}	0.91
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1240.59	1243.24
Dry Weight (g)	937.34	937.34
w (%)	32.4	32.6
ρ_{dry} (kg/m^3)	1349	1371
S (%)	94.8	99.1
e	0.85	0.82
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
3-31-23	07:19	0	304.9	--	--	0.0	0.0	22	-	--
		7	304.7	304.9	304.7	0.1	0.1	22	20.12	2.50E-07
		60	303.2	304.9	303.2	0.8	0.9	22	20.02	2.48E-07
		125	301.3	304.9	301.3	1.8	1.8	22	19.90	2.53E-07
		195	299.2	304.9	299.2	2.9	2.8	22	19.76	2.57E-07
		267	297.5	304.9	297.5	3.7	3.7	22	19.65	2.45E-07
		327	296.1	304.9	296.1	4.7	4.1	22	19.55	2.39E-07
		387	294.0	304.9	294.0	5.5	5.4	22	19.41	2.51E-07
		447	292.6	304.9	292.6	6.3	6.0	22	19.32	2.45E-07
		507	291.2	304.9	291.2	7.1	6.6	22	19.23	2.42E-07
		567	289.8	304.9	289.8	7.7	7.4	22	19.14	2.39E-07
		1453	270.2	304.9	270.2	17.7	17.0	22	17.84	2.22E-07
		1495	269.1	304.9	269.1	18.2	17.6	22	17.77	2.23E-07
		1552	268.1	304.9	268.1	18.8	18.0	22	17.70	2.21E-07
		1889	262.4	304.9	262.4	21.7	20.8	22	17.33	2.12E-07
						Trial 1 - Average k_{20} (cm/s)				**2.19E-07**

Remarks

Soil cement

Could not achieve a B of 0.95 due to equipment limitations.

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T. Madera	March 31, 2023	D. Lim	April 6, 2023
TESTED BY	DATE	CHECKED BY	DATE

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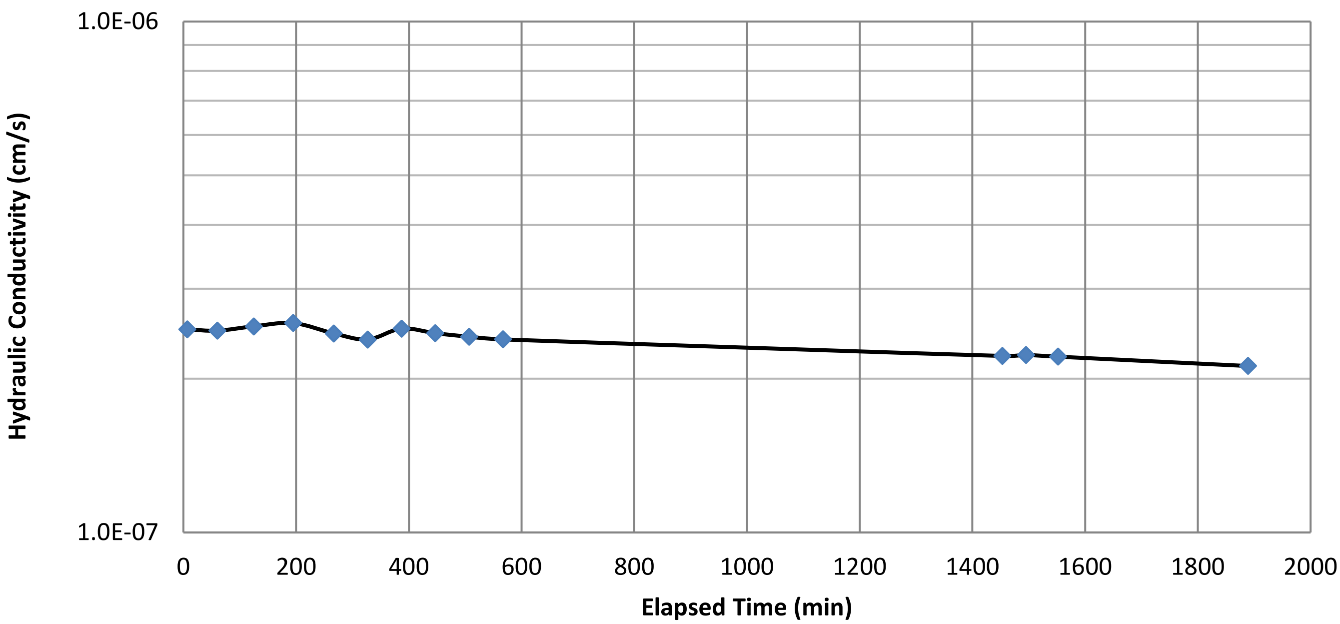
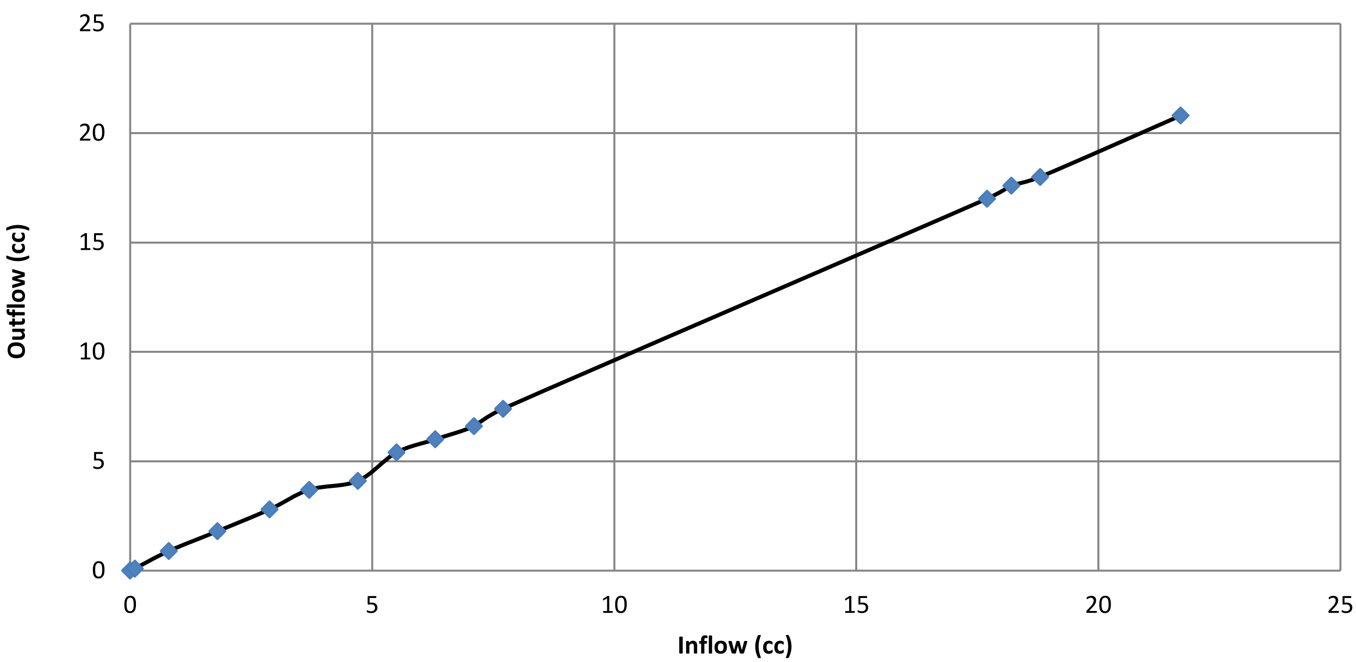
Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M7-3B-5C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	B
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	March 31, 2023	D. Lim	April 6, 2023
TESTED BY	DATE	CHECKED BY	DATE

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Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M9-3B-15C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	B
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.63	7.62
Length (cm)	15.28	15.29
Area (cm^2)	45.67	45.66
Volume (cm^3)	697.88	698.06
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	930.8
Back Pressure (kPa)	827.4
σ'_{3c} (kPa)	103.4
δV_c (cc)	-0.85
B_{value}	0.89
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1251.10	1270.86
Dry Weight (g)	1069.72	1069.72
w (%)	17.0	18.8
ρ_{dry} (kg/m^3)	1533	1532
S (%)	67.2	74.4
e	0.63	0.63
Gs (assumed)	2.50	2.50

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
4/27/23	09:57	0	366.3	--	--	0.0	0.0	22	-	--
		8600	365.8	366.3	365.8	0.4	0.1	22	23.93	4.22E-10
		15831	365.3	366.3	365.3	0.7	0.3	22	23.90	4.59E-10
		30238	364.8	366.3	364.8	1.0	0.5	22	23.86	3.61E-10
		41798	364.2	366.3	364.2	1.3	0.8	22	23.82	3.66E-10
		51728	363.8	366.3	363.8	1.5	1.0	22	23.80	3.52E-10
		87990	362.7	366.3	362.7	2.1	1.5	22	23.73	2.99E-10

Trial 1 - Average k_{20} (cm/s) **3.44E-10**

Remarks

Soil cement

The test data given herein pertain to the sample provided only. This report constitutes a testing service only.

T. Madera	April 27, 2023	DL	July 3, 2023
TESTED BY	DATE	CHECKED BY	DATE

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Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M9-3B-15C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021

Sample Geometry

	Initial	*Final*
Diameter (cm)	7.63	7.63
Length (cm)	15.28	15.29
Area (cm^2)	45.78	45.74
Volume (cm^3)	699.46	699.30
Sample Type	Intact	
Permeant Liquid	Tap water	

Test Parameters

Cell Pressure (kPa)	999.7
Back Pressure (kPa)	896.3
σ'_{3c} (kPa)	103.4
δV_c (cc)	-1.80
B_{value}	0.92
Headwater/Tailwater Tubes	
a_{in} cm^2	1.00
a_{out} cm^2	1.00

Phase Relationships

	Initial	*Final*
Wet Weight (g)	1246.81	1267.63
Dry Weight (g)	1058.48	1058.48
w (%)	17.8	19.8
ρ_{dry} (kg/m^3)	1513	1514
S (%)	62.8	69.7
e	0.75	0.75
Gs (assumed)	2.65	2.65

Date	Time	Elapsed Time	Head Loss			Flow Volume		Temp	Hydraulic Gradient	Hydraulic Conductivity k_{20}
mm/dd/yy	hh:mm	(min)	h (cm)	h_1 (cm)	h_2 (cm)	V_{in} (cc)	V_{out} (cc)	(°C)		(cm/s)
5/2/23	08:21	0	235.0	--	--	0.0	0.0	22	-	--
		1266	234.6	235.0	234.6	0.3	0.1	22	15.35	3.57E-09
		8567	234.4	235.0	234.4	0.5	0.1	22	15.33	7.92E-10
		22974	233.7	235.0	233.7	0.9	0.4	22	15.29	6.41E-10
		34534	233.2	235.0	233.2	1.2	0.6	22	15.26	5.91E-10
		44464	232.6	235.0	232.6	1.5	0.9	22	15.22	6.13E-10
		80726	231.2	235.0	231.2	2.3	1.5	22	15.12	5.36E-10

Trial 1 - Average k_{20} (cm/s) **5.95E-10**

Remarks

Soil cement

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T. Madera	May 2, 2023	DL	July 3, 2023
TESTED BY	DATE	CHECKED BY	DATE

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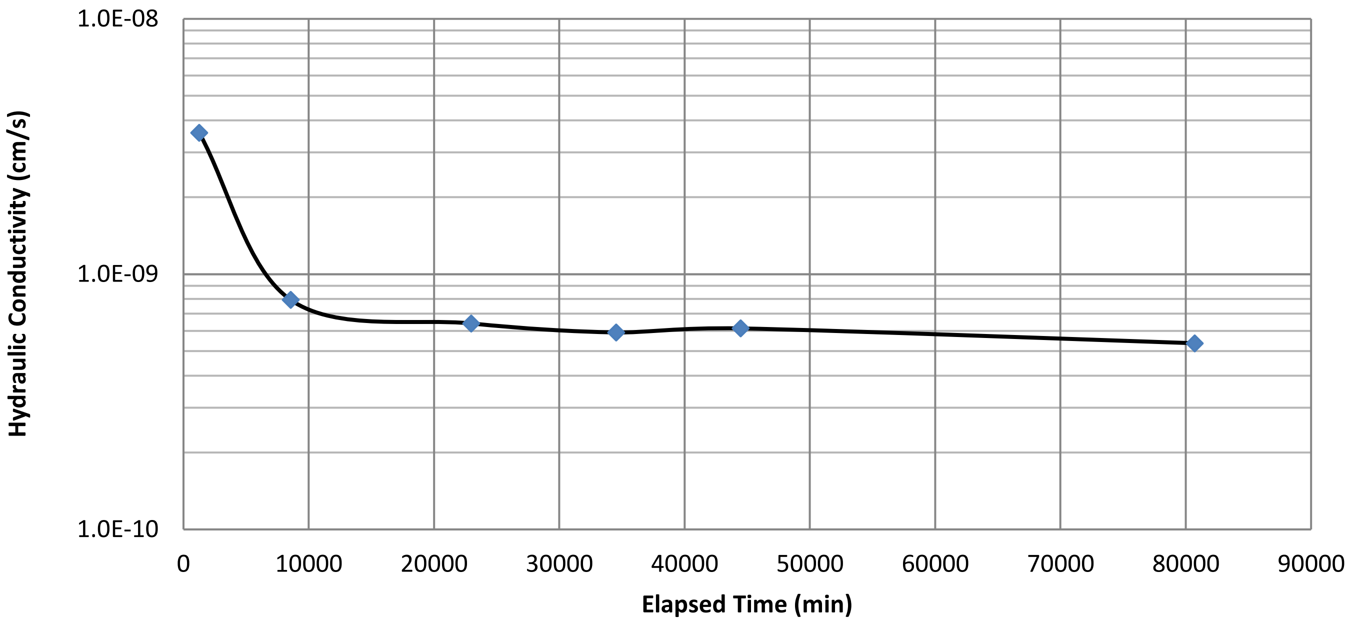
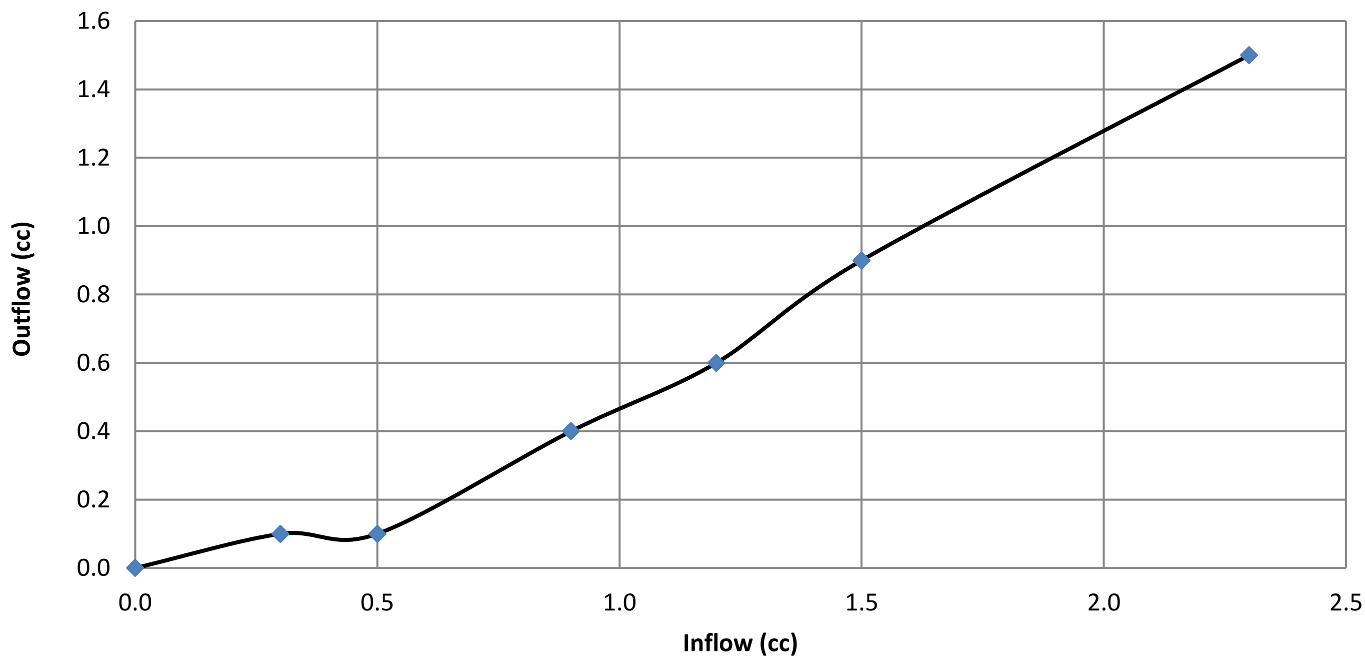
Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M9-3B-15C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	D
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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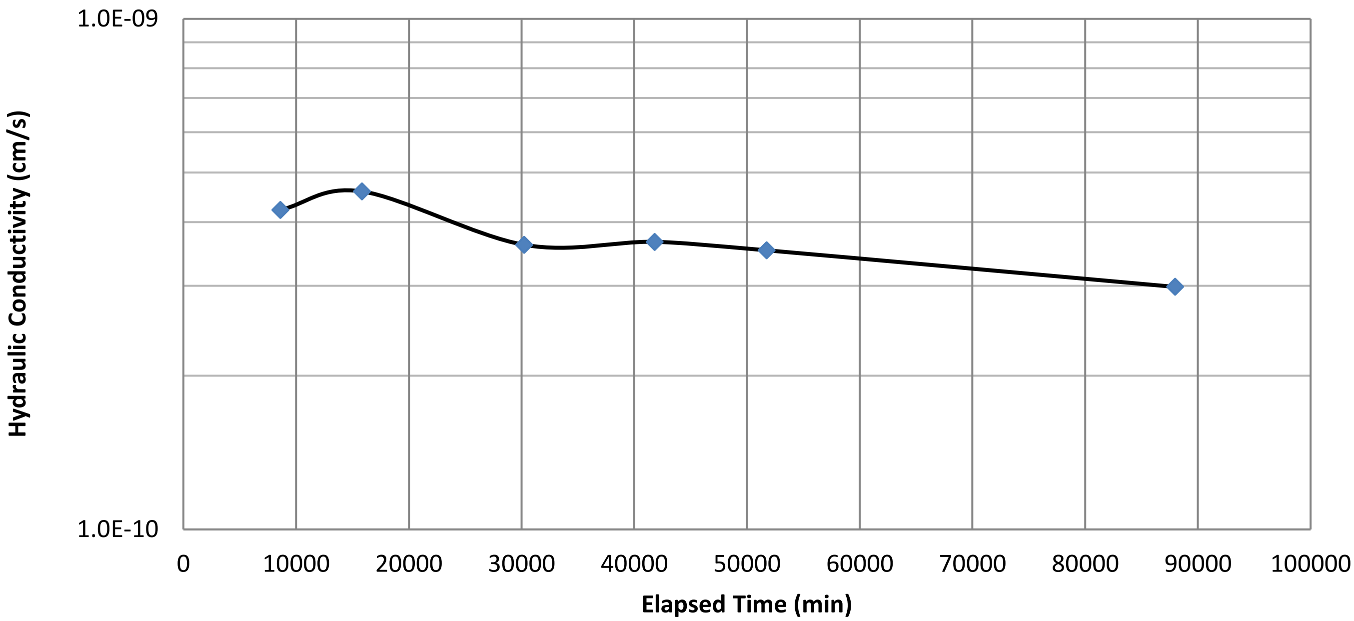
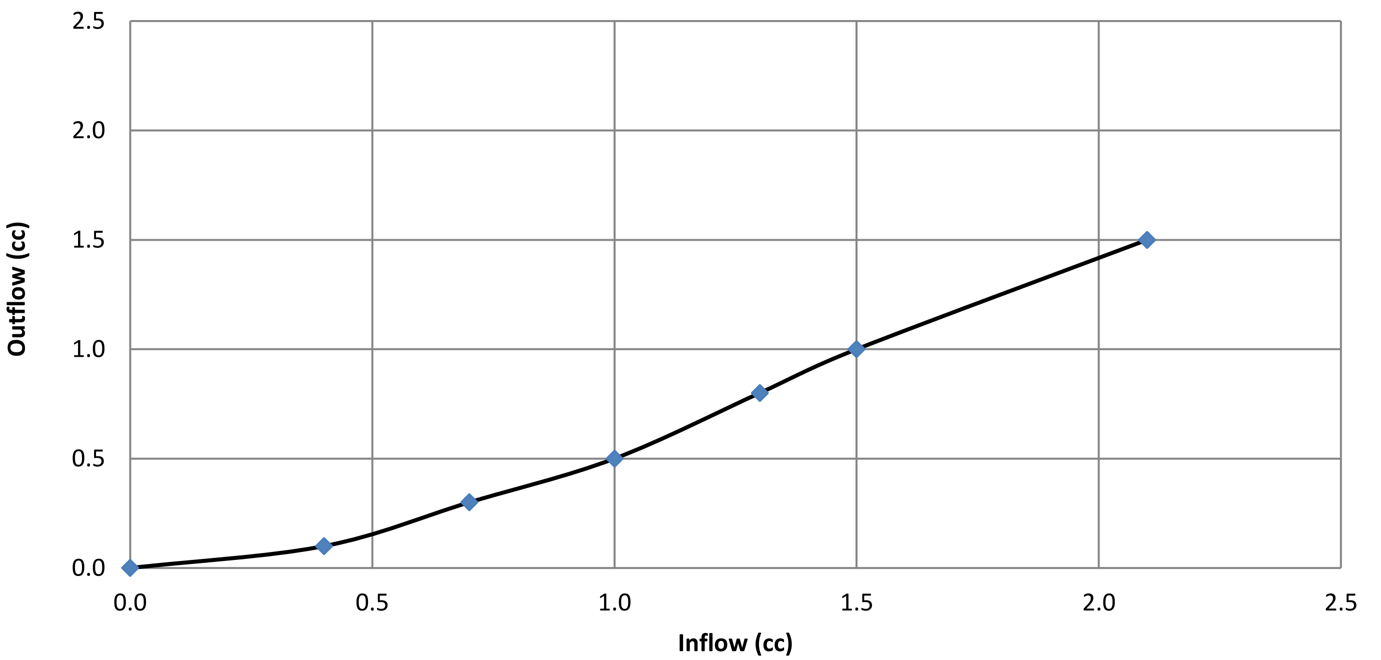
T. Madera	May 2, 2023	DL	July 3, 2023
TESTED BY	DATE	CHECKED BY	DATE

Measurements of Hydraulic Conductivity Using a Flexible Wall Permeameter

ASTM D5084-10 Method C

Project No.:	23591773-2000-2300	**Borehole:**	M9-3B-15C
Project:	SCS/PLT MCDONOUGH AP1	**Sample:**	B
Location:	N/A	**Depth (m):**	N/A
Client:	GOLDER ASSOCIATES USA INC.	**Lab ID No:**	B23-021





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T. Madera	April 27, 2023	DL	July 3, 2023
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