PLANT McDONOUGH-ATKINSON
CCR SURFACE IMPOUNDMENTS
(CCR UNIT AP-2, COMBINED CCR UNIT AP-3/4)
COBB COUNTY, GEORGIA
PART B SECTION 2 – ENGINEERING REPORT

FOR

Georgia Power

Revision 01 – November 2020

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Executive Summary

Golder Associates Inc. (Golder) has compiled supporting calculations for the closure of inactive CCR Units AP-2 and Combined Unit AP-3/4 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
- Contact Water Management System
- Final Cover System
- Surface Water Management

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

Golder Associates Inc. (Golder) and Georgia Power Company (Georgia Power) have prepared design calculations to support the design and permitting of CCR Unit AP-2 and Combined Unit AP-3/4 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 disposal 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-2, AP-3 and AP-4 becoming inactive CCR surface impoundments prior to closure construction activities. In January 2016, closure activities were initiated for the units, and consisted of closure by removal of CCR for AP-2, and a combination of closure by removal and consolidating and closing in place as a combined unit for AP-3 and AP-4, referred to as Combined Unit AP-3/4.

Closure activities for AP-2 and AP-3/4 were conducted following the closure design presented in the Closure Plan Drawings of Part A of this permit application. 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 (AP-3/4)
- A contact water management system to collect water that has contacted CCR material for storage and treatment
- 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 a detention pond to reduce discharge from the units to levels below existing conditions

The Closure Plan Drawings provides 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
- Contact Water Collection System
- Final Cover System
- Surface Water Management

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 units 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 units under final conditions

This geotechnical design discussion presents Golder's stability evaluation of the containment berms (dikes) surrounding inactive CCR Units AP-2, AP-3, and AP-4 at Plant McDonough related to the requirements in the US EPA's 2015 Final Rule on the Disposal of Coal Combustion Residuals (CCR; EPA Rule) and the State of Georgia Solid Waste Management Rule 391-3-4-.10.

This report presents the calculated geotechnical stability and settlement of the final closure condition of the AP-2 and AP-3/4 units. As previously described, CCR materials have been excavated from within Unit AP-2, and units AP-3 and AP-4 are being closed as Combined Unit AP-3/4 using a combination of closure by removal to consolidate ash into a smaller footprint for capping in place. According to section § 257.73(e) of the rule, stability of earth structures must be assessed under four loading conditions:

- Storage Pool (§ 257.73(e)(i))
- Surcharge Pool (§ 257.73(e)(ii))
- Seismic Loading Conditions (§ 257.73(e)(iii))
- Post-Seismic Liquefaction Conditions (when liquefaction susceptible materials are present; § 257.73(e)(iv)).

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-2 and four cross-sections at AP-3/4 as shown in Appendix B. Subsurface stratigraphy at each cross-section was developed from data from historical boring and well records and data collected during Golder's subsurface explorations completed in multiple mobilizations from October 2015 to January 2016. Similarly, geotechnical material properties were developed for the dike, foundation, and impounded materials from the references mentioned herein. The Material Properties Calculation Package (Appendix A) provides details on Golder's geotechnical exploration 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

Golder modeled the storage pool using the long-term water levels at post-closure conditions. Long-term water levels at AP-2 are below the lowest grade of the impounded area, and any stormwater routed to AP-2 will be pumped out prior to future development backfill conditions; thus, AP-2 will not retain a storage pool. For AP-2 maximum pool storage stability analyses, Golder used long term water levels estimated for AP-2.

Likewise, AP-3/4 will not retain a storage pool. Water levels in AP-3/4 are modelled to drop below the bottom of the impounded ash in the long term due to capping and active and passive dewatering. For conservatism, Golder used the water levels at the end of dewatering for maximum pool storage stability calculations.

2.2.2 Surcharge Pool Conditions

For the surcharge pool scenario, Golder considered the impact of the 100-year, 24-hour rain event for Atlanta, GA. This event was calculated to cause a temporary pool of elevation 781.3 ft-msl to develop in AP-2 (Hydrology and Hydraulic Design for AP-2). AP-2 dike stability under surcharge pool was calculated with this pool elevation.

At AP-3/4, the rain event will cause storm water flow in the lined channels on the pond final cover, but will not significantly impact the water level below the final cover. Thus, Golder evaluated the stability of AP-3/4 slopes with channels to the flow depths (fully flowing) as calculated based on the Hydrology and Hydraulic Design for AP-3/4. The table below lists the depth of water considered in channels at each section.

Section	Channel Flow Depth (ft)
3/4A (North)	8.7
3/4A (South)	0.2
3/4B	9.3
3/4D	1.2
3/4J	9.5

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). The Bray and Travasarou displacement-based seismic slope stability screening method was used 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 (g = standard gravity). Details on the calculation of the pseudo-static coefficient are available in the Seismic Hazard Calculation Package (Appendix C).

2.2.4 Liquefaction Assessment

The CCR Rule specifies a required factor of safety of 1.2 against liquefaction for pond impoundment structures (§ 257.73(e)(iv)). The dikes and foundation soils at the location of the AP-2 and AP-3/4 analysis sections were evaluated for liquefaction susceptibility, and the calculated factors of safety against liquefaction are above 1.2. Details on the calculation of the liquefaction susceptibility are available in the Liquefaction Assessment Calculation Package (Appendix D).

2.3 Slope Stability Assessment Results

The table below presents the results of the slope stability analyses for the AP-2 and AP-3/4 dikes. For all cases analyzed, the calculated factors of safety are in excess of those required in Sections § 257.73(e)(i) to (iv) of the CCR Rule. The detailed stability results are presented in Figures 3 through 10 of Appendix B.

Long-Term Post-Closure Stability Analysis Results				
Analysis Case	Max. Storage Pool	Max. 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**			
Surface Impoundment AP-2				
2A	1.9	1.9	1.8	Not Applicable
2B	1.9	1.9	1.8	
2C	1.8	1.8	1.7	
Surface Impoundment AP-3/4				
3/4A (North)	2.1	2.1	1.8	Not Applicable
3/4A (South)	1.6	1.6	1.5	
3/4B	2.1	2.1	1.8	
3/4D	1.8	1.8	1.6	
3/4J	2.1	2.1	1.9	

2.4 Geotechnical Analysis Conclusions

Golder evaluated the slope stability of dikes surrounding AP-2 and AP-3/4 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 four loading scenarios presented in section § 257.73(e) of the EPA Rule:

- Storage Pool (§ 257.73(e)(i))
- Surcharge Pool (§ 257.73(e)(ii))
- Seismic Loading Conditions (§ 257.73(e)(iii))
- Post-Seismic Liquefaction Conditions (when liquefaction susceptible materials are present; § 257.73(e)(iv)).

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

- Settlement Analysis

Long-term settlement potential for AP-3/4 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-3/4 conditions consider settlement following closure from dewatering of the CCR and indicate that no settlement induced issues are calculated to occur following closure as detailed in Appendix E.

- Veneer Stability Analysis

Long-term and short-term veneer stability analyses were performed for the critical the access road conditions applicable to AP-3/4, including incorporation of equipment acceleration on the roads and were found to meet the required factors of safety as detailed in Appendix H.

- Anchor Trench Analysis

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

3.0 CONTACT WATER MANAGEMENT SYSTEM

AP-2 has fully removed all CCR materials from the unit during closure and as such does not require long term or post closure contact water management.

A primary objective of the AP-3/4 closure configuration is to limit long term contact of CCR with surface and ground water. This goal is achieved through capping of the CCR unit with a synthetic liner and removal of CCR materials from the topographic low-lying areas of AP-4 to allow for the long term drying of the stored CCR within the closed AP-3/4 unit.

The contact water management system for Combined Unit AP-3/4 provides for a method of controlled collection and treatment of contact water as a result of a series of drains and temporary dewatering wells located along the eastern and southwestern slope areas of AP-3/4.

The temporary dewatering wells are included as part of efforts to help expedite the natural long drying of CCR within the Combined Unit AP-3/4. These dewatering wells are proposed to be operated until such time that the area of influence around each well reaches equilibrium conditions, following which they are schedule to be decommissioned on a well by well basis when no longer needed to accelerate natural drainage.

3.1 Contact Water Generation

Contact water collected from the closed conditions of AP-3/4 is expected to be a result of the active and passive lowering of water levels within the inactive surface impoundment from its pre-closure conditions. Infiltration through the final cover system is designed to be limited, as discussed in Section 4.0 below.

3.2 Contact Water Management

Contact water from AP-3/4 is designed to be collected via a combination of the under slope drainage system, existing dam toe drains, and the temporary dewatering wells and forcemain as identified in the Closure Plan Drawings presented in Part A of this permit submittal. The contact water forcemain will convey water to the contact water sump, which along with the under slope drainage system sump will be pumped and undergo water treatment per the facility's water treatment plan.

3.2.1 Under Slope Drainage System

The under slope drainage system for AP-3/4 is designed for the collection and conveyance of contact water at the eastern slope of the proposed closed design for AP-3/4. Details for the under slope drainage system are located in the Plant McDonough AP-2 and AP-3/4 Closure Plan Drawings (Section 10 of Part A). The under slope drainage system is designed to collect interstitial seepage from the covered CCR mass and serve as the drainage layer for water that has contacted the CCR. The under slope drainage system consists of a combination of on-slope and toe drainage systems. The on slope collection system consists of 15-ft. wide geocomposite strips located below the lower portions of the soil buttress, and 3-ft. by 3-ft. sand trench drains spaced 25 ft. apart along the outer face of the eastern slope. The on-slope systems are hydraulically connected and convey flows to toe collection trenches with 4-in. or 6-in. nominal diameter HDPE drainage pipes within gravel drainage trenches. The under slope drainage system flows are directed via gravity to the under slope drainage system sump. Detailed calculations for the under slope drainage system are presented in Appendix F.

The in sump pump system for the under slope drainage sump is included in a 24 inch HDPE riser access pipe and outfitted with level monitoring and controls placed at the ground surface. Pumped flows from the under drain sump are directed to the combined AP-3/4 contact water collection sump being constructed within the lower portion of the eastern soil buttress, and then pumped and conveyed to the AP-3/4 water treatment area for treatment and discharge. If in the future flows are limited as expected, Georgia Power Company's long term contact water plans may either continue on-site treatment or potentially transition to a system of storage followed by conveyance to publicly owned treatment works (POTW) for treatment and discharge.

3.2.2 Toe Drains

The original construction of the AP-3 and AP-4 dams included internal drainage with toe drain outlets. The toe drains have historically been monitored with flows collected and directed to the AP-4 pond in recent years. The AP-3 toe drains were confirmed to be dry during the early portions of closure and were abandoned via grouting as part of the AP-3/4 closure efforts.

Existing toe drains for the AP-4 dam are located along the eastern portions of the AP-4 dam and were retrofitted in past efforts by Georgia Power to be collected at a series of sump locations for pumping to AP-4. As part of CCR closure efforts the toe drains to remain and not be over excavated by the lowering of the AP-4 dam will continue to be collected in sumps and directed to the post closure contact water collection sump being constructed at the toe of the eastern portion of the AP-4 slope. Flows into the AP-4 contact water collection sump will be pumped to the water treatment area for treatment prior to discharge through the site's AP-4 NPDES outfall.

The final toe drain locations and configurations are presented in the Closure Design Plans.

3.2.3 Contact Water Conveyance and Sump Systems

Following closure construction activities, the contact water conveyance system is comprised of seven (7) dewatering wells designed to withdraw a combined contact water flow rate of 50 gallons per minute (gpm). Contact water is then routed to the twin eight-inch diameter precast sumps located to the east of the AP-3/4 closure.

3.3 Contact Water Treatment

All contact water collected through the under slope drainage system and contact water conveyance system will be collected at the sump location and routed to the wastewater treatment system located south of Combined Unit AP-3/4. The water treatment facility is located on a built platform over an area of natural high ground to the south of AP-3/4 and adjacent to the closed AP-4 outfall area. Following pumping of CCR contact water into the wastewater treatment system, the treated water is ultimately discharged through the existing permitted NPDES outfall at AP-4.

4.0 FINAL COVER SYSTEM

4.1 General

AP-2 has fully removed all CCR materials from the unit during closure and as such does not require or include a final cover lining system.

The closure of AP-3/4 has been designed with a final cover system that consists of two options for the final cover system of the unit.

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 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) which consists of the following layers:

- 18-inch thick 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 can be found on Sheet 28 “Closure Details” of the Closure Plan Drawings for Plant McDonough (Part A of this Permit Application).

4.2 Alternative Final Cover Design

As indicated in Section 4.1, the final cover system designed for AP-3/4 consists of a ClosureTurf™ geosynthetic cap system utilizing a variety of infill options as delineated in the Permit Closure Design Plans. As part of the closure design, Golder completed an evaluation of the percolation potential and liner performance for the final cover system designed for AP-3/4 in comparison to a CCR Unit final cover system (§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 (Georgia Solid Waste Management Regulations, Section. 391-3-4-.10(7) and 40 CFR 257.102(d)). Additional detail on the cover equivalency calculations can be found in Appendix G.

4.3 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-3/4 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.

Golder 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 H).

4.4 Final Cover Anchor Trench

The ClosureTurf™ final cover system is designed to cover the AP-3/4 waste limits following consolidation and capping of the CCR material. Appendix I 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. Surface Water Management

4.5 General

The surface water management system for combined unit AP-3/4 includes several controls for limiting peak stormwater discharge flows from the closed CCR unit, including attenuation storage in the three (3) designed stormwater retention ponds, minimizing erosion from high velocity flow, and conveying stormwater safely below access roads and other structures. Golder has developed a comprehensive calculation package for the stormwater management system as outlined in Section 5.2 below that consists of a series of ditches, ponds, and culverts.

4.6 Surface Water Management Analysis

Appendix J includes a comprehensive surface water management calculation for the closed Unit AP-3/4 conditions. The calculation package estimates run-off for a variety of storm events, ranging from the 2-year, 24-

hour storm event to the 1,000-year, 24-hour storm events under final development conditions for the unit to the stormwater management system's three stormwater retention ponds. Type II rainfall distribution was used for all modeling efforts, and all structures were ultimately designed based on the discharge from the 100-year, 24-hour storm event.

Details of the hydrologic analysis are included in the calculation narrative provided in Appendix J. Three separate watersheds were delineated to route to Detention Pond 1, Detention Pond 2, and Detention Pond 3 located around the outer extents of the closed unit. Terraces and perimeter channels are designed to convey stormwater from the closed CCR unit surface, and are designed to maintain sufficient freeboard under the design storm; these are designed as either HydroTurf or riprap lined channels. Similarly, Armorflex articulated concrete block (ACB) lined downslope groin channels are designed to convey stormwater from the perimeter at the northeast and southeast down to Retention Pond 2. Stilling basins are designed to dissipate energy from flow traveling along the north and south downslope channels (Table 13 of Appendix J). Culverts for road and berm crossings have also been designed and are summarized in Table 15 in the calculation package included in Appendix J.

The detention ponds were also analyzed based on the proposed design for each storm event. The three retention ponds were designed to provide run-off storage capacity, as well as for the attenuation of floods. Detention Pond 1 was designed with an outfall structure to convey stormwater to Detention Pond 2. Detention Ponds 2 and 3 were designed with outfall structures for stormwater discharge. Maximum outflows from Detention Ponds 1, 2, and 3 are estimated to be 12 cfs, 30 cfs, and 9 cfs respectively for the 100-year, 24-hour design storm.

In August 2020, the outlet structure for Detention Pond 3 was further evaluated with regards to outflow and time required to drain the contributing watershed areas following a rain event. Detention Pond 3 is intended to serve as an attenuation pond in order to route surface water following a rain event away from the capped unit, and during normal conditions does not contain surface water. Based on this analysis, presented in Appendix K of this Engineering Report, the low level outlet configuration has been modified to include six 3-inch low level orifice outlets to serve in combination with the upper two stage overflow weirs in the Pond 3 outlet structure. This modification was evaluated to provide for a combination of controlled stormwater conveyance attenuation and control to protect downstream infrastructure, and resulting in drainage times under 1 day for all storm events up to and including the 100-year, 24-hour storm event. The outlet design is included in the Closure Plan Drawings in Part A Section 9 of this Permit Application.

5.0 CLOSING

This engineering design report provides a summary of key calculations for the design of the final closure for Plant McDonough's AP-2 and Combined Unit AP-3/4 Inactive CCR Impoundments. Appendices to this report include calculations as discussed herein.

GOLDER ASSOCIATES INC.

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6.0 REFERENCES

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APPENDIX A

Material Properties Calculations Package

	SUBJECT: Geotechnical Material Property Package	
GOLDER	**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 ($Su/\sigma'v$) 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.

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
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.

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.

<table>
<tr><th colspan="5">Summary of Selected Strength Parameters for Alluvial Soils</th></tr>
<tr><th rowspan="2">Material</th><th rowspan="2">Unit Weight (pcf)</th><th colspan="2">Drained</th><th>Undrained</th></tr>
<tr><th>ϕ' (deg)</th><th>c' (psf)</th><th>Su (tsf)</th></tr>
<tr><td>Alluvial Soil</td><td>115</td><td>28</td><td>50</td><td>0.5</td></tr>
</table>

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.

<table>
<tr><th colspan="9">Selected Strength Parameters</th></tr>
<tr><th colspan="2" rowspan="2">Material</th><th rowspan="2">Total Unit Weight (pcf)</th><th colspan="2">Drained Strength</th><th colspan="2">Undrained Strength</th><th colspan="2">Post-Earthquake Strength</th></tr>
<tr><th>ϕ' (deg)</th><th>c' (psf)</th><th>ϕ (deg)</th><th>c (tsf)</th><th>ϕ (deg)</th><th>c (tsf)</th></tr>
<tr><td rowspan="2">Sluiced CCR</td><td>Above GW</td><td rowspan="2">90</td><td rowspan="2">24</td><td rowspan="2">0</td><td rowspan="2">12</td><td rowspan="2">0.05</td><td>24</td><td>0</td></tr>
<tr><td>Below GW</td><td colspan="2">Su/σ'_{vo} = 0.08 (min 100 psf)</td></tr>
<tr><td colspan="2">Stacked / Compacted CCR</td><td>110</td><td>30</td><td>0</td><td>24</td><td>0.18</td><td>30</td><td>0</td></tr>
<tr><td colspan="2">Fill Soils</td><td>125</td><td>30</td><td>100</td><td colspan="2">Su = 1.0 tsf</td><td>30</td><td>100</td></tr>
<tr><td colspan="2">Upper Residuum</td><td>125</td><td>30</td><td>50</td><td colspan="2">Su/σ'_{vo} = 0.65</td><td>30</td><td>50</td></tr>
<tr><td colspan="2">Lower Residuum</td><td>125</td><td>30</td><td>100</td><td colspan="2">Su = 1.5 tsf</td><td>30</td><td>100</td></tr>
<tr><td colspan="2">Alluvium</td><td>115</td><td>28</td><td>50</td><td colspan="2">Su = 0.5 tsf</td><td>28</td><td>50</td></tr>
</table>

8.0 ATTACHMENTS

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

9.0 REFERENCES

AMEC (2010). "Report of Dam Safety Assessment of Coal Combustion Surface Impoundments Plant McDonough, Smyrna, GA." December 2010

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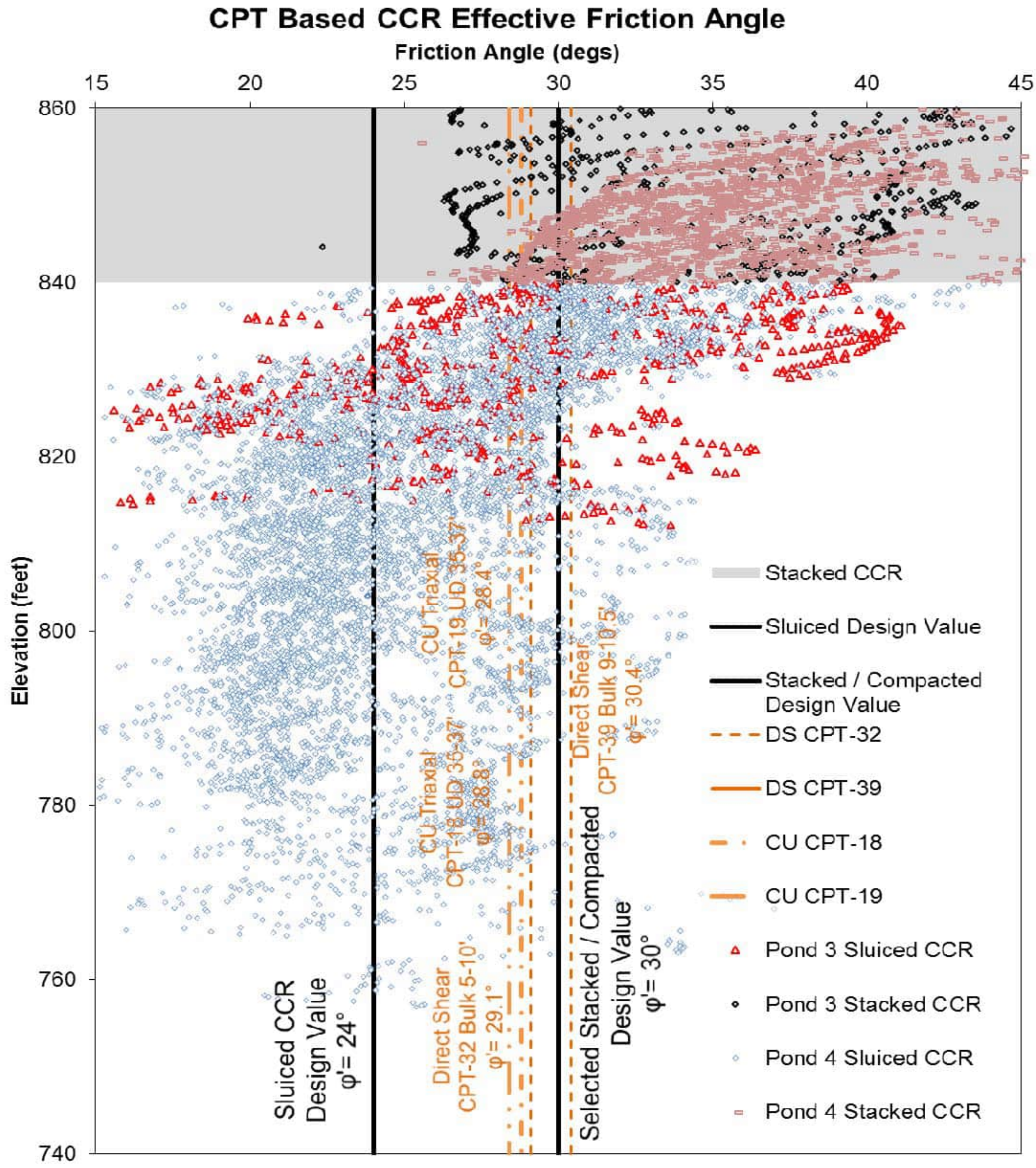
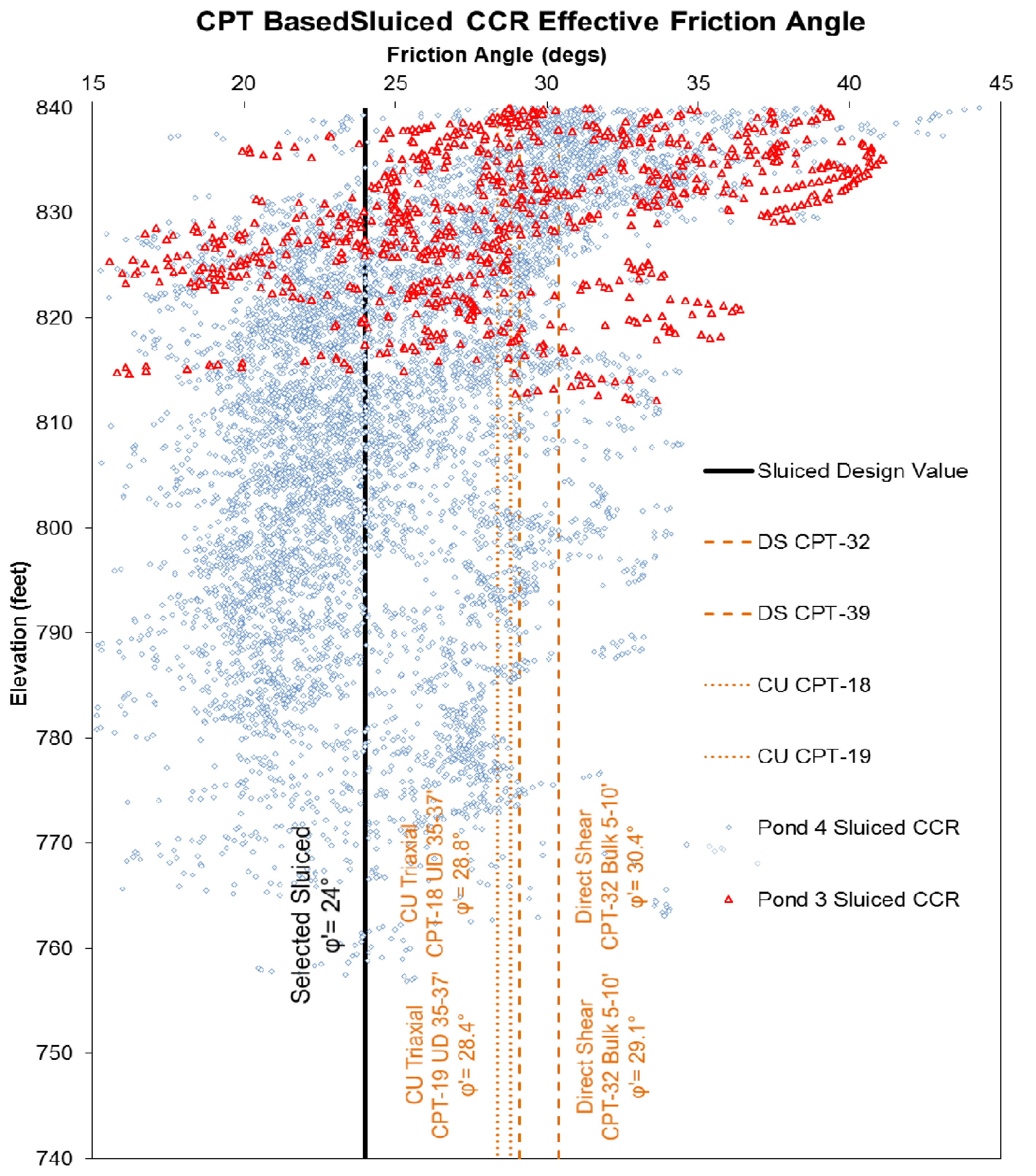
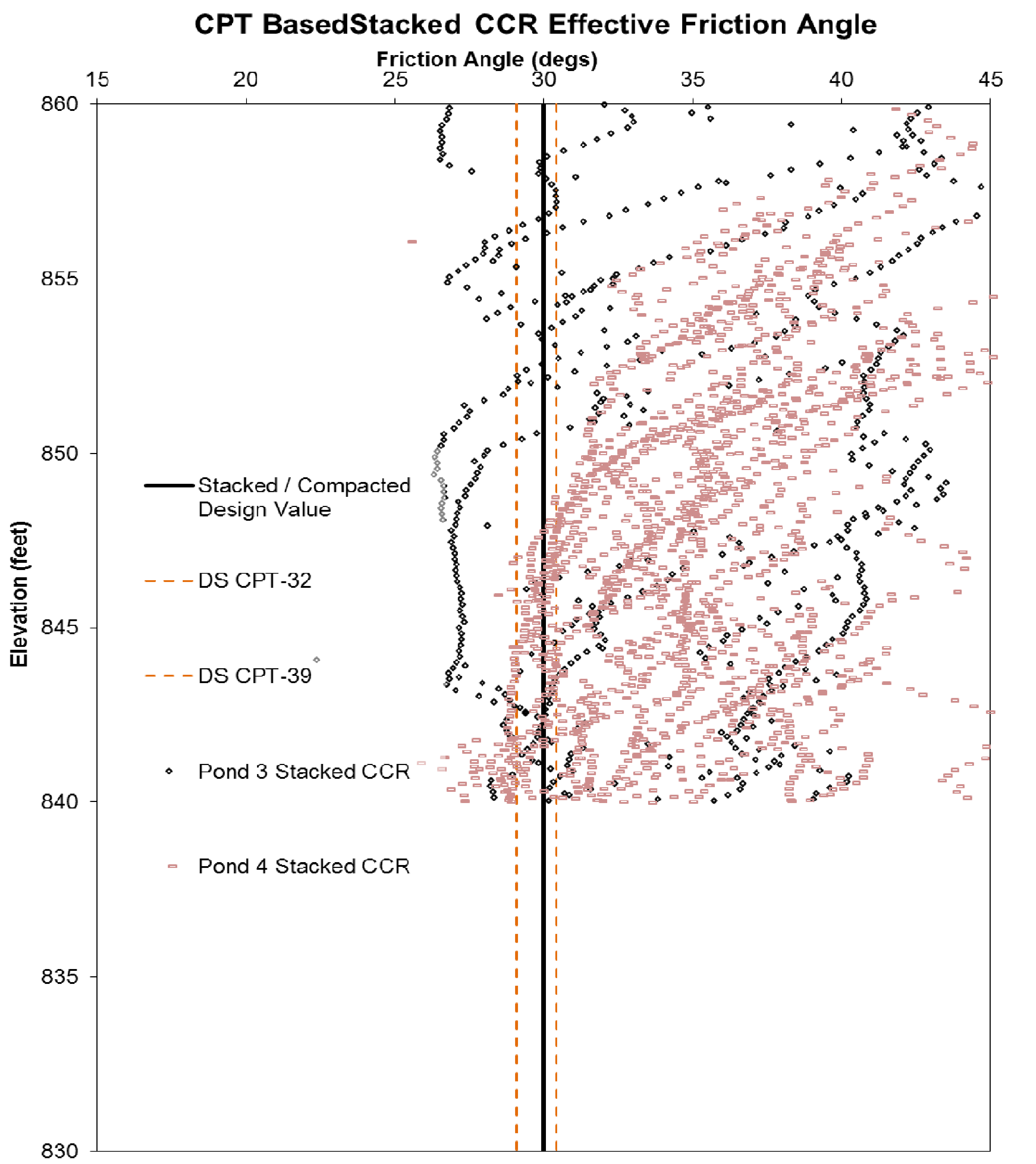


GOLDER Golder Associates Inc.	SCALE AS SHOWN DATE Apr 2016 DESIGN DRAWN WRP		**Peak Friction Angle Based On CPT Correlation and Lab Results (Direct Shear & Triaxial)** **Correlation Developed By Mayne And Kulhawy 1996**
Figure 1	CHECK TPC		
PROJECT No. 1539180	REV. 1	REVIEW GLH	



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**
		DATE	Dec 2015	
		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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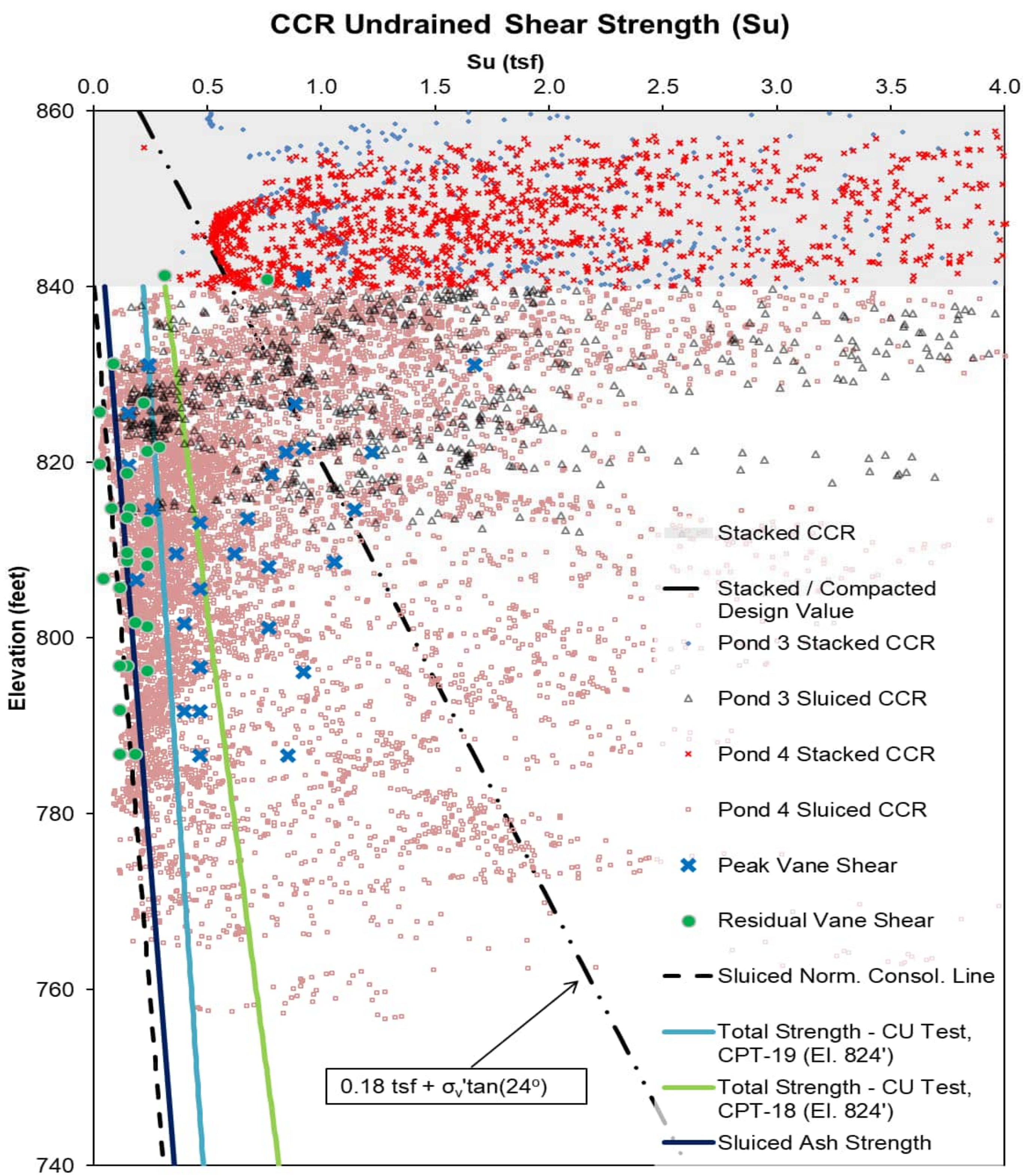
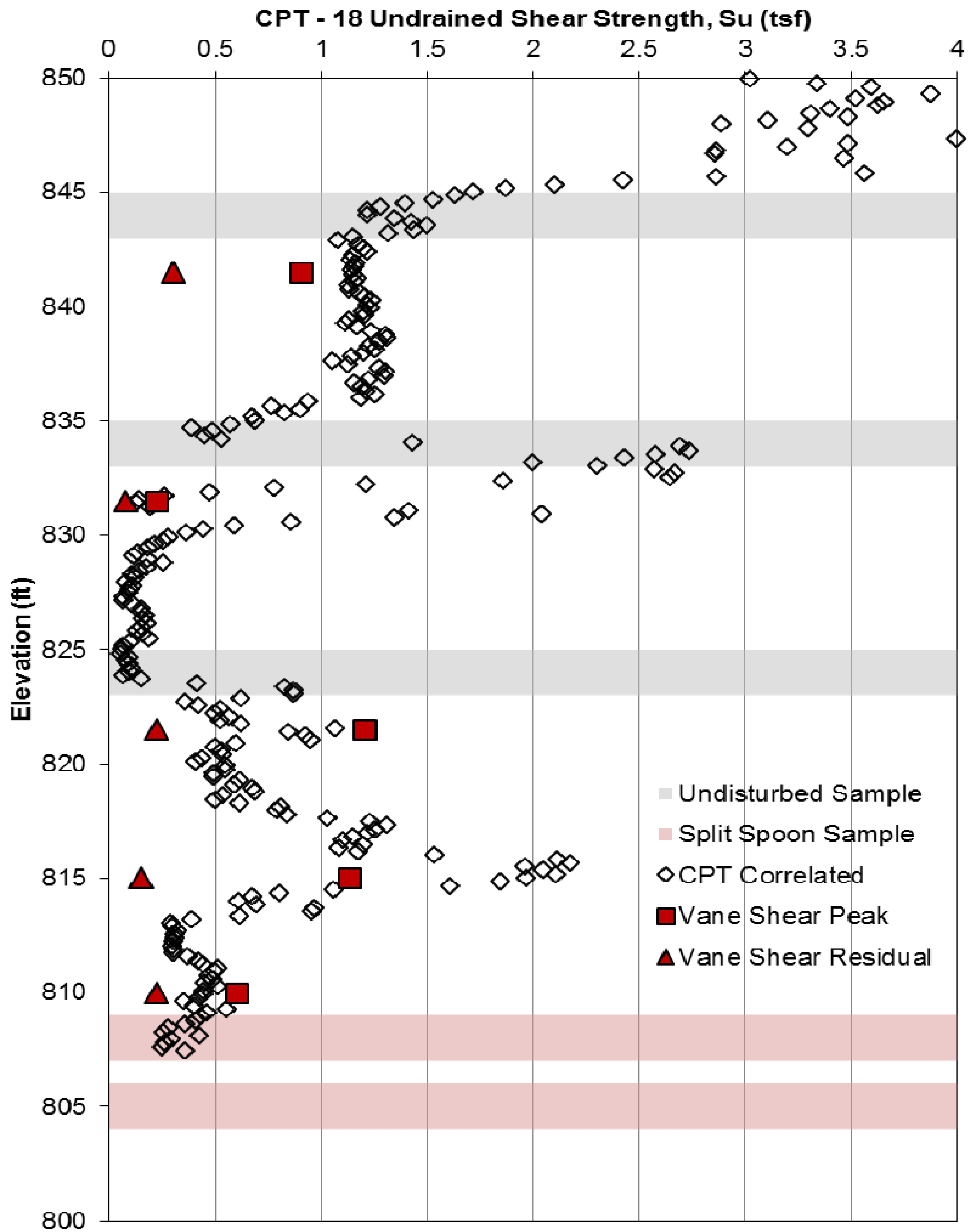
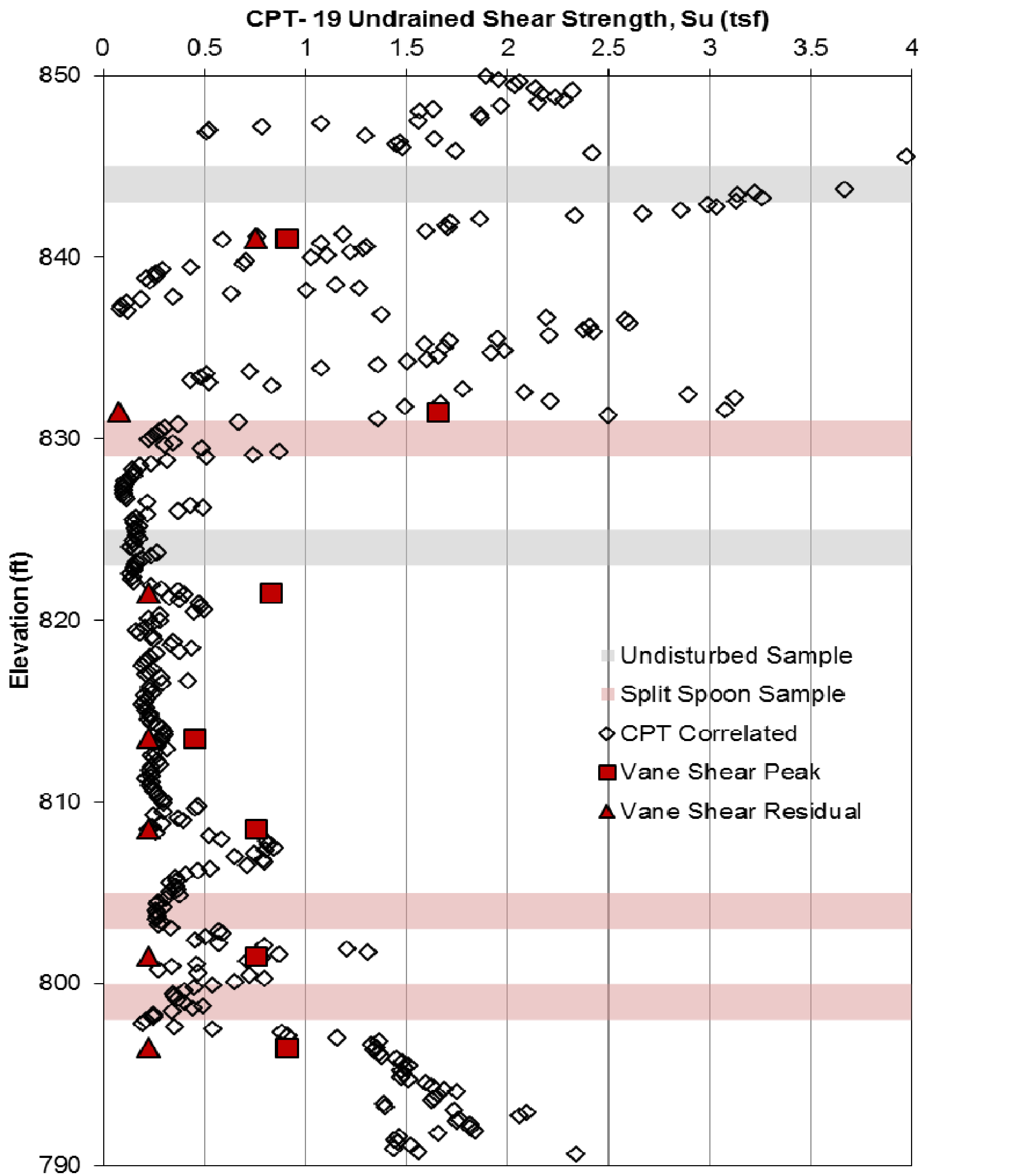
	DESIGN		
	DRAWN	WRP	
Figure 3	CHECK	TPC	
PROJECT No. 1539180 REV. 1	REVIEW	GLH	



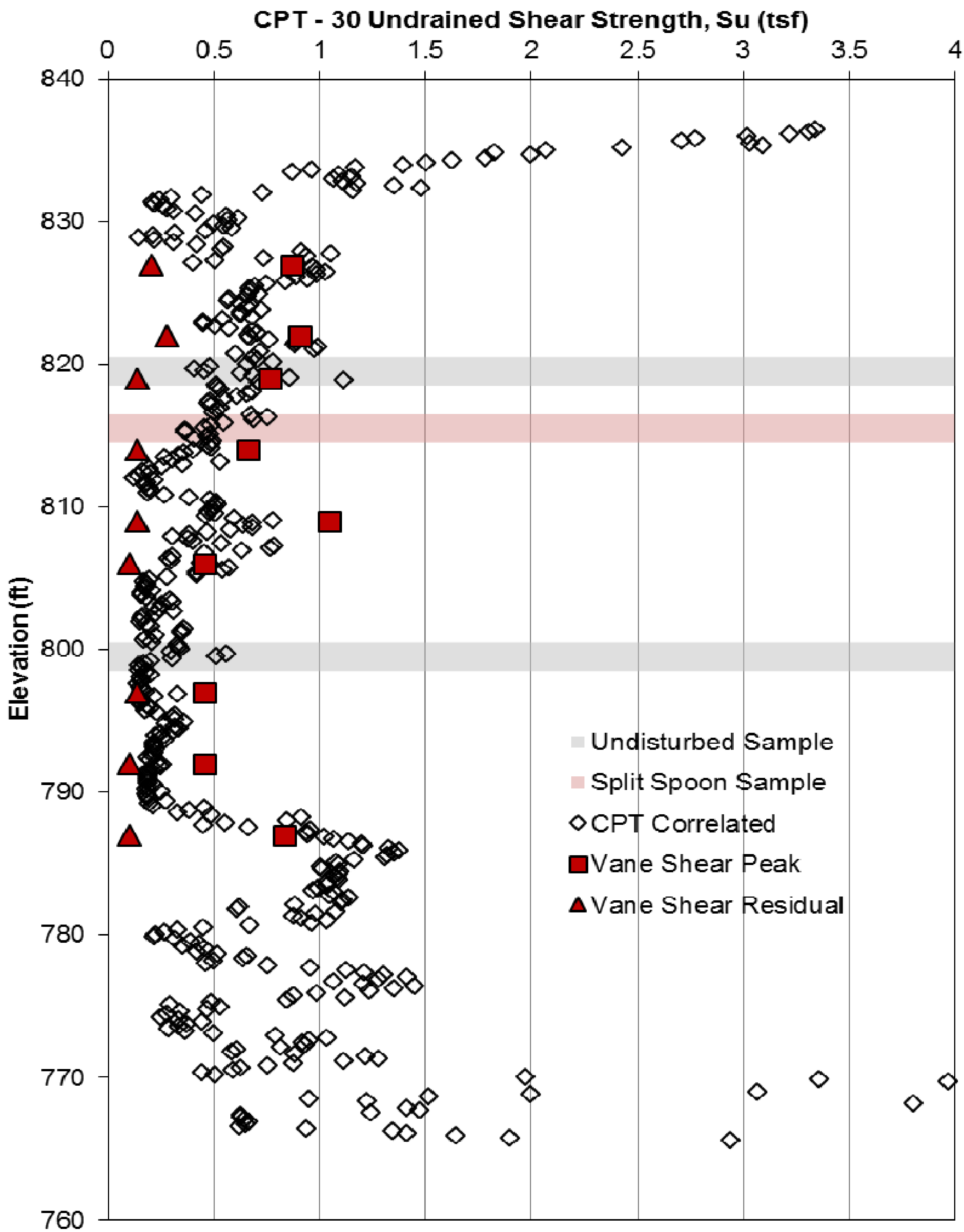
GOLDER Golder Associates Inc.		SCALE AS SHOWN DATE Jul 2018 DESIGN - DRAWN WRP CHECK JGM REVIEW GLH	**Measured and Correlated Undrained Shear Strengths** **CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
Figure 4a			
PROJECT No. 1539180	REV. 1		



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	



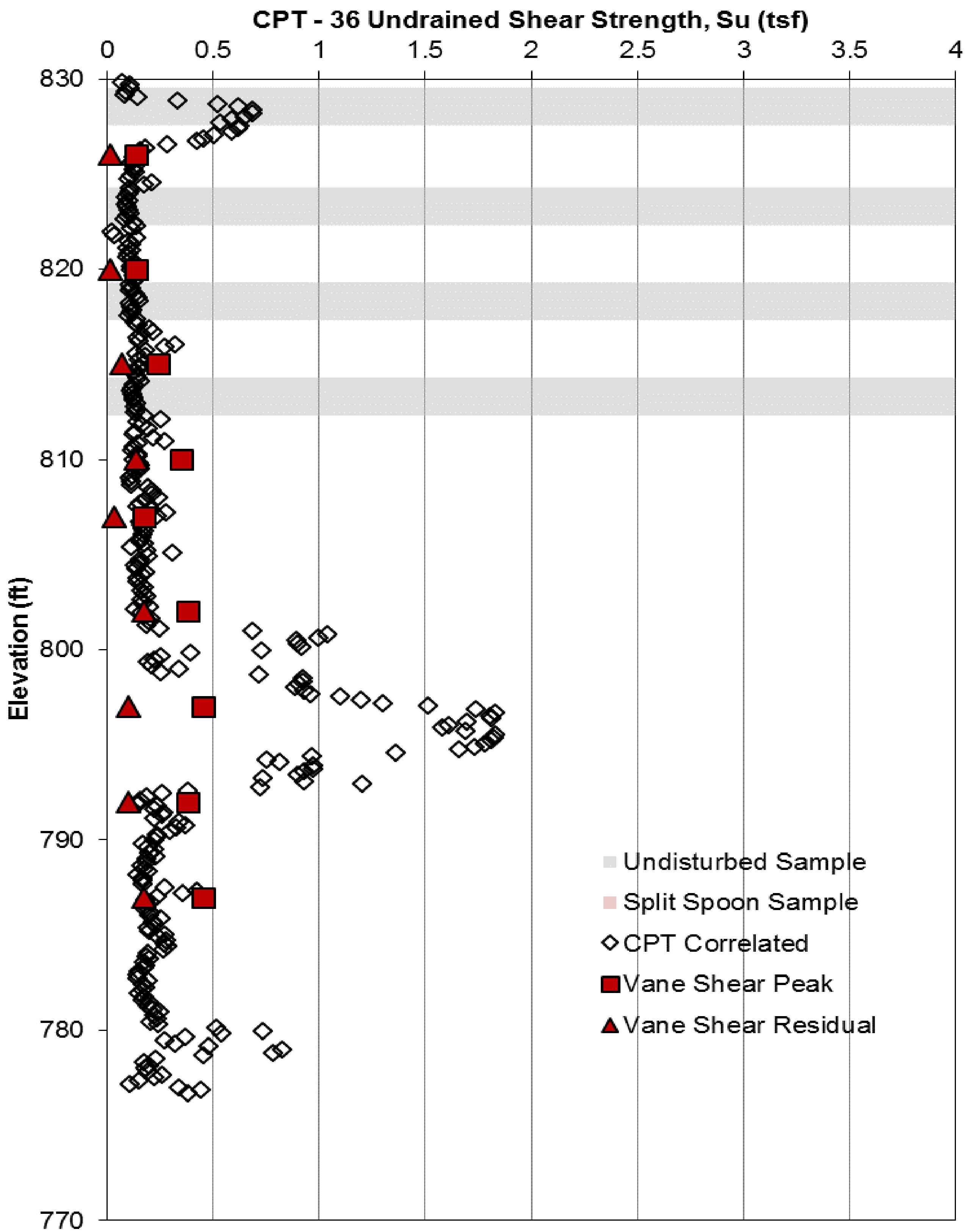
GOLDER Golder Associates Inc.		SCALE	AS SHOWN	**Measured and Correlated Undrained Shear Strengths at CPT-19** **CPT Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
		DATE	Apr 2016	
		DESIGN	-	
		DRAWN	WRP	
Figure 4c		CHECK	JGM	
PROJECT No. 1539180	REV. 1	REVIEW	GLH	



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

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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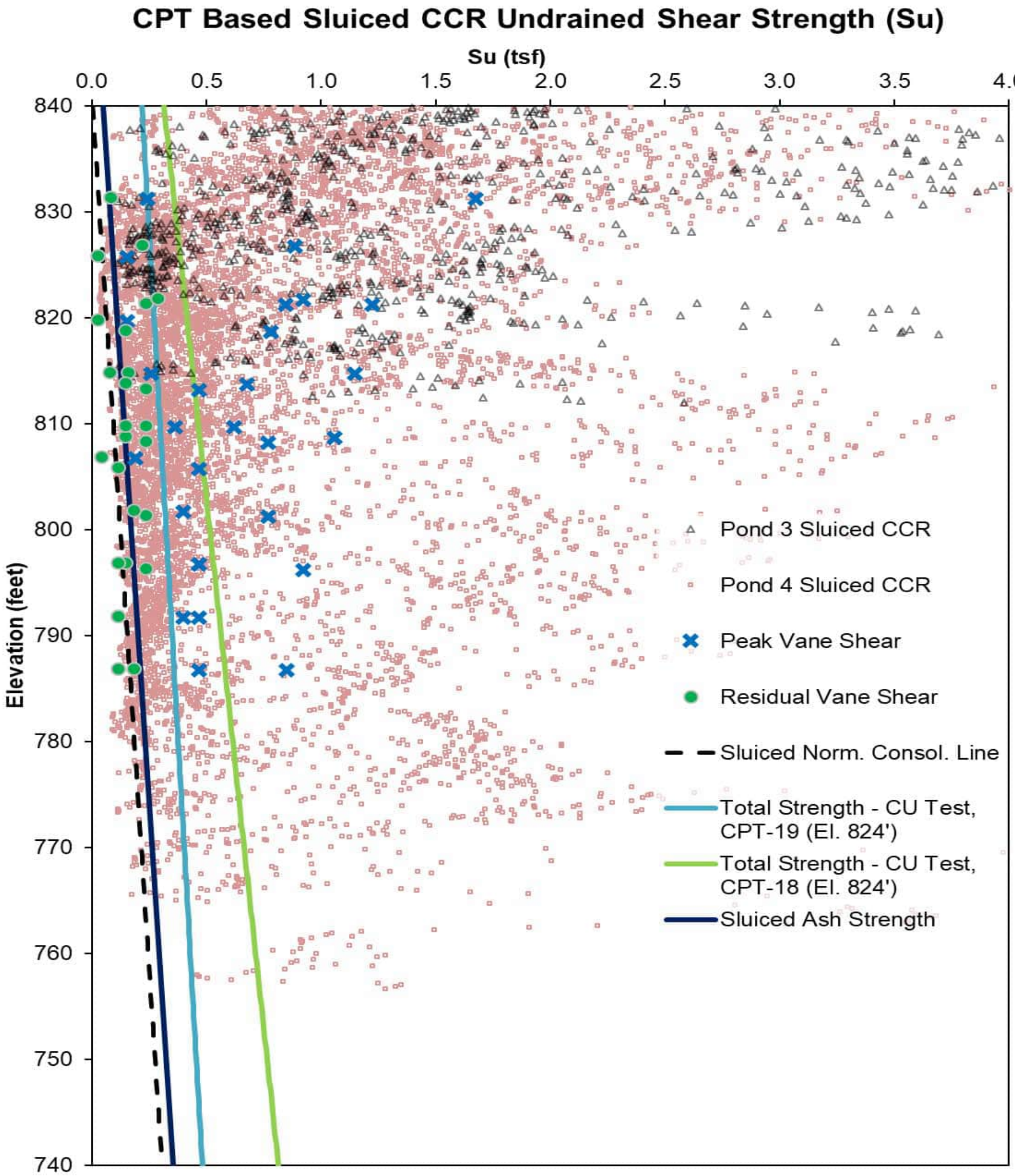
PROJECT No. 1539180 REV. 1	REVIEW	GLH	



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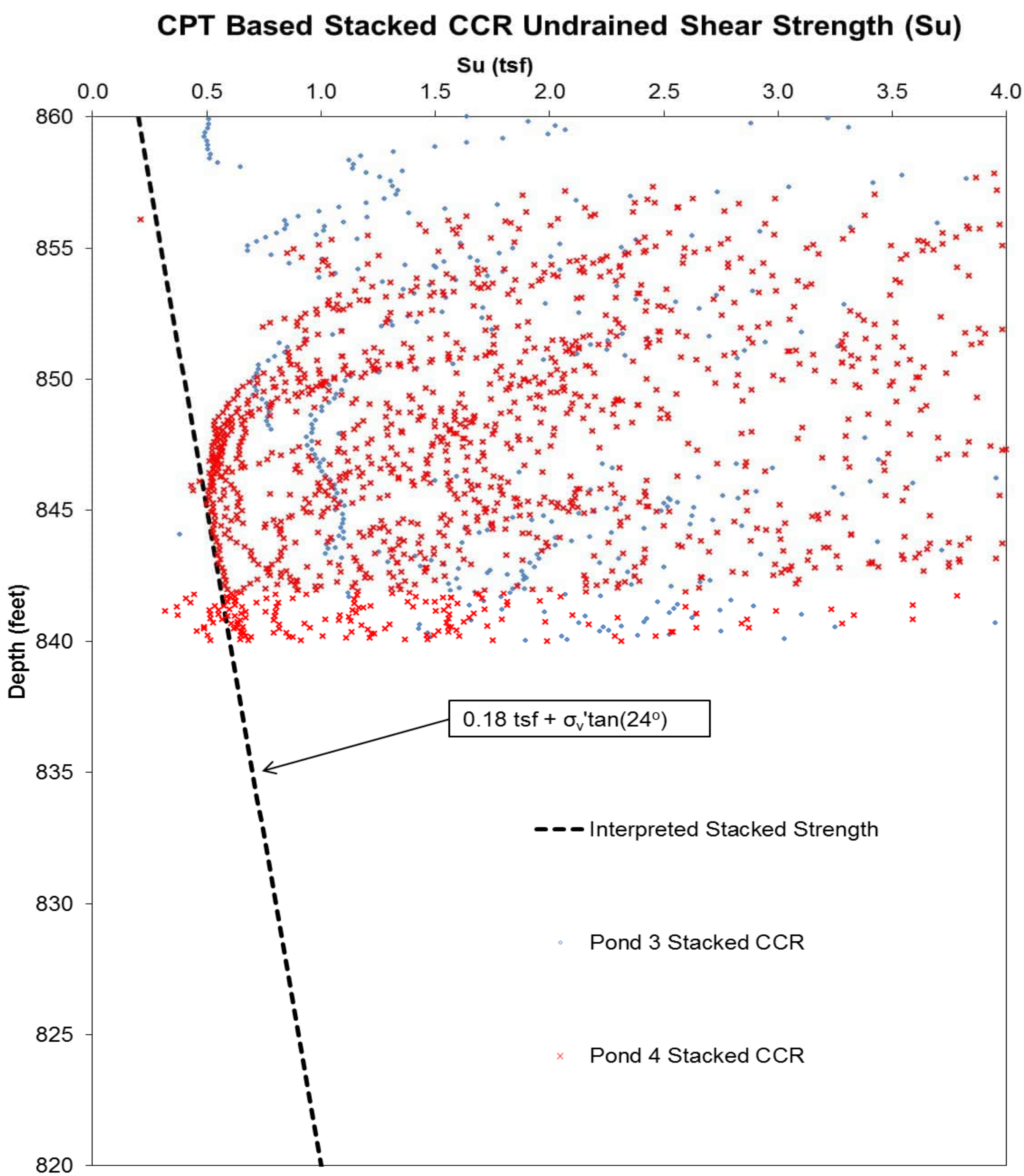
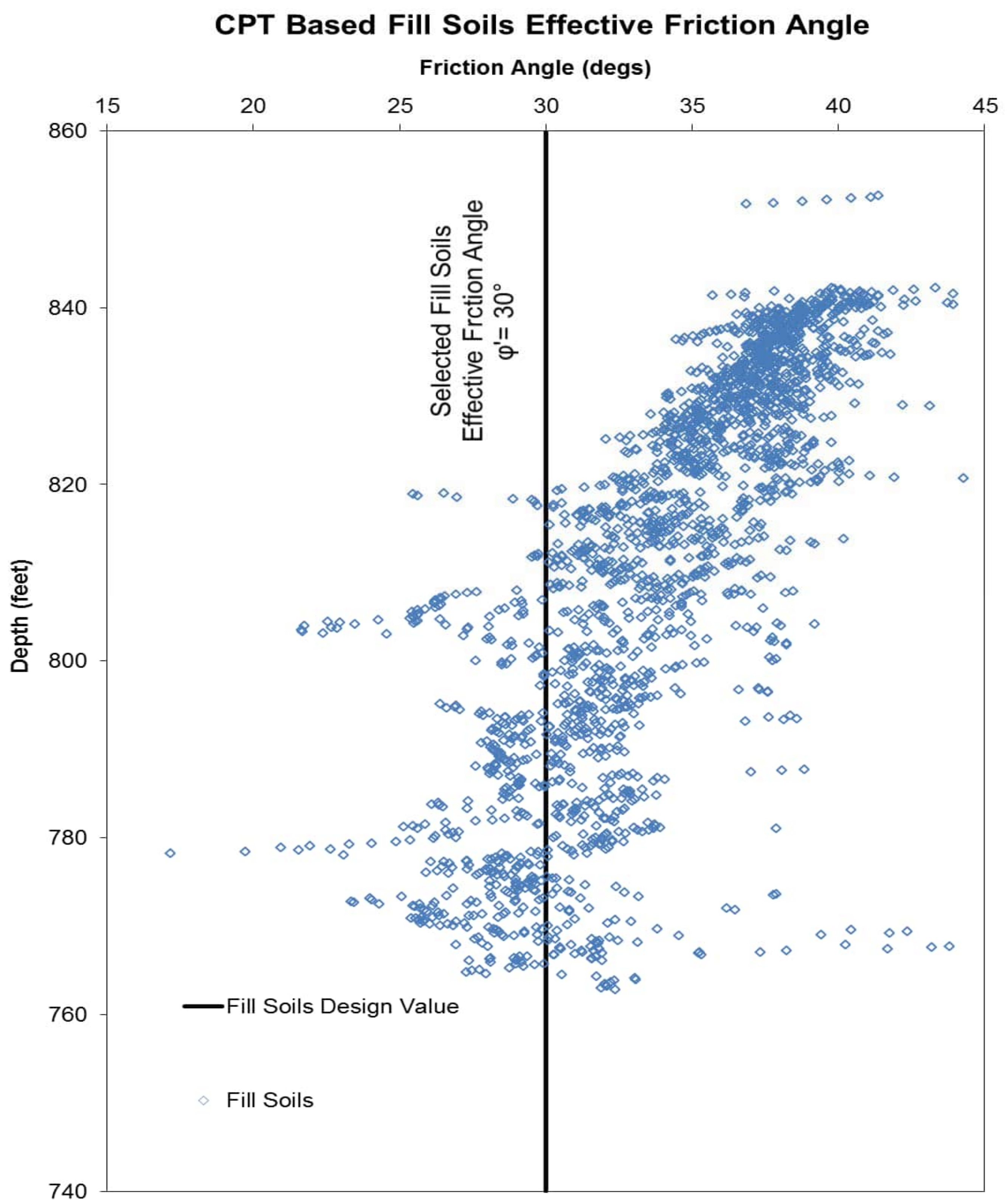
		DATE Jul 2018	
		DESIGN	
		DRAWN WRP	**Correlation Based On Robertson et al. 1986 and Lunne et al. 1996**
Figure 5		CHECK TPC	
PROJECT No. 1539180	REV. 1	REVIEW GLH	



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**
		DATE	Dec 2015	
		DESIGN		
		DRAWN	WRP	
Figure 6		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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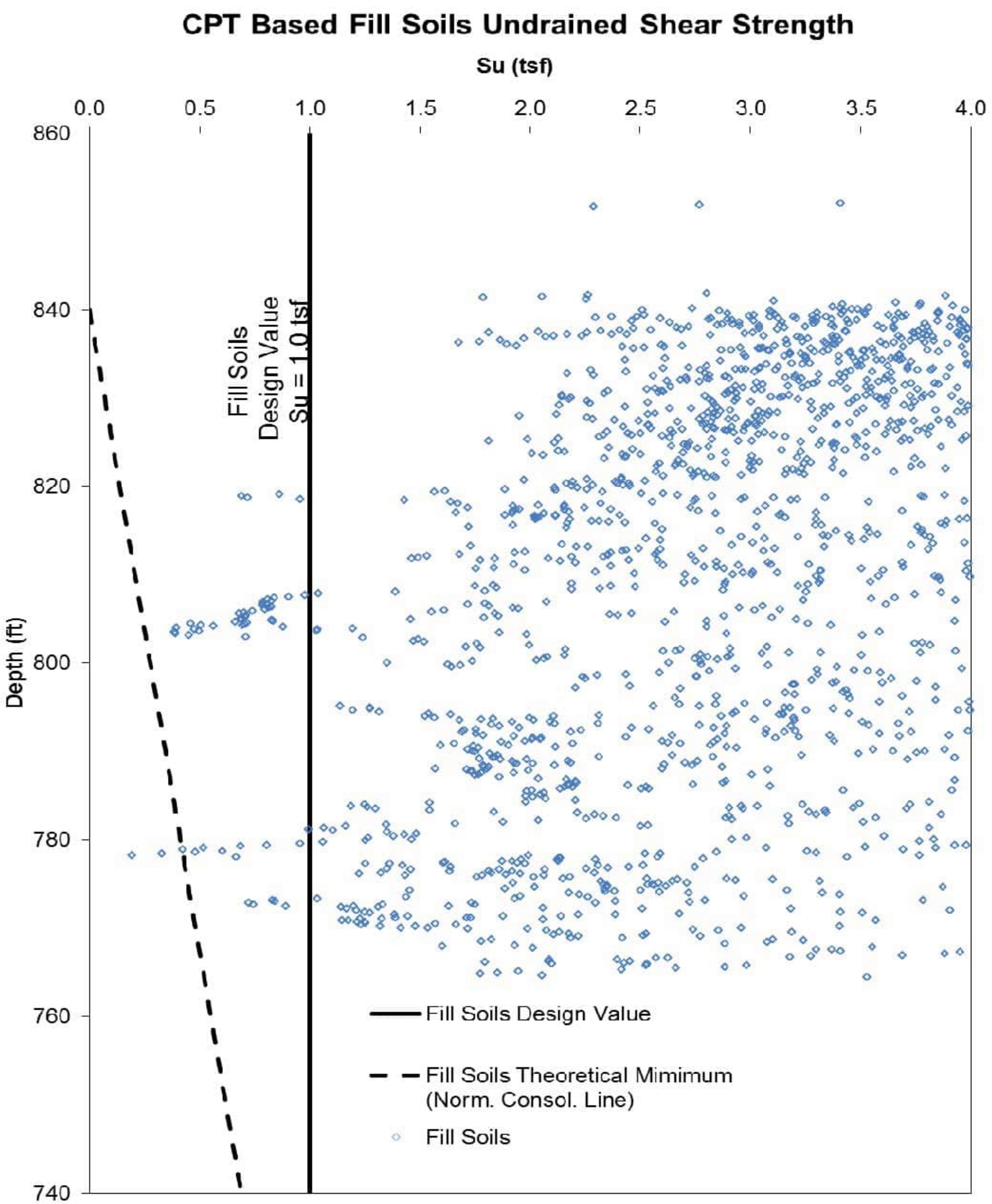
	DESIGN		
	DRAWN	WRP	
Figure 7	CHECK	TPC	
PROJECT No. 1539180 REV. 0	REVIEW	GLH	



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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		DRAWN	WRP	
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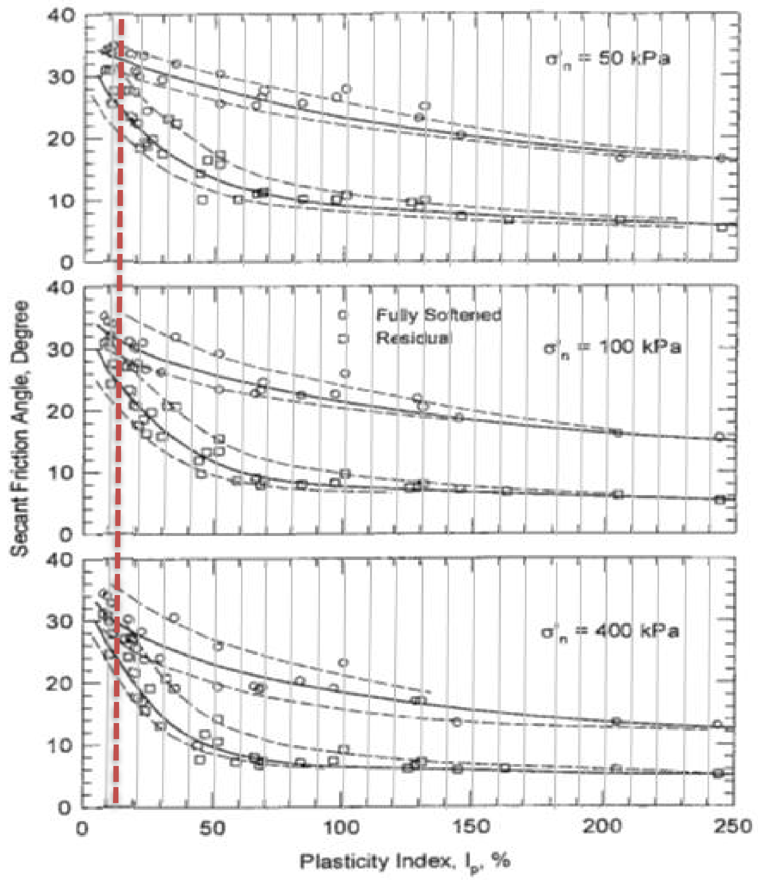
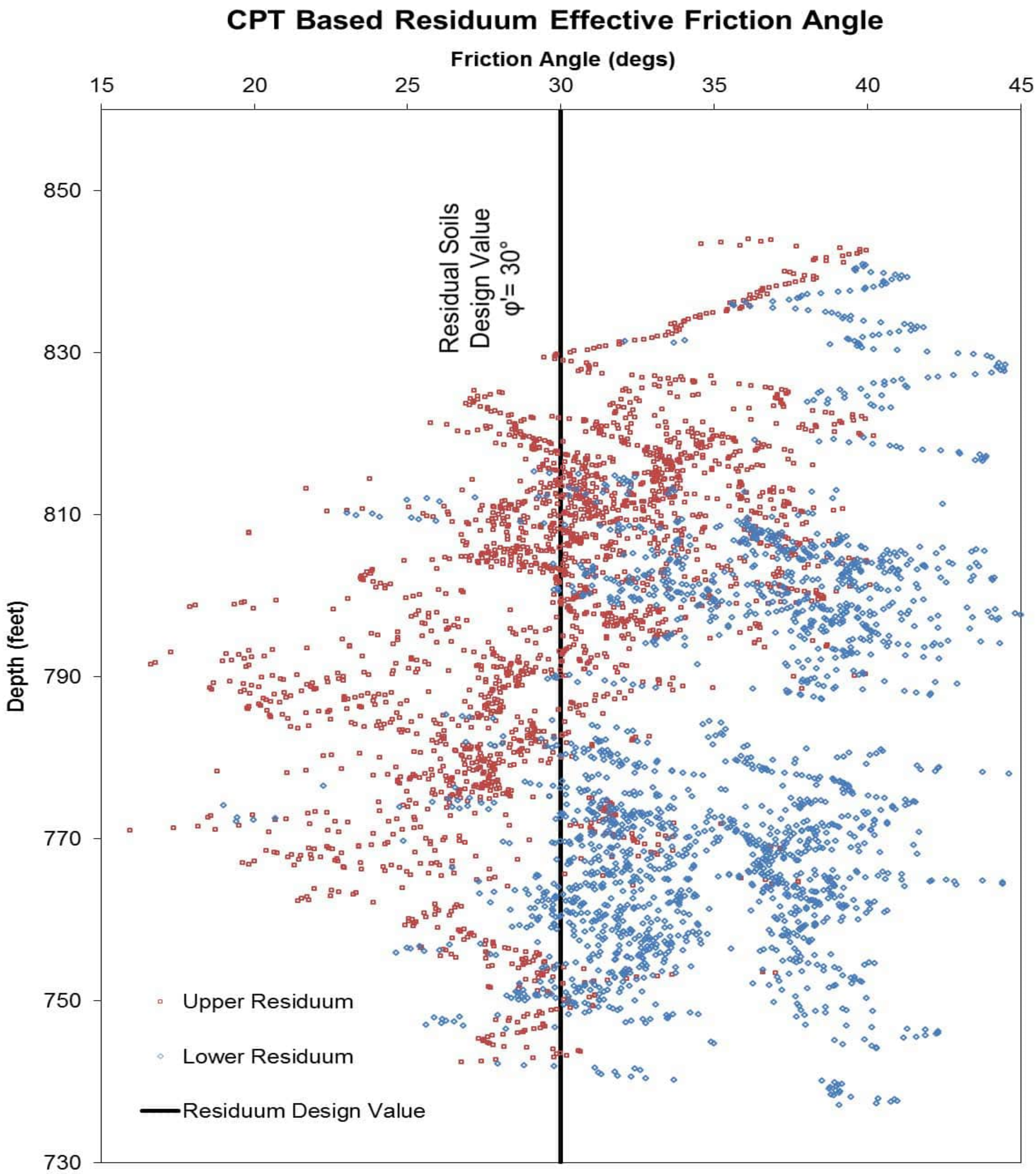
PROJECT No. 1539180	REV. 0	REVIEW	GLH	



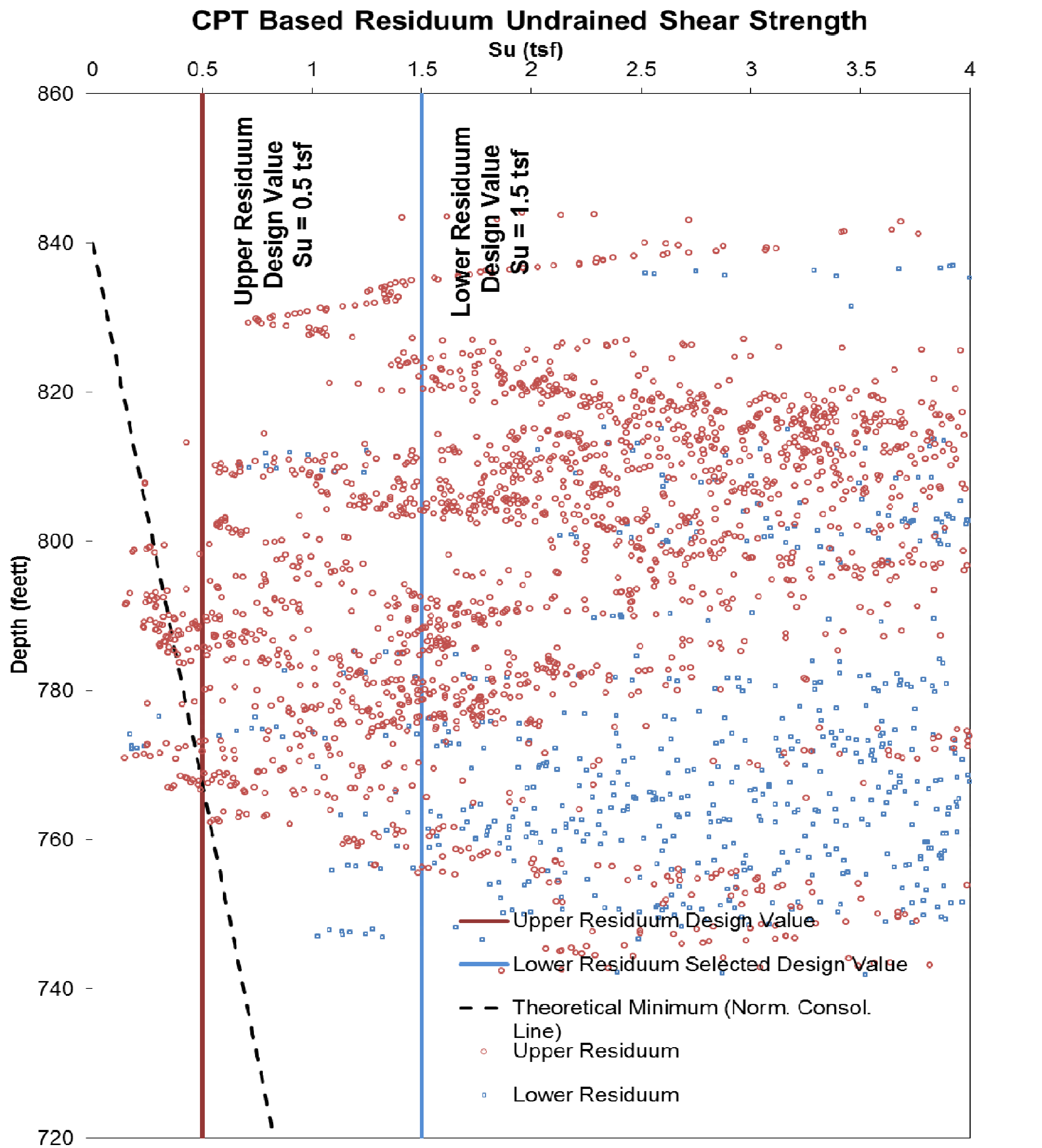
Note - Vertical Lines on Figure are Approximate and Not on Original

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

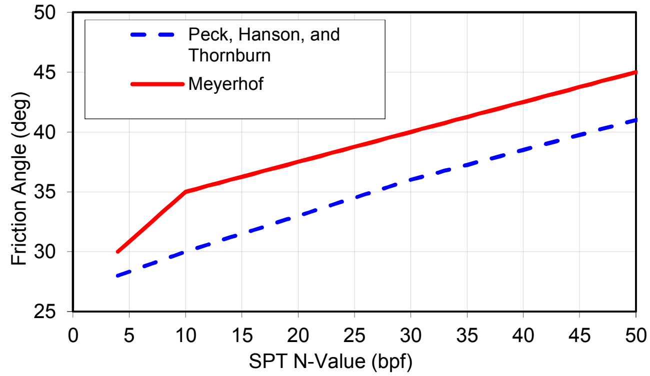


Figure 1: Graphical Representation of Table 1

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-2 and AP-3/4

LEGEND

EXISTING CONTOURS

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.

0 60 120
1" = 60' FEET

CLIENT
GEORGIA POWER COMPANY /
SOUTHERN COMPANY SERVICES

Southern Company

PROJECT
PLANT MCDONOUGH
CLOSURE PERMIT PLANS

TITLE
ASH POND 2 - STABILITY SECTIONS PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2018/02/15
DESIGNED	LJ
PREPARED	RMS
REVIEWED	JGM
APPROVED	GLH

PROJECT NO.
1777449

SHEET
1

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

Path: \\atlanta\Cadd\Southern Company\1777449 Plant Mcdonough State Permitting\PRODUCTION\CLOSURE PERMIT\ | File Name: 1777449 AP 01-02 CLOSURE PERMIT.dwg

LEGEND

- EXISTING CONTOURS
- PROPERTY BOUNDARY MARKERS/LIMITS
- FINAL CLOSURE CONTOURS & FINAL LIMITS OF ASH
- CLEAN CLOSURE CONTOURS
- EXISTING OVERHEAD ELECTRIC LINES IN ASH POND 3 & 4

REFERENCES

1. THE EXISTING TOPOGRAPHY AND CONTOUR ELEVATIONS WERE PROVIDED BY GEORGIA LAND DEPARTMENT AND METRO ENGINEERING AND SURVEYING CO, INC. THE DATE OF THE SURVEY PROVIDED AND SHOWN ON THIS SET OF PLANS IS 10-16-2012. REFER TO THE SURVEY DRAWING TITLED "TOPOGRAPHIC MAP PREPARED FOR GEORGIA POWER COMPANY PLANT MCDONOUGH - GEORGIA STATE PLANE WEST SURVEY FEET - DATE OF PHOTOGRAPHY 10-26-12. PROJECT NO. 13225 - 01-13-2013."

2. THE REVISED TOPOGRAPHY & CONTOUR ELEVATIONS WERE PROVIDED BY GEORGIA POWER LAND DEPARTMENT. THE DATA SHOWN IS AN UPDATE TO THE PLANS DONE ON 10-16-2012 & THE ONSITE CHANGES SINCE THAT 2012 SURVEY. THE REVISED SURVEY WAS DONE ON 1-12-2016 & MERGED WITH THE DATA ON 10-16-2012.

3. GEORGIA POWER COMPANY PLANT MCDONOUGH ASH PONDS - GEORGIA STATE PLANE WEST SURVEY FEET - DATE OF SURVEY 1-12-2016 - LAND ENG. PROJECT # 20160020.

0 150 300
1" = 150' FEET

CLIENT
GEORGIA POWER COMPANY /
SOUTHERN COMPANY SERVICES

Southern Company

PROJECT
PLANT MCDONOUGH
CLOSURE PERMIT PLANS

TITLE
ASH POND 3 & 4 - STABILITY SECTIONS PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2018/02/15
DESIGNED	LJ
PREPARED	RMS
REVIEWED	JGM
APPROVED	GLH

PROJECT NO.
177749

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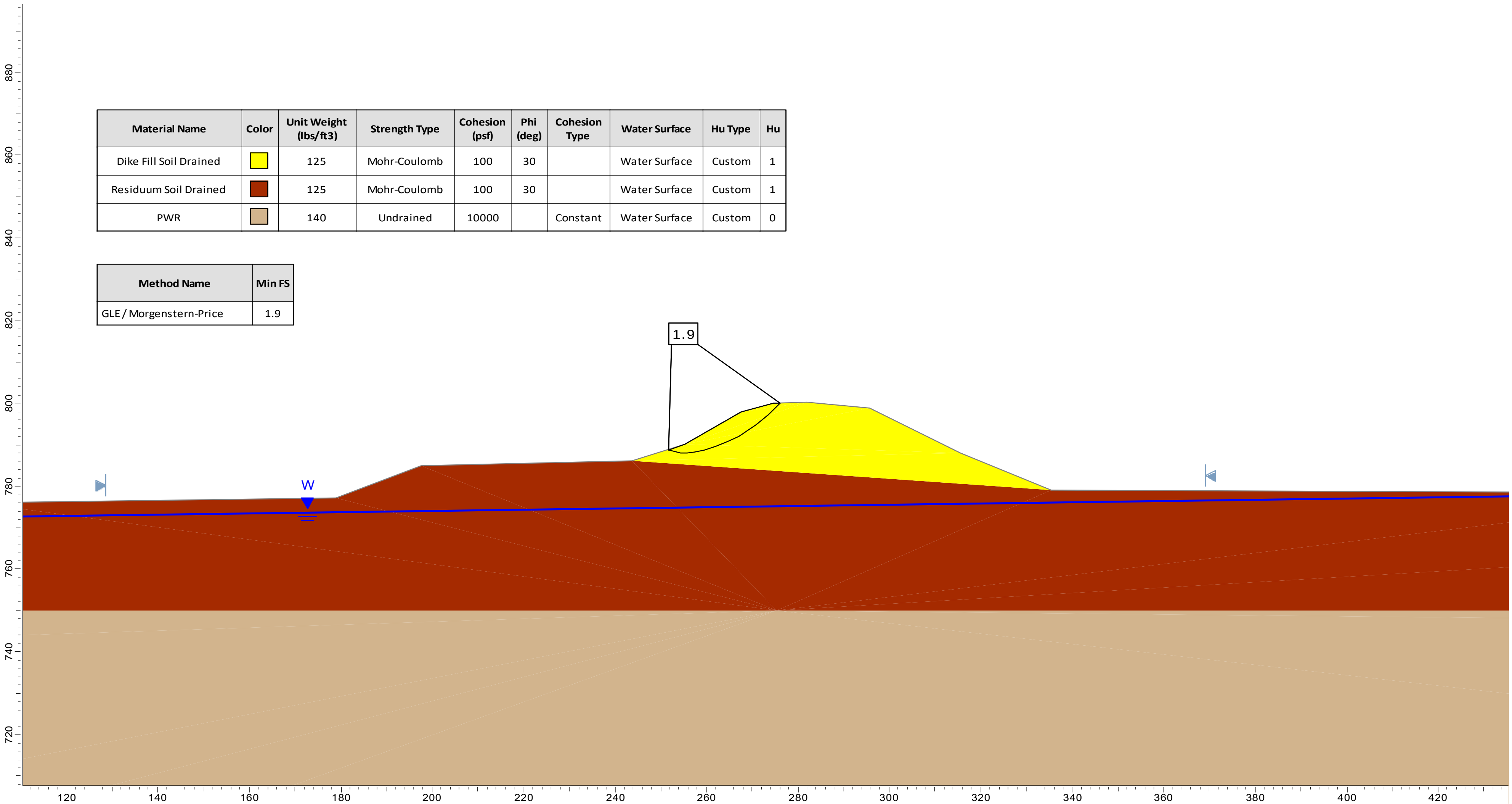
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Dike Fill Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Residuum Soil Drained		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
PWR		140	Undrained	10000		Constant	Water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

Method Name	Min FS
GLE / Morgenstern-Price	1.9



GOLDER

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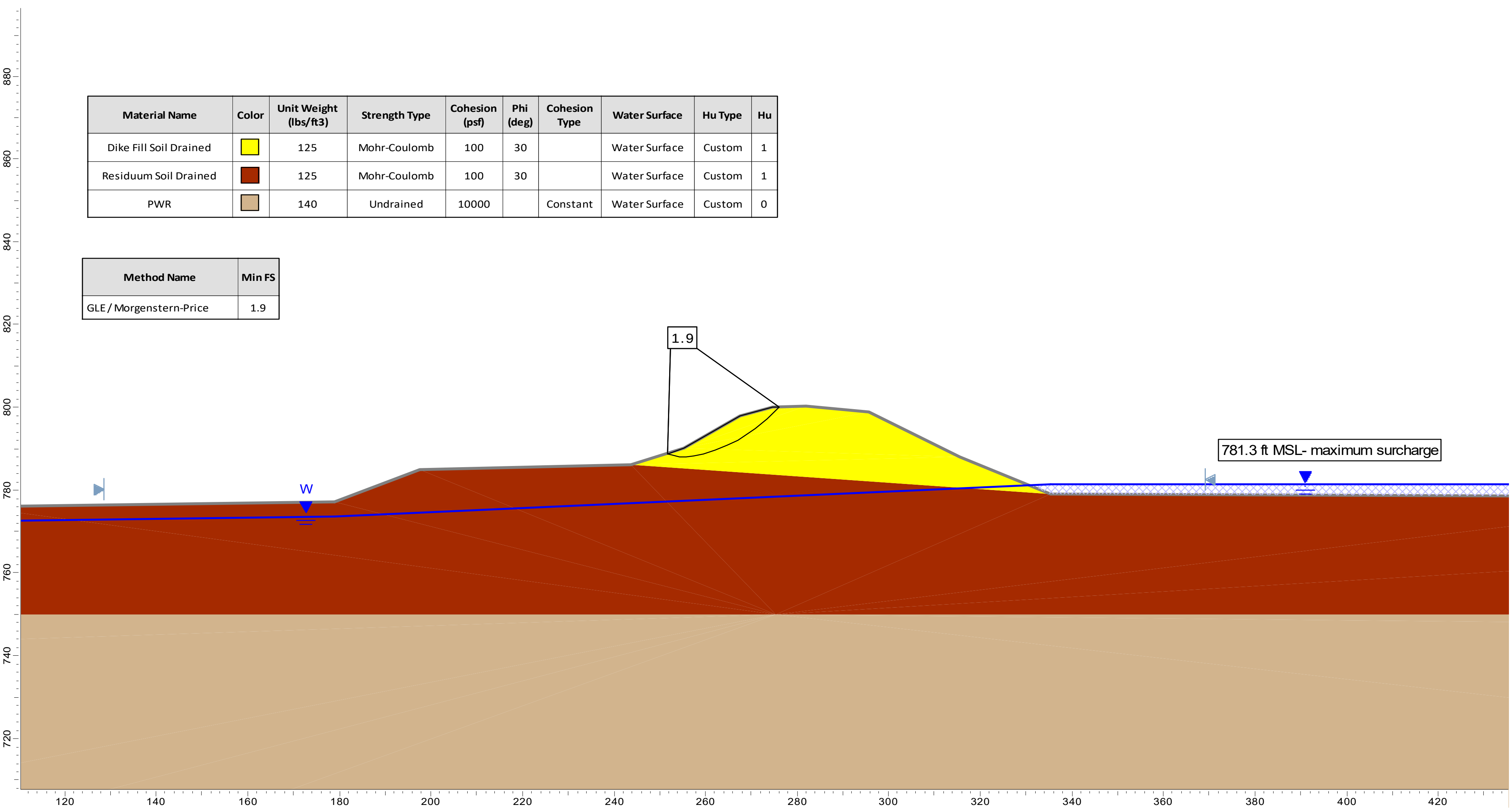
DATE	Jan 2018	TITLE	Section 2A-2A Storage Pool
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	Georgia Power Company
REVIEW	GLH	FIGURE	3(a)

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
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	1.9

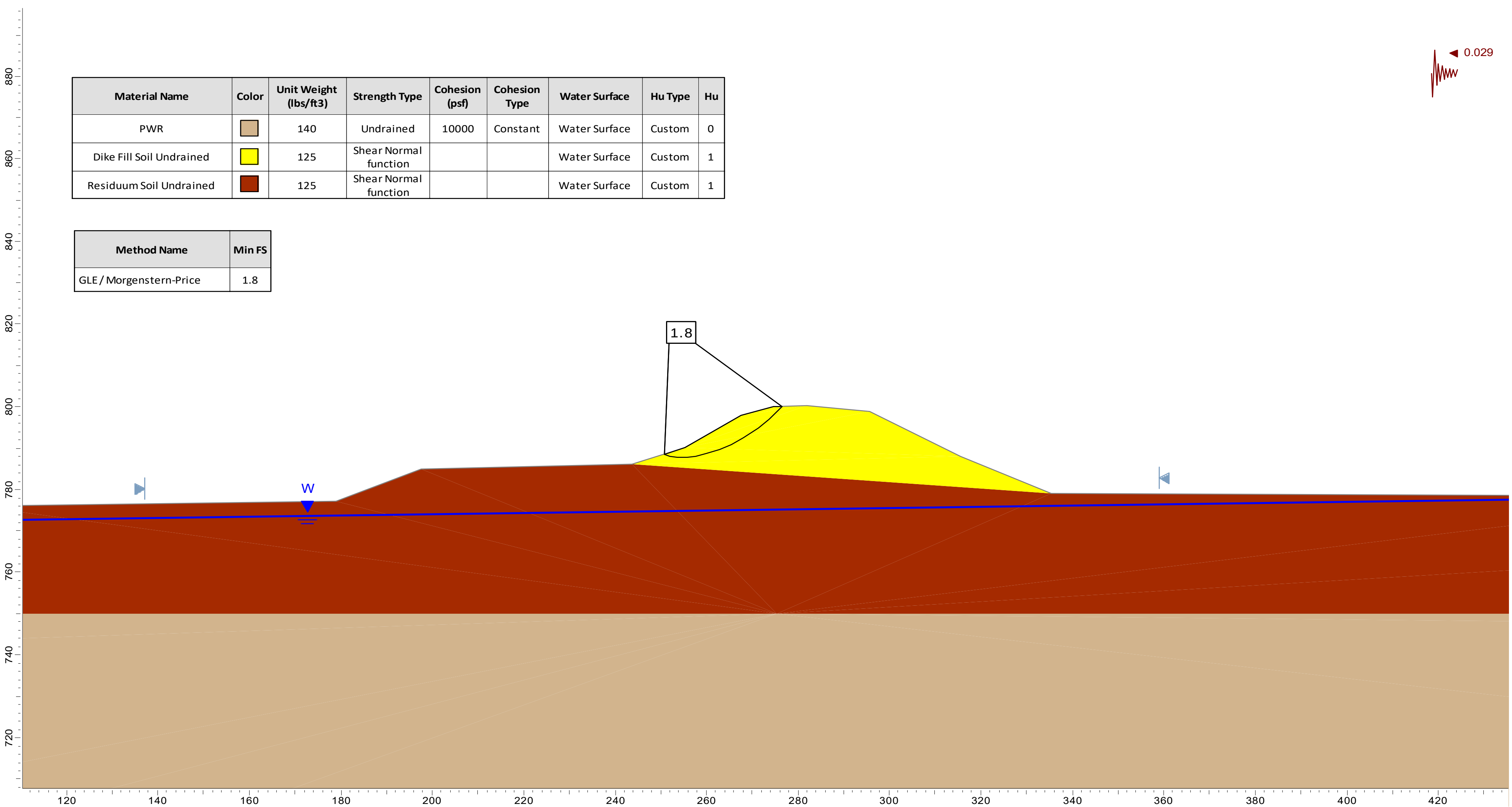
GOLDER

SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-2**
DATE	Jan 2018	TITLE	**Section 2A-2A Surcharge Pool**
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	**Georgia Power Company**
REVIEW	GLH	FIGURE	**3(b)**

FILE	STABILITY		
PROJECT No.	1777449	REV.	0

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

Method Name	Min FS
GLE / Morgenstern-Price	1.8



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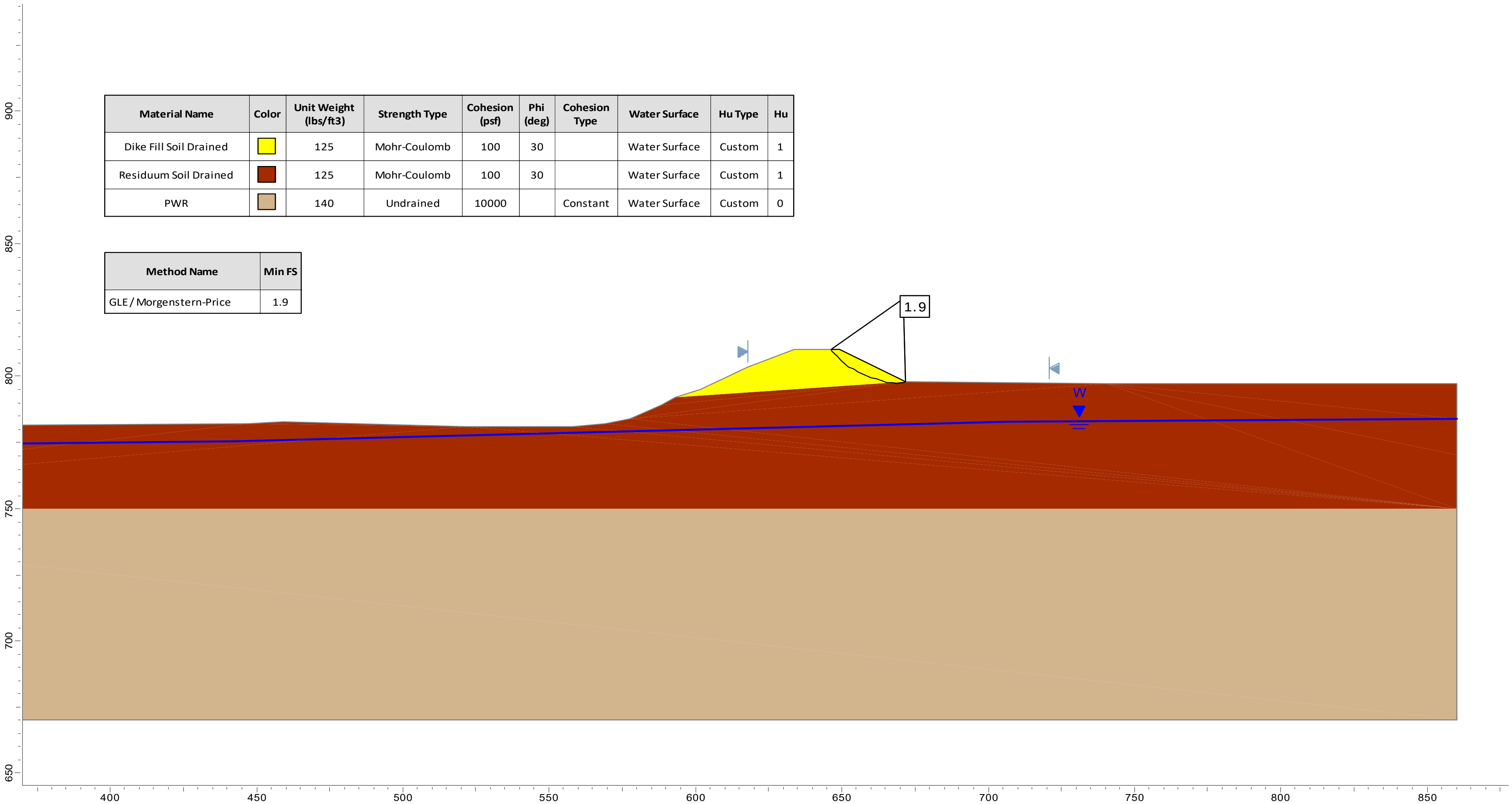
	DATE Jan 2018	TITLE	Section 2A-2A Seismic Screening
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	3(c)



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
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

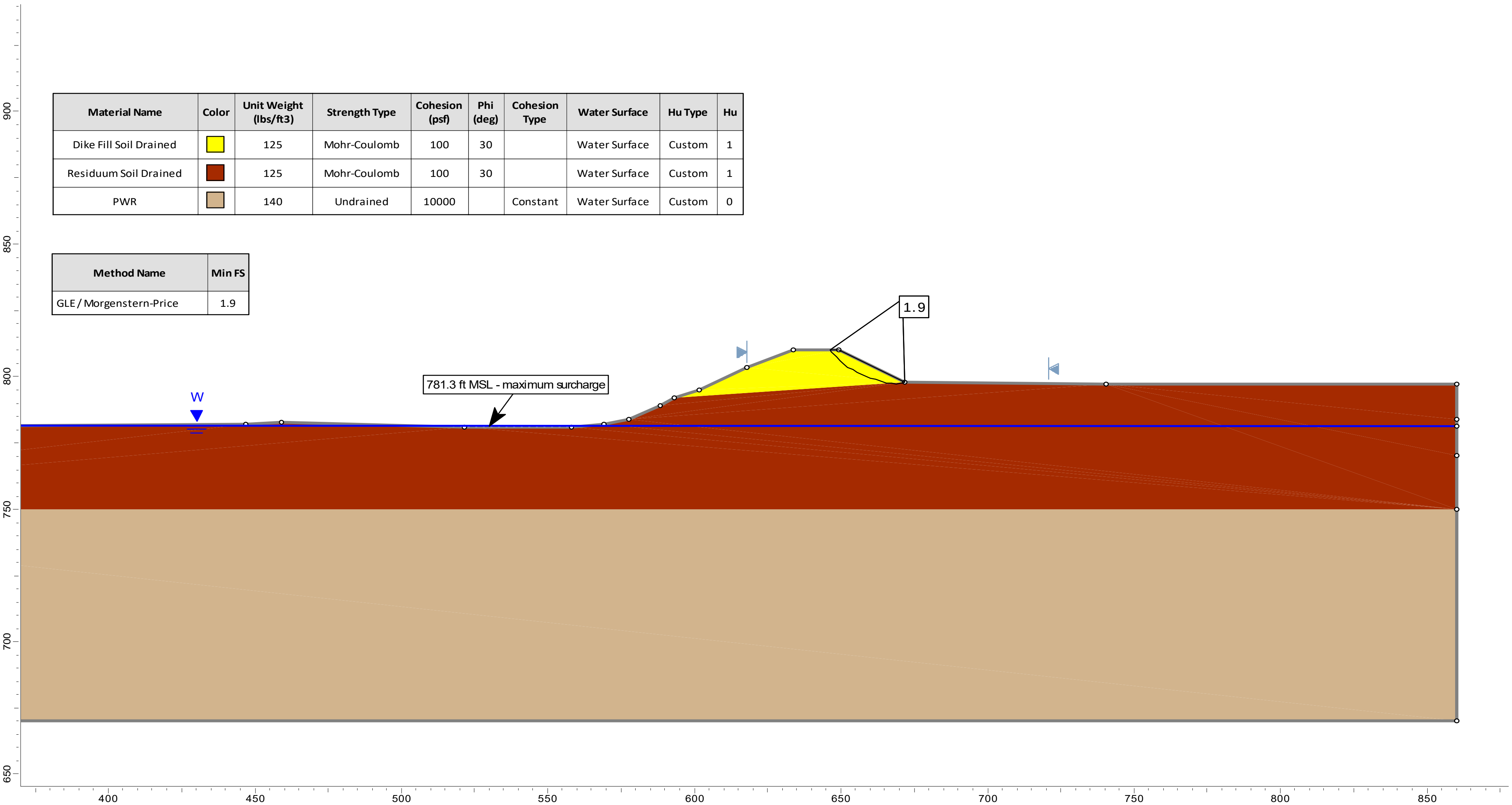
Method Name	Min FS
GLE / Morgenstern-Price	1.9

GOLDER

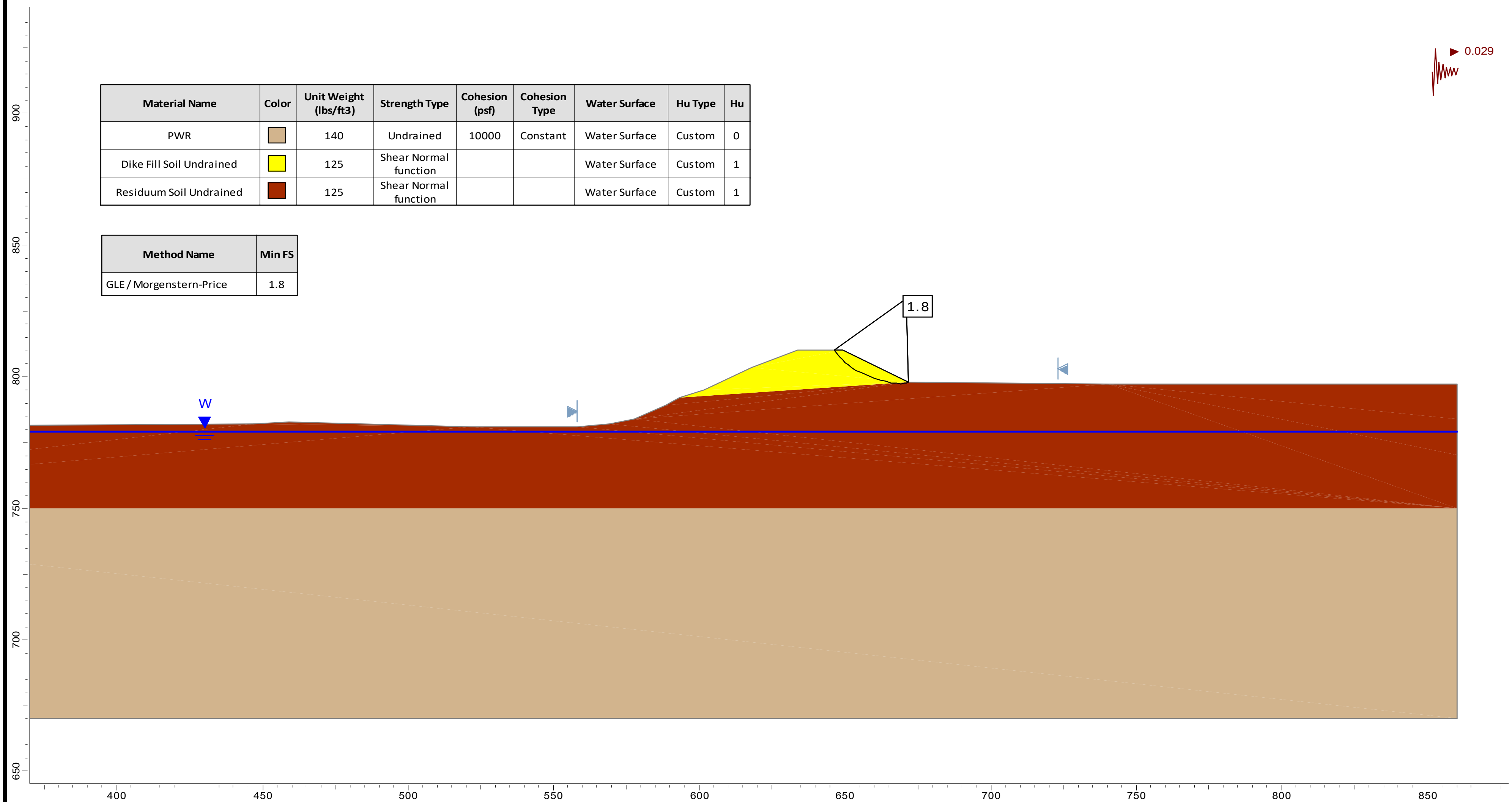
SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-2**
DATE	Jan 2018	TITLE	**Section 2B-2B Storage Pool**
MADE BY	LJ		
CAD	-		
FILE: STABILITY	CHECK: JGM	CLIENT	**Georgia Power Company**
PROJECT No.: 1777449	REV.: 0	REVIEW: GLH	FIGURE: **4(a)**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
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	1.9



GOLDER	SCALE AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-2
	DATE Jan 2018	TITLE	Section 2B-2B Surcharge Pool
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	4(b)



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

Method Name	Min FS
GLE / Morgenstern-Price	1.8

GOLDER

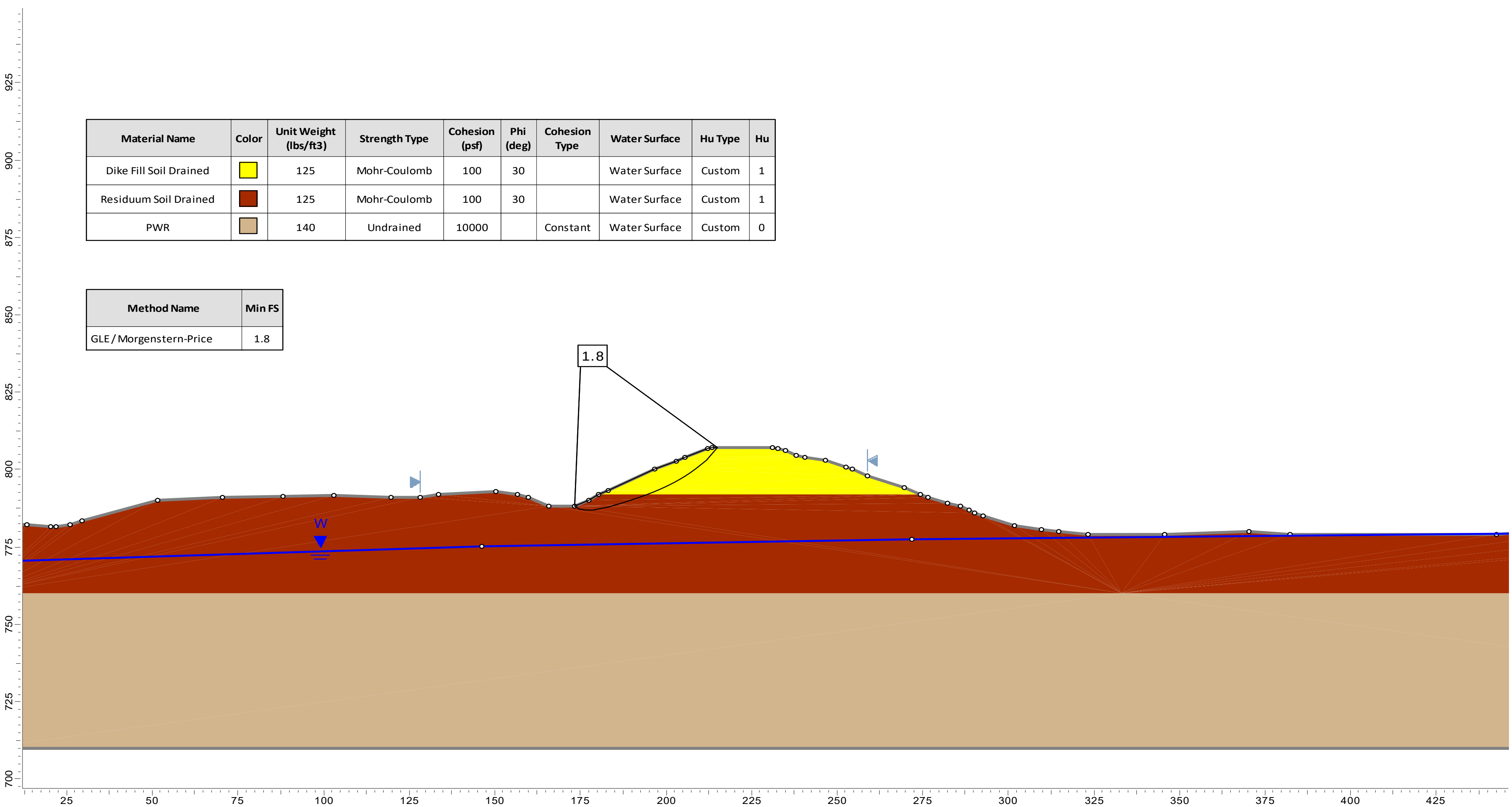
SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-2**
DATE	Jan 2018	TITLE	**Section 2B-2B Seismic Screening**
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	**Georgia Power Company**
REVIEW	GLH	FIGURE	**4(c)**

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
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

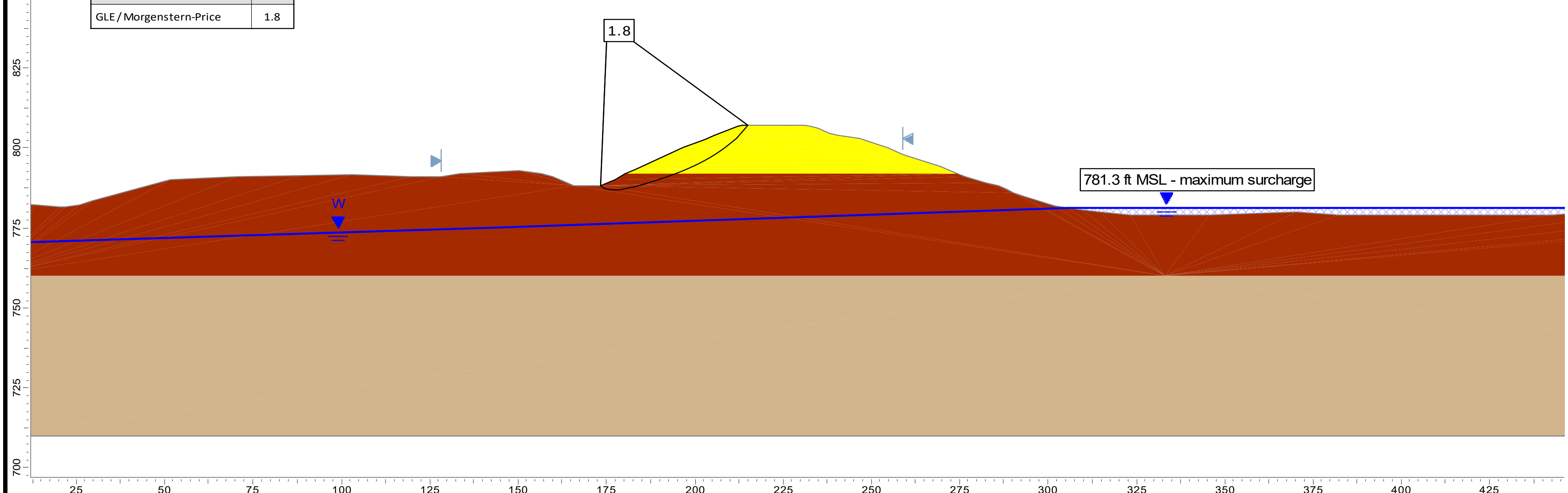
Method Name	Min FS
GLE / Morgenstern-Price	1.8



GOLDER	SCALE AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-2**
	DATE Jan 2018	TITLE	**Section 2C-2C Storage Pool**
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	**5(a)**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
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

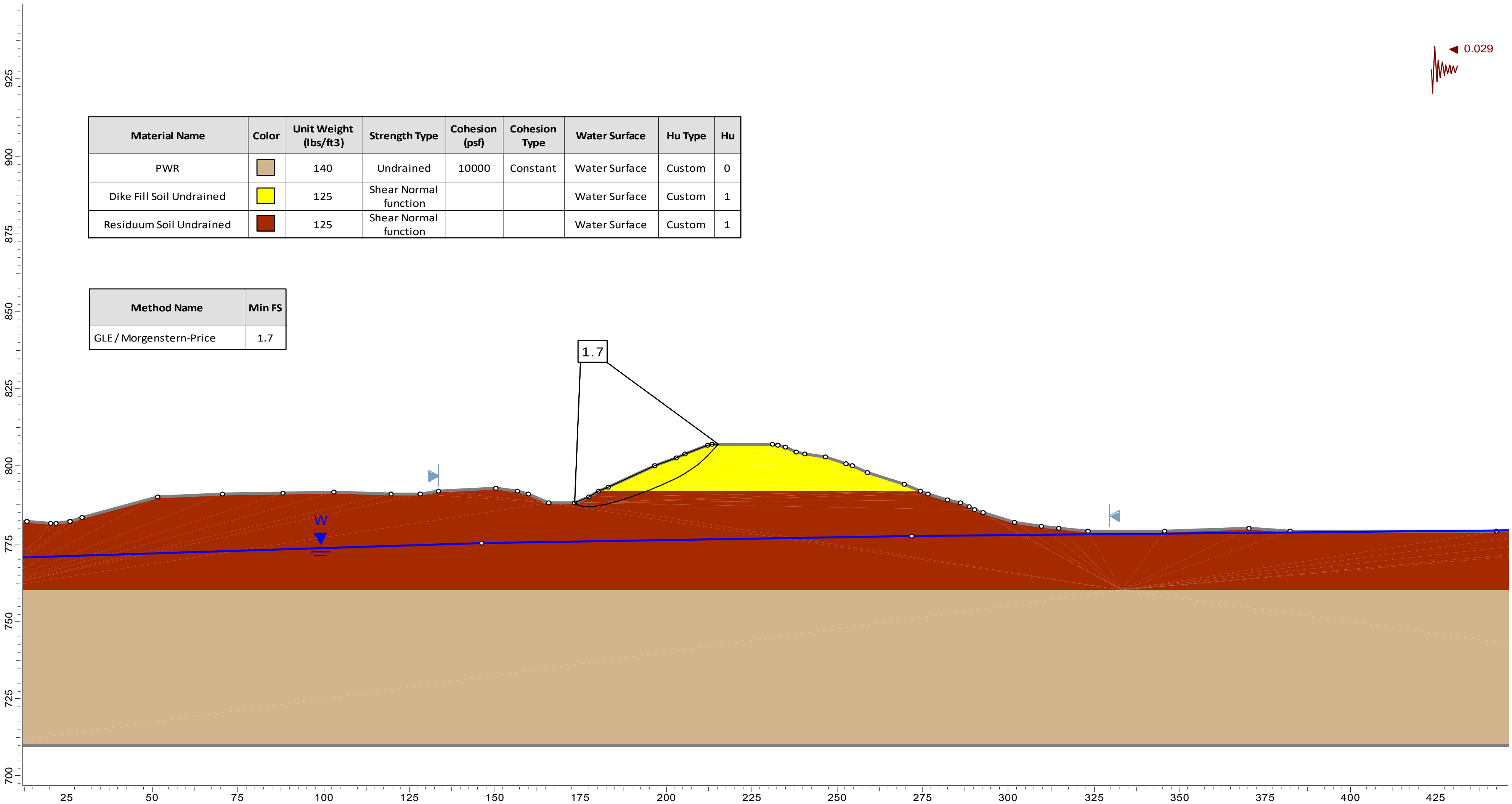
Method Name	Min FS
GLE / Morgenstern-Price	1.8



GOLDER	SCALE AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-2**
	DATE Jan 2018	TITLE	**Section 2C-2C Surcharge Pool**
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	**5(b)**

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

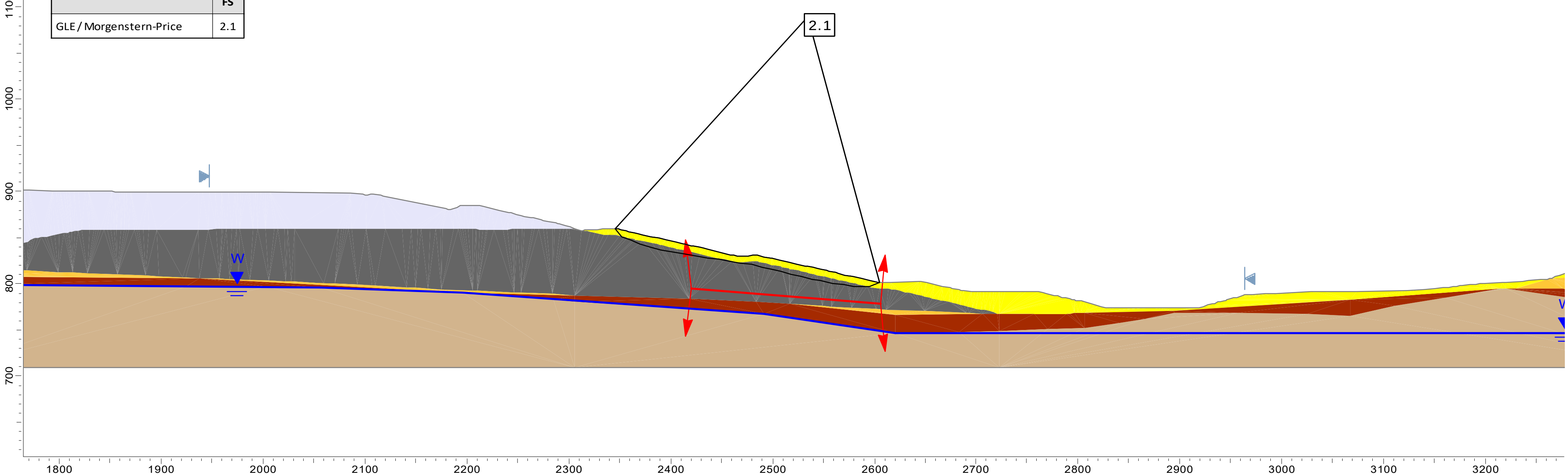
Method Name	Min FS
GLE / Morgenstern-Price	1.7



GOLDER	SCALE AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-2
	DATE Jan 2018	TITLE	Section 2C-2C Seismic Screening
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	5(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	2.1

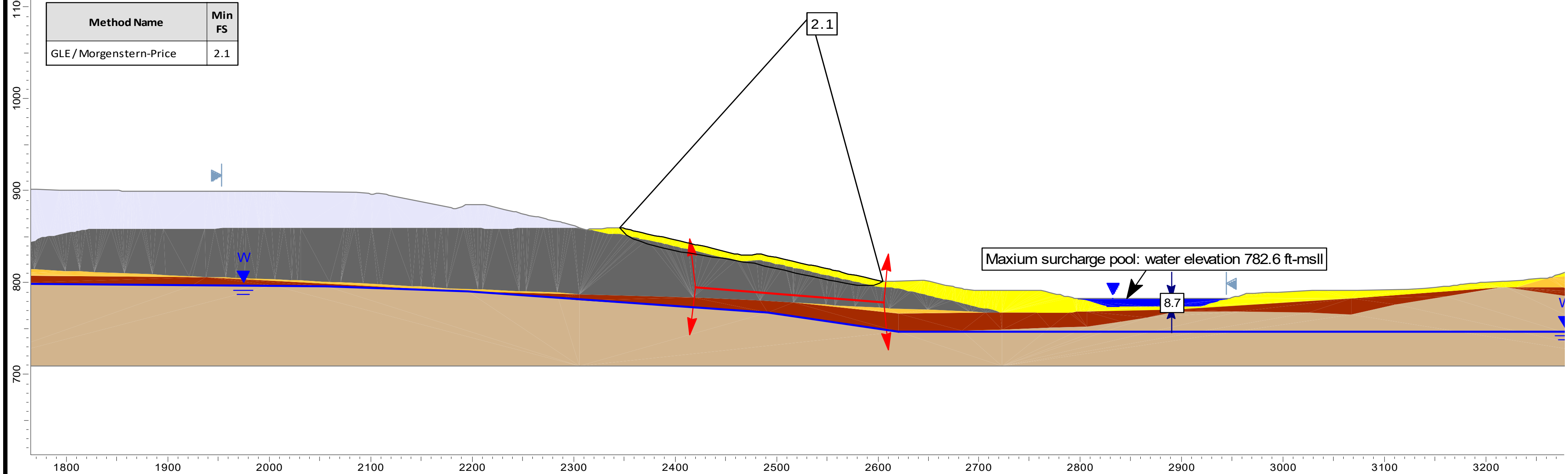


GOLDER

SCALE	AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-3/4
DATE	Jan 2018	TITLE	Section 3/4A-3/4A (North) Storage Pool
MADE BY	LJ		
CAD	-		
FILE	STABILITY	CHECK	JGM
PROJECT No.	1777449	REV.	0
REVIEW	GLH	CLIENT	Georgia Power Company
FIGURE	6(a)		

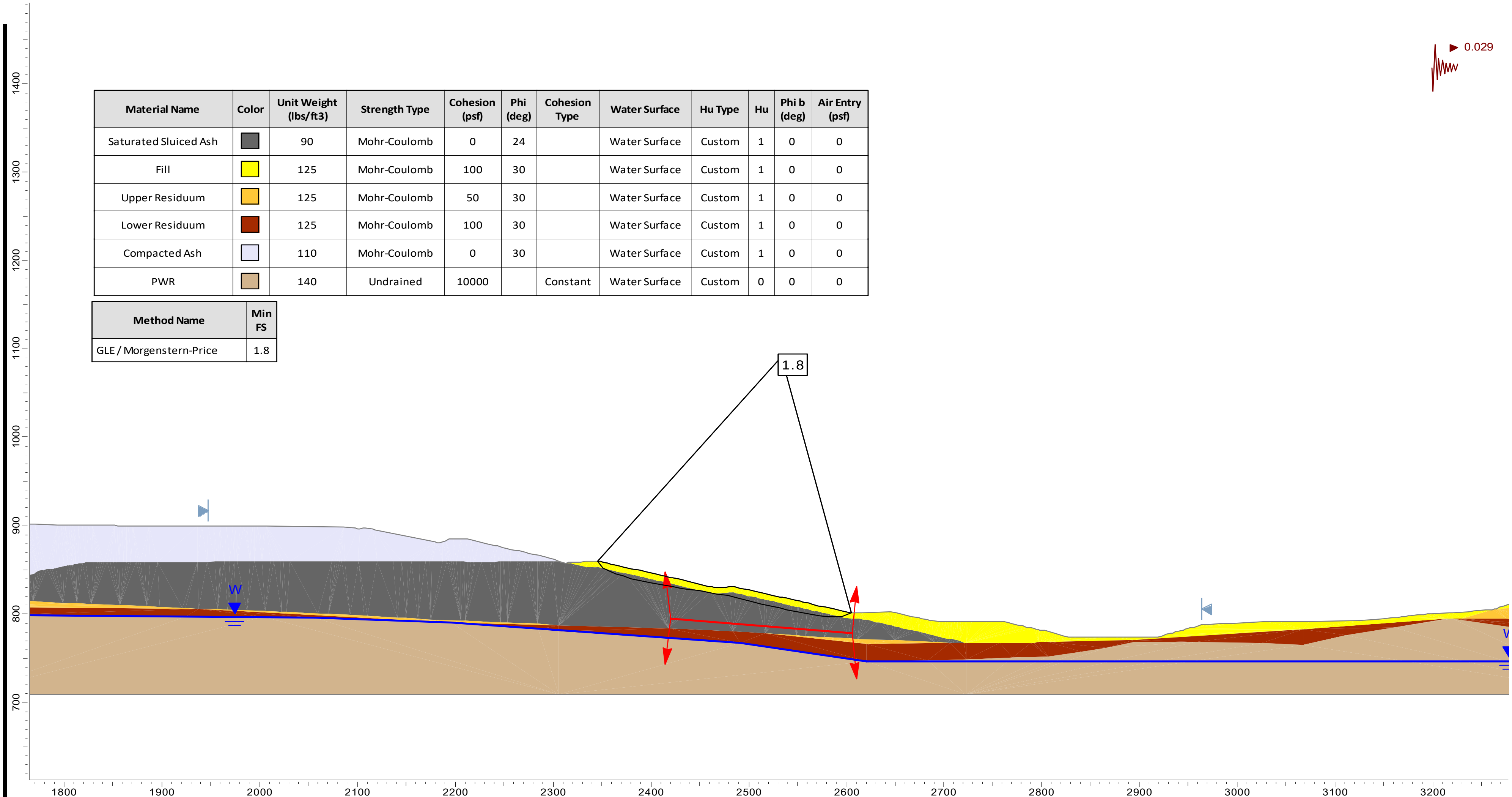
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0
Water		62.4	No strength				Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	2.1



GOLDER

SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
DATE	Jan 2018	TITLE	**Section 3/4A-3/4A (North) Surchage Pool**
MADE BY	LJ		
CAD	-		
FILE STABILITY	CHECK JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	**6(b)**



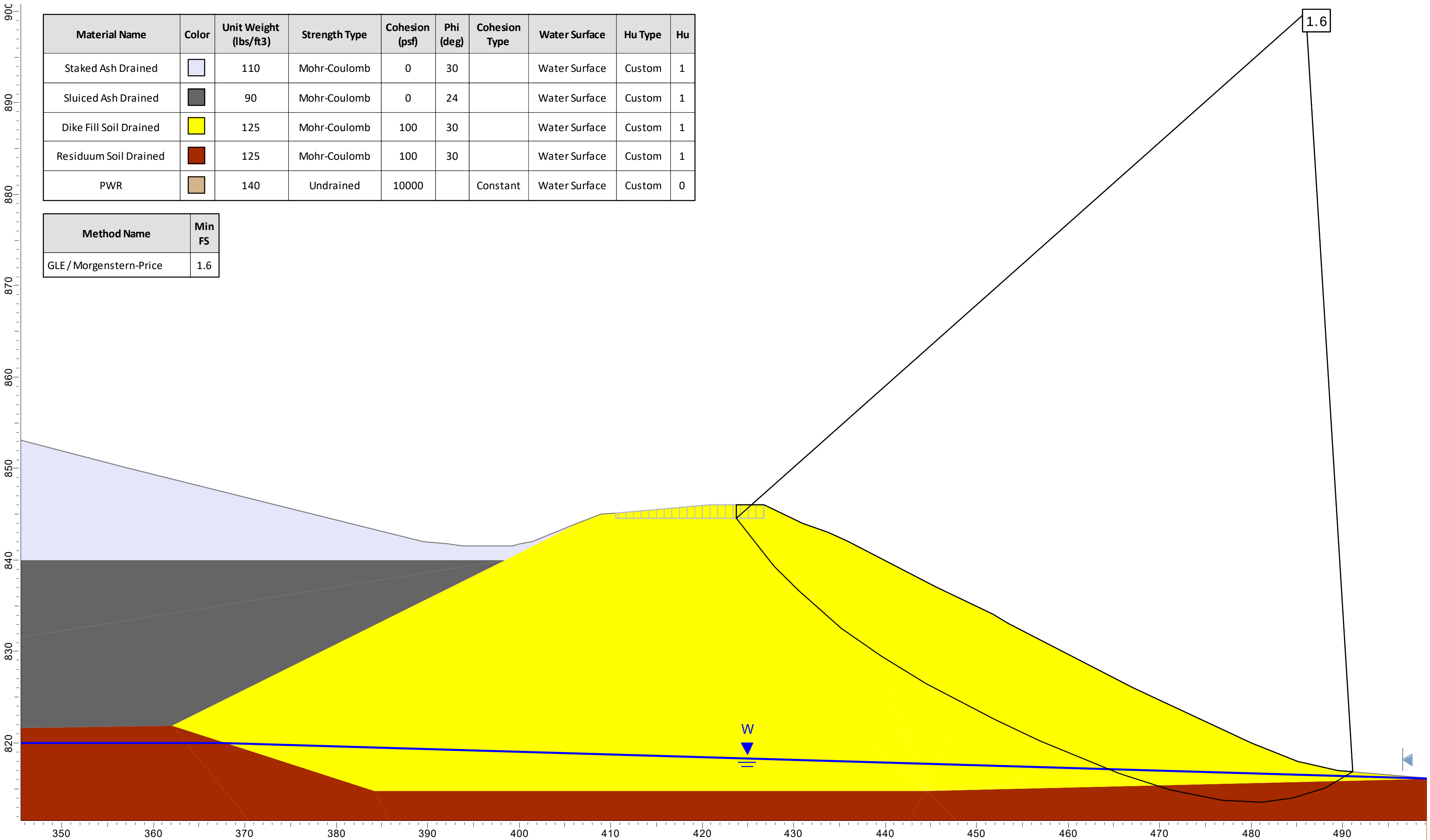
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	1.8

GOLDER	SCALE: AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-3/4
	DATE: Jan 2018	TITLE	Section 3/4A-3/4A (North) Seismic Screening
	MADE BY: LJ		
	CAD: -		
FILE: STABILITY	CHECK: JGM	CLIENT	Georgia Power Company
PROJECT No. 1777449 REV. 0	REVIEW: GLH	FIGURE	6(c)

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

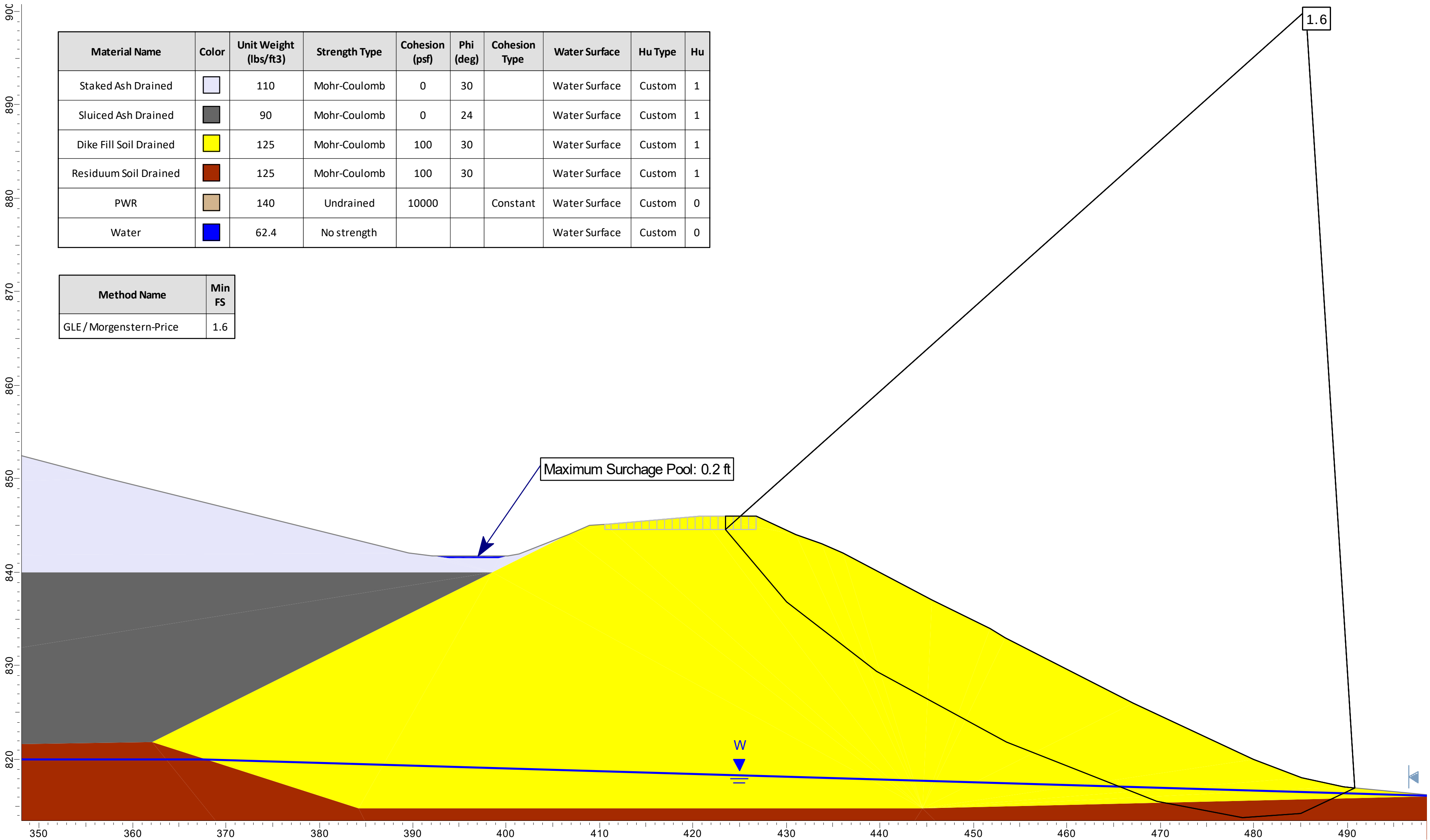
Method Name	Min FS
GLE / Morgenstern-Price	1.6



GOLDER	SCALE AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
	DATE Jul 2018	TITLE	**Section 3/4A-3/4A (South) Storage Pool**
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	7(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
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

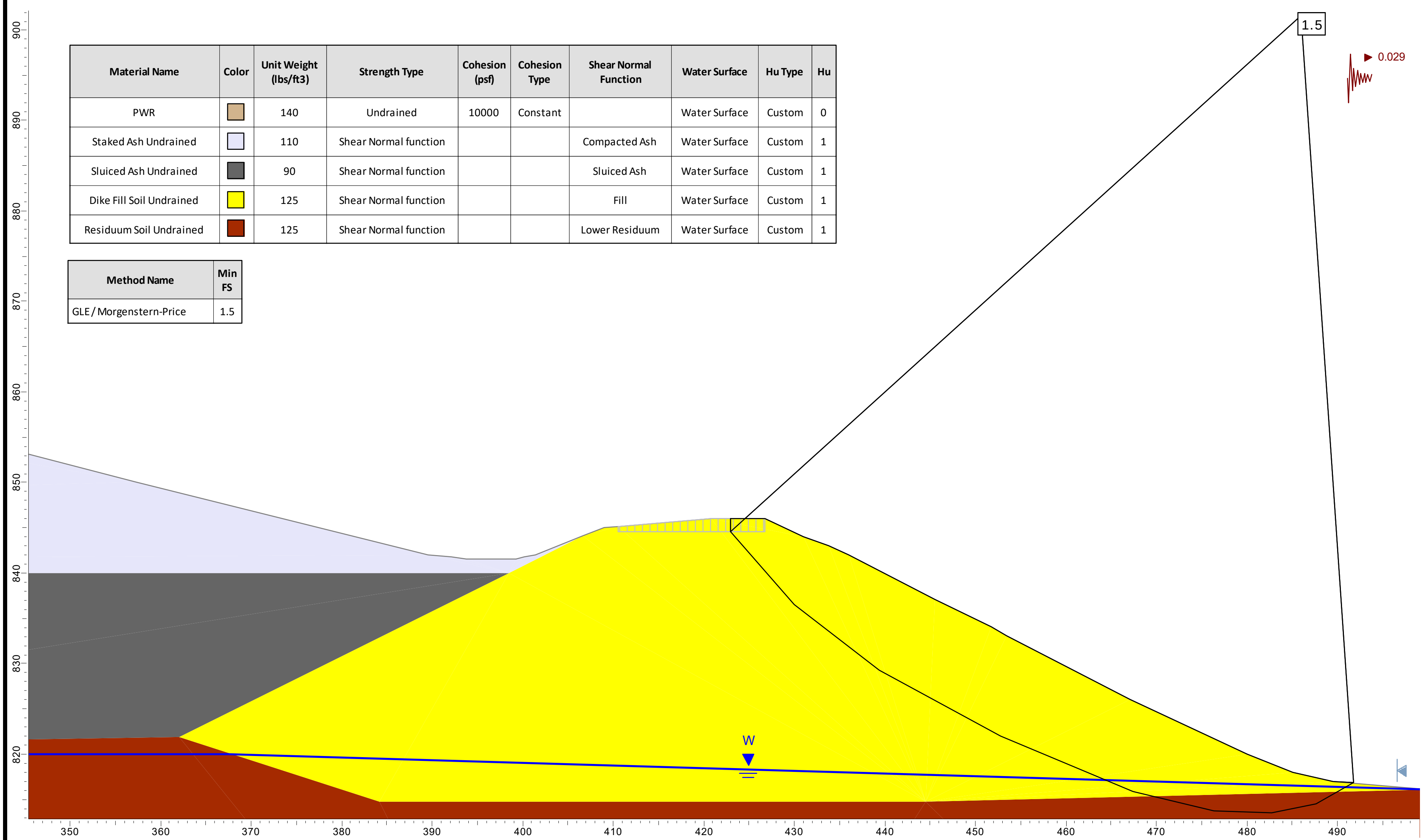
Method Name	Min FS
GLE / Morgenstern-Price	1.6



GOLDER	SCALE: AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
	DATE: Jul 2018	TITLE	**Section 3/4A-3/4A (South) Surcharge Pool**
	MADE BY: LJ		
	CAD: -		
FILE: STABILITY	CHECK: JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 / REV. 0	REVIEW: GLH	FIGURE	7(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
Residuum Soil Undrained		125	Shear Normal function			Lower Residuum	Water Surface	Custom	1

Method Name	Min FS
GLE / Morgenstern-Price	1.5

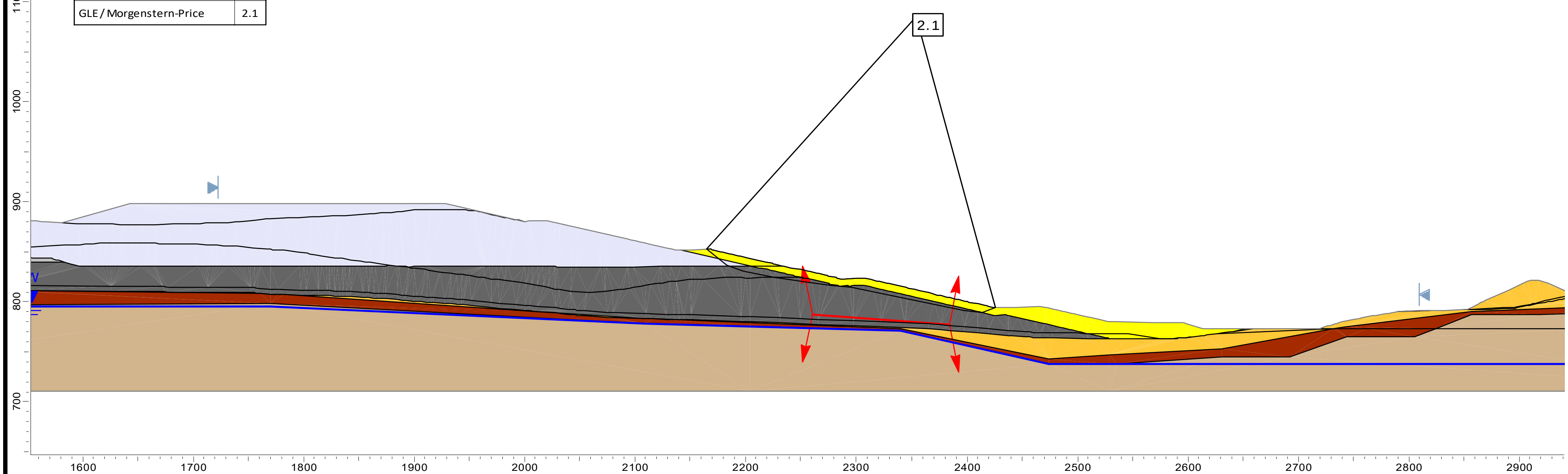


GOLDER

SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
DATE	Jul 2018	TITLE	**Section 3/4A-3/4A (South) Seismic Screening**
MADE BY	LJ		
CAD	-		
FILE	STABILITY		
CHECK	JGM	CLIENT	**Georgia Power Company**
PROJECT No.	1777449	REV.	0
REVIEW	GLH	FIGURE	7(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	1
Fill		125	Mohr-Coulomb	100	30		Water Surface	1
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	1
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	1
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	1
PWR		140	Undrained	10000		Constant	Water Surface	0

Method Name	Min FS
GLE / Morgenstern-Price	2.1

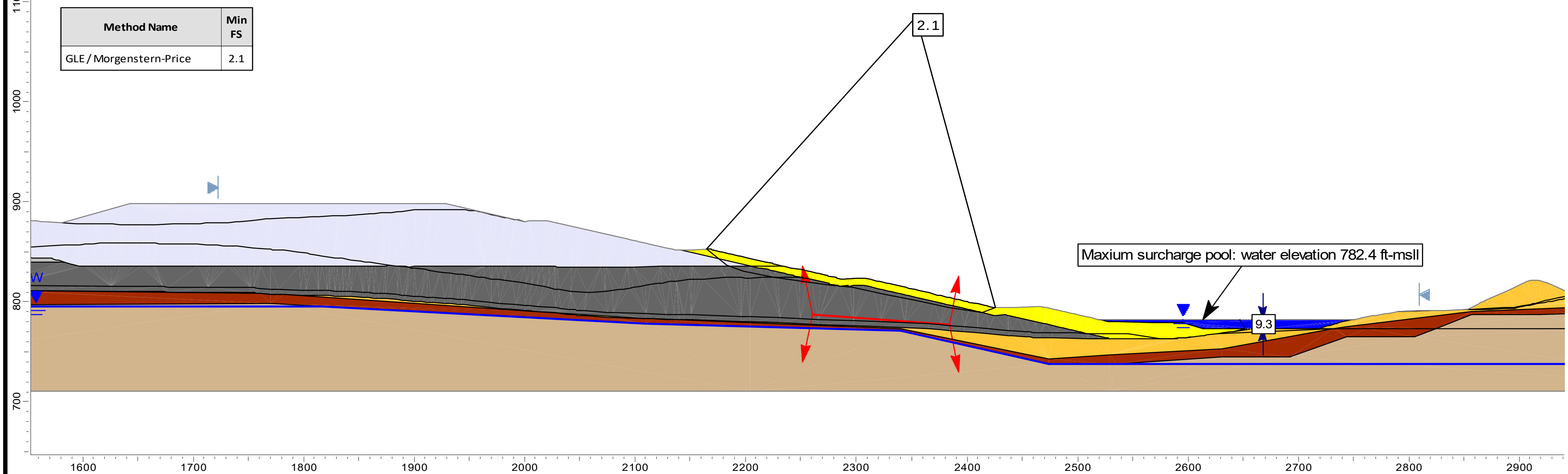


GOLDER

SCALE	AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-3/4
DATE	Jan 2018	TITLE	Section 3/4B-3/4B Storage Pool
MADE BY	LJ		
CAD	-		
FILE	STABILITY	CHECK	JGM
PROJECT No.	1777449	REV.	0
REVIEW	GLH	CLIENT	Georgia Power Company
FIGURE	8(a)		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	Custom	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	Custom	1
Compacted Ash		110	Mohr-Coulomb	0	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	2.1



GOLDER

SCALE	AS SHOWN
DATE	Jan 2018
MADE BY	LJ
CAD	-
CHECK	JGM
REVIEW	GLH
FILE	STABILITY
PROJECT No.	1777449
REV.	0

PROJECT **Closure Design - Plant McDonough AP-3/4**

TITLE **Section 3/4B-3/4B Surchage Pool**

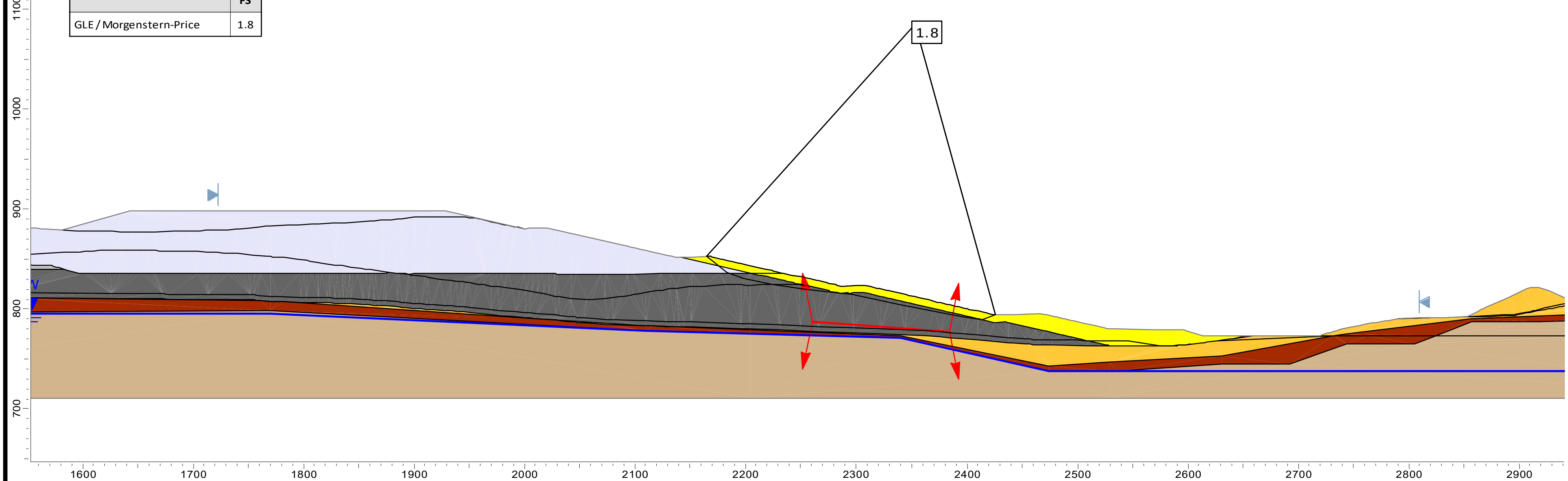
CLIENT **Georgia Power Company**

FIGURE **8(b)**

0.029

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	1
Fill		125	Mohr-Coulomb	100	30		Water Surface	1
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	1
Lower Residuum		125	Mohr-Coulomb	100	30		Water Surface	1
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	1
PWR		140	Undrained	10000		Constant	Water Surface	0

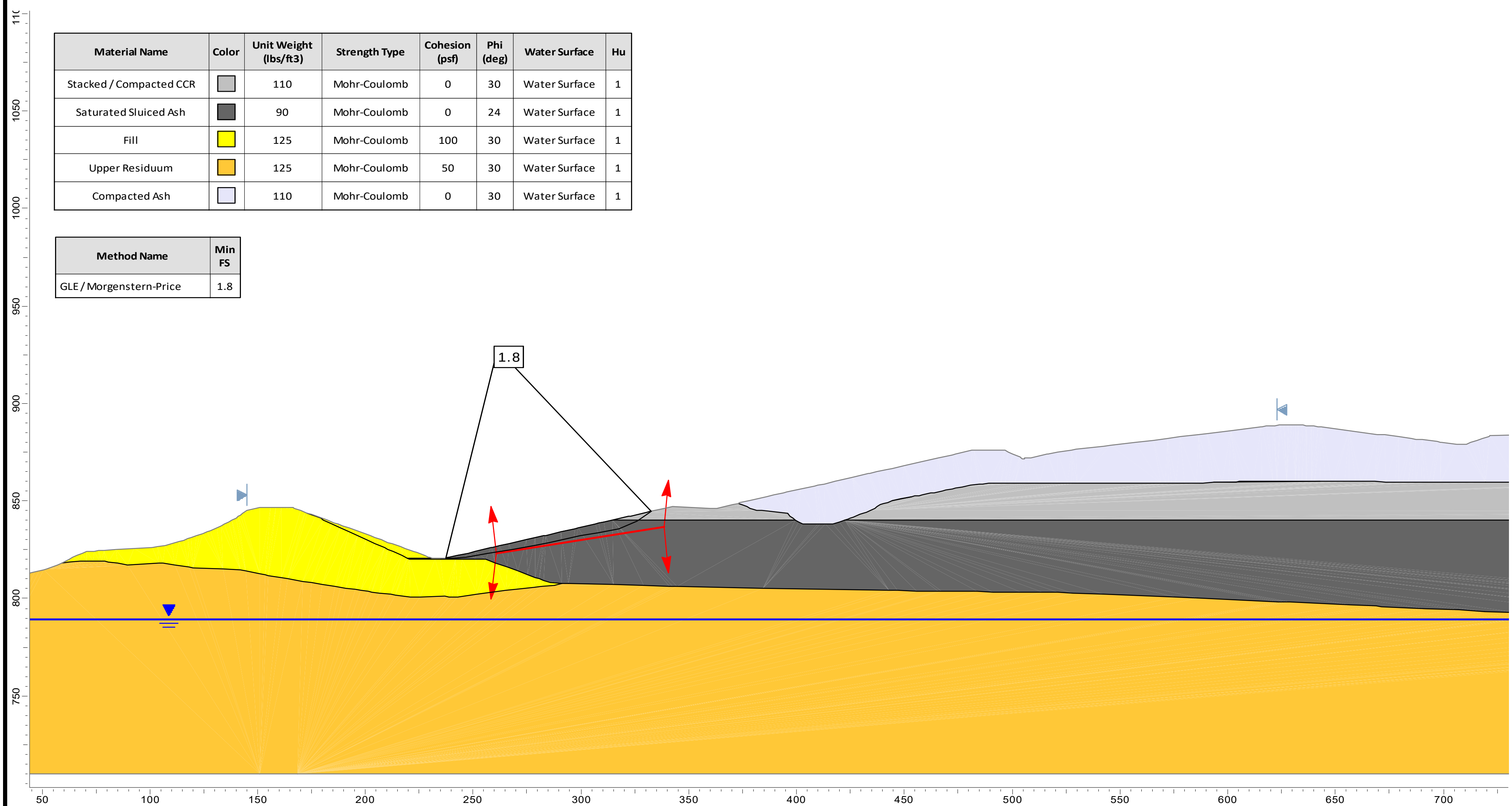
Method Name	Min FS
GLE / Morgenstern-Price	1.8



GOLDER	SCALE: AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-3/4
	DATE: Jan 2018	TITLE	Section 3/4B-3/4B Seismic Screening
	MADE BY: LJ		
	CAD: -		
FILE: STABILITY	CHECK: JGM	CLIENT	Georgia Power Company
PROJECT No. 1777449 — REV. 0	REVIEW: GLH	FIGURE	8(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30	Water Surface	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24	Water Surface	1
Fill		125	Mohr-Coulomb	100	30	Water Surface	1
Upper Residuum		125	Mohr-Coulomb	50	30	Water Surface	1
Compacted Ash		110	Mohr-Coulomb	0	30	Water Surface	1

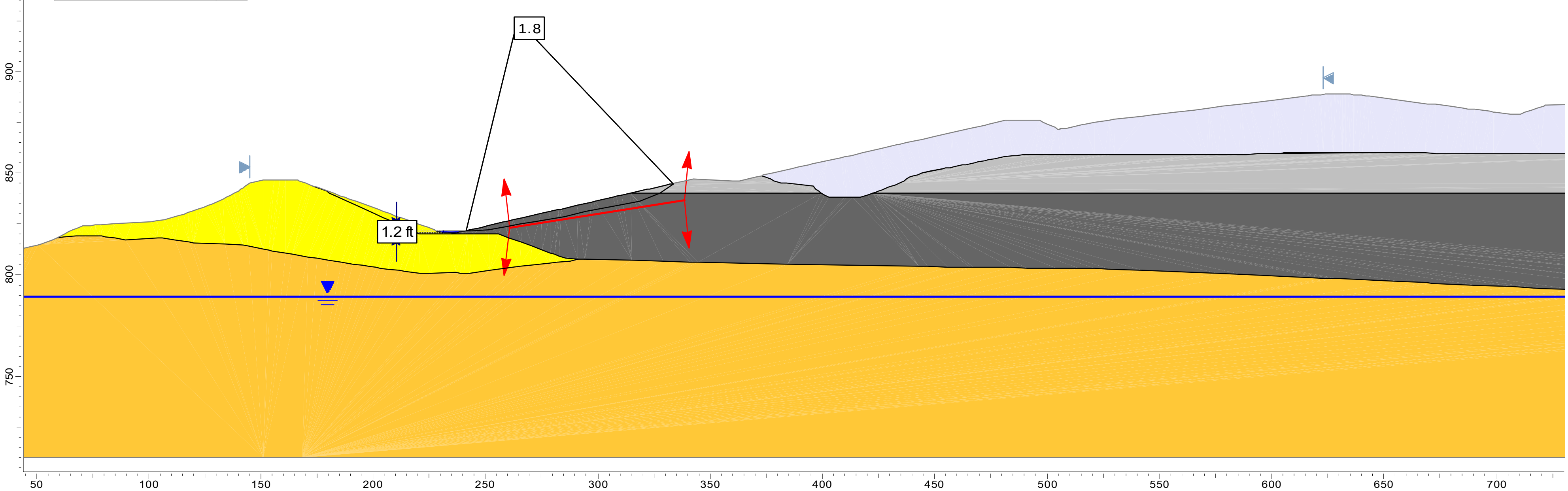
Method Name	Min FS
GLE / Morgenstern-Price	1.8



GOLDER	SCALE AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
	DATE Jan 2018	TITLE	**Section 3/4D-3/4D Storage Pool**
	MADE BY LJ		
	CAD -		
FILE STABILITY	CHECK JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW GLH	FIGURE	9(a)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30	Water Surface	Custom	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24	Water Surface	Custom	1
Fill		125	Mohr-Coulomb	100	30	Water Surface	Custom	1
Upper Residuum		125	Mohr-Coulomb	50	30	Water Surface	Custom	1
Compacted Ash		110	Mohr-Coulomb	0	30	Water Surface	Custom	1
Water		62.4	No strength			Water Surface	Custom	0

Method Name	Min FS
GLE / Morgenstern-Price	1.8



SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
DATE	Jan 2018	TITLE	**Section 3/4D-3/4D Surchage Pool**
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	**Georgia Power Company**
REVIEW	GLH	FIGURE	**9(b)**

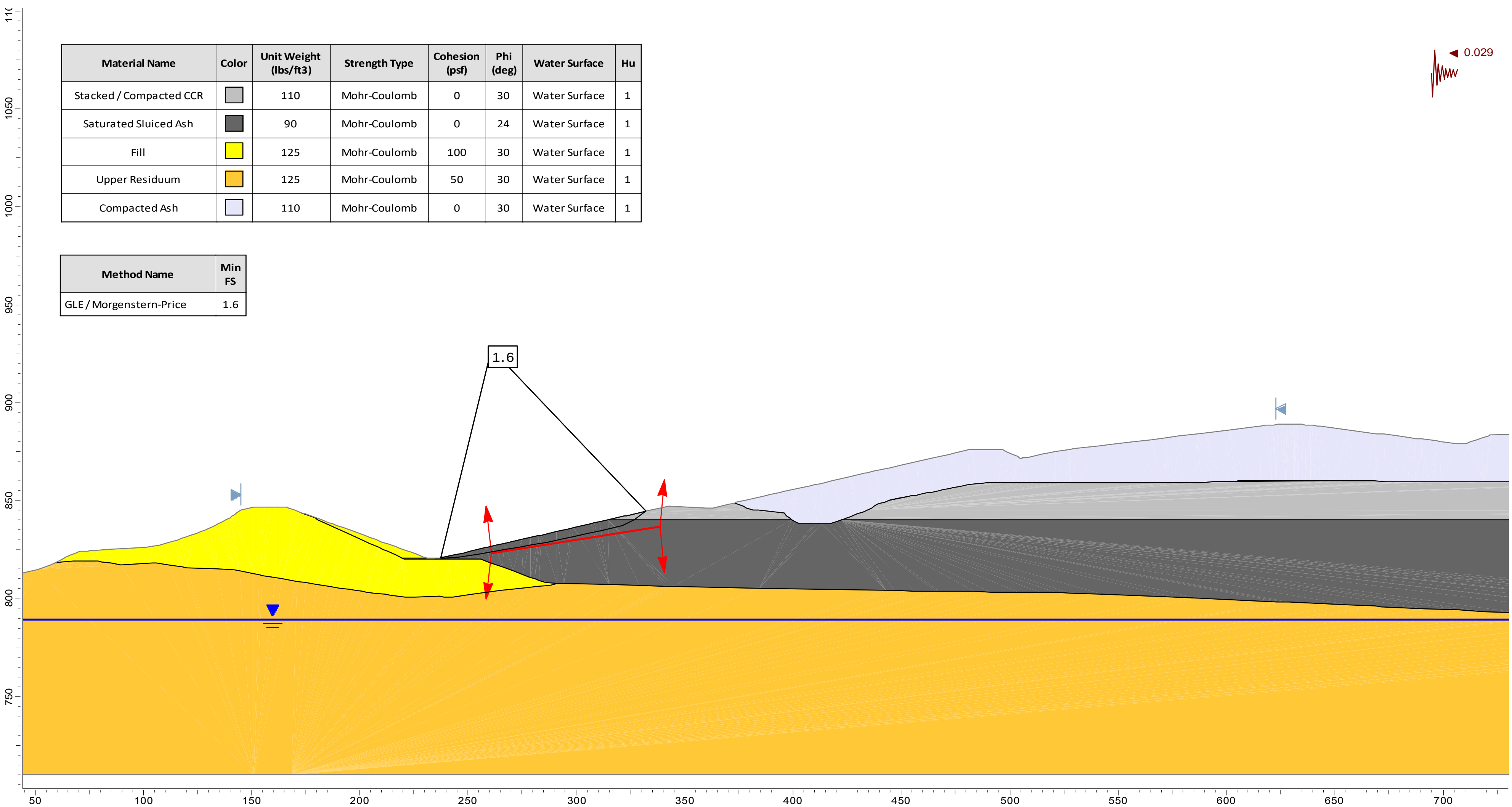
GOLDER

FILE: STABILITY

PROJECT No. 1777449 REV. 0

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30	Water Surface	1
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24	Water Surface	1
Fill		125	Mohr-Coulomb	100	30	Water Surface	1
Upper Residuum		125	Mohr-Coulomb	50	30	Water Surface	1
Compacted Ash		110	Mohr-Coulomb	0	30	Water Surface	1

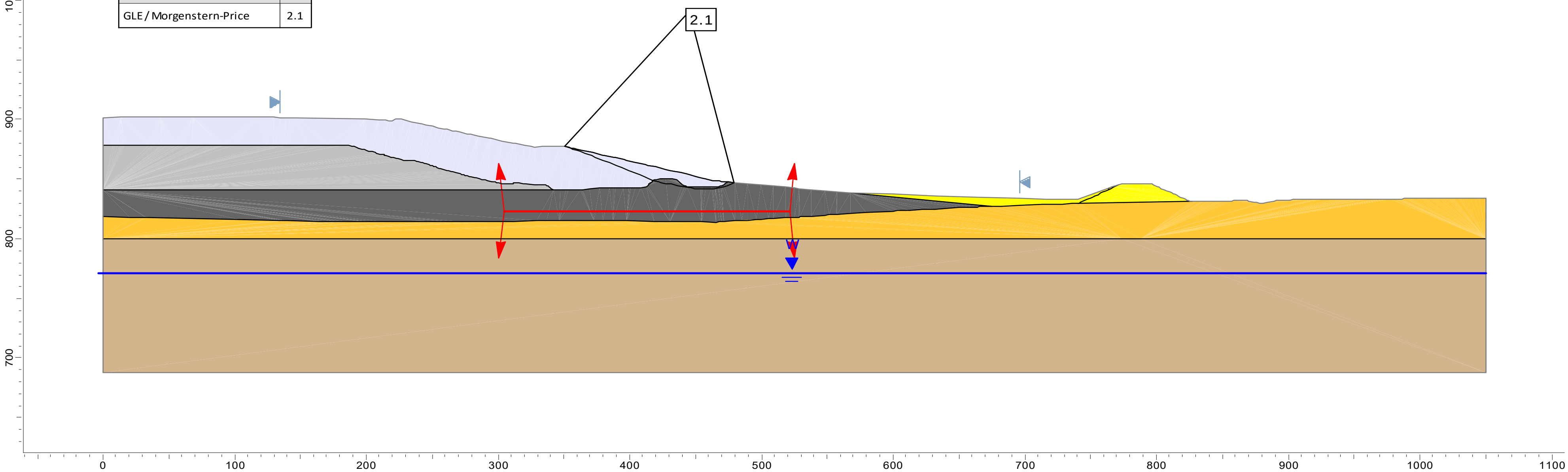
Method Name	Min FS
GLE / Morgenstern-Price	1.6



GOLDER	SCALE: AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
	DATE: Jan 2018	TITLE	**Section 3/4D-3/4D Seismic Screening**
	MADE BY: LJ		
	CAD: -		
FILE: STABILITY	CHECK: JGM	CLIENT	**Georgia Power Company**
PROJECT No. 1777449 REV. 0	REVIEW: GLH	FIGURE	9(c)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	2.1



GOLDER

SCALE AS SHOWN
DATE Jan 2018
MADE BY LJ
CAD -
CHECK JGM
REVIEW GLH

FILE STABILITY
PROJECT No. 1777449 REV. 0

PROJECT **Closure Design - Plant McDonough AP-3/4**

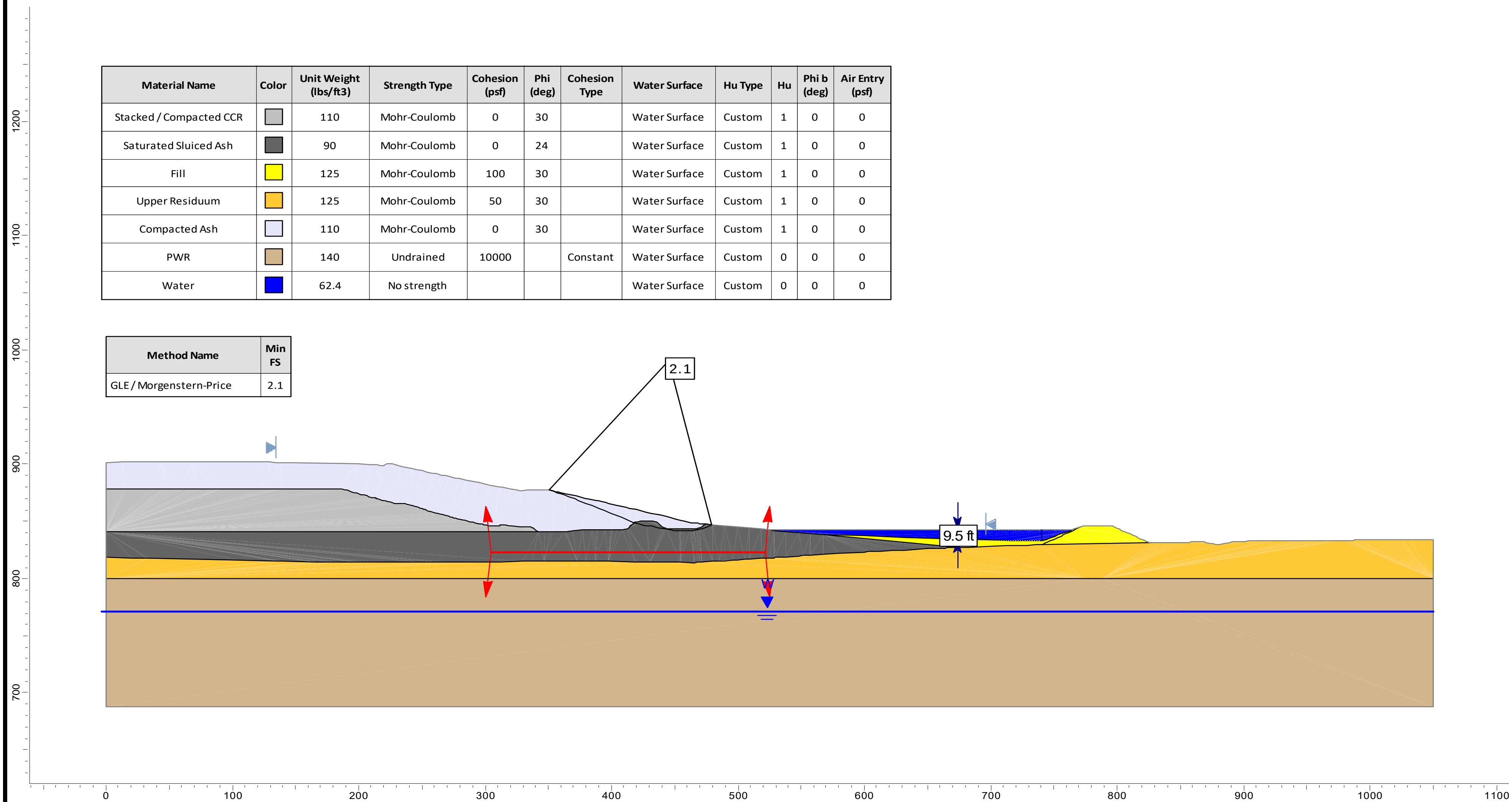
TITLE **Section 3/4J-3/4J Storage Pool**

CLIENT **Georgia Power Company**

FIGURE **10(a)**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Phi b (deg)	Air Entry (psf)
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0
Water		62.4	No strength				Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	2.1



GOLDER

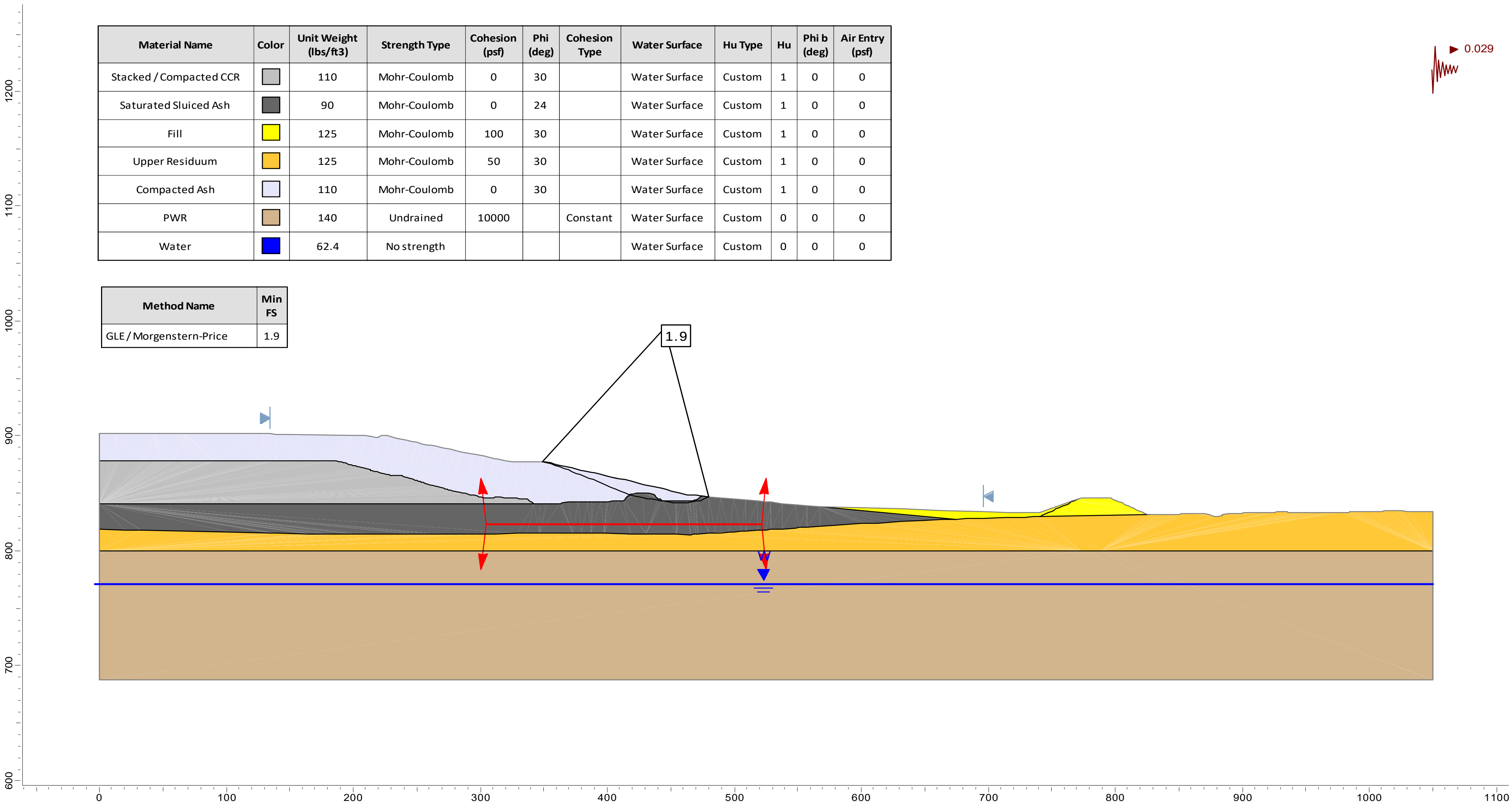
SCALE	AS SHOWN	PROJECT	Closure Design - Plant McDonough AP-3/4
DATE	Jan 2018	TITLE	Section 3/4J-3/4J Surchage Pool
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	Georgia Power Company
REVIEW	GLH	FIGURE	10(b)

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	Phi b (deg)	Air Entry (psf)
Stacked / Compacted CCR		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
Saturated Sluiced Ash		90	Mohr-Coulomb	0	24		Water Surface	Custom	1	0	0
Fill		125	Mohr-Coulomb	100	30		Water Surface	Custom	1	0	0
Upper Residuum		125	Mohr-Coulomb	50	30		Water Surface	Custom	1	0	0
Compacted Ash		110	Mohr-Coulomb	0	30		Water Surface	Custom	1	0	0
PWR		140	Undrained	10000		Constant	Water Surface	Custom	0	0	0
Water		62.4	No strength				Water Surface	Custom	0	0	0

Method Name	Min FS
GLE / Morgenstern-Price	1.9



GOLDER

SCALE	AS SHOWN	PROJECT	**Closure Design - Plant McDonough AP-3/4**
DATE	Jan 2018	TITLE	**Section 3/4J-3/4J Seismic Screening**
MADE BY	LJ		
CAD	-		
CHECK	JGM	CLIENT	**Georgia Power Company**
REVIEW	GLH	FIGURE	**10(c)**

FILE STABILITY

PROJECT No. 1777449 REV. 0

APPENDIX C

Seismic Hazard Calculation Package

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.

Golder Associates Inc.
3730 Chamblee Tucker Road Atlanta, Georgia, USA 30341
T: +1 770 496-1893 | F: +1 770 934-9476

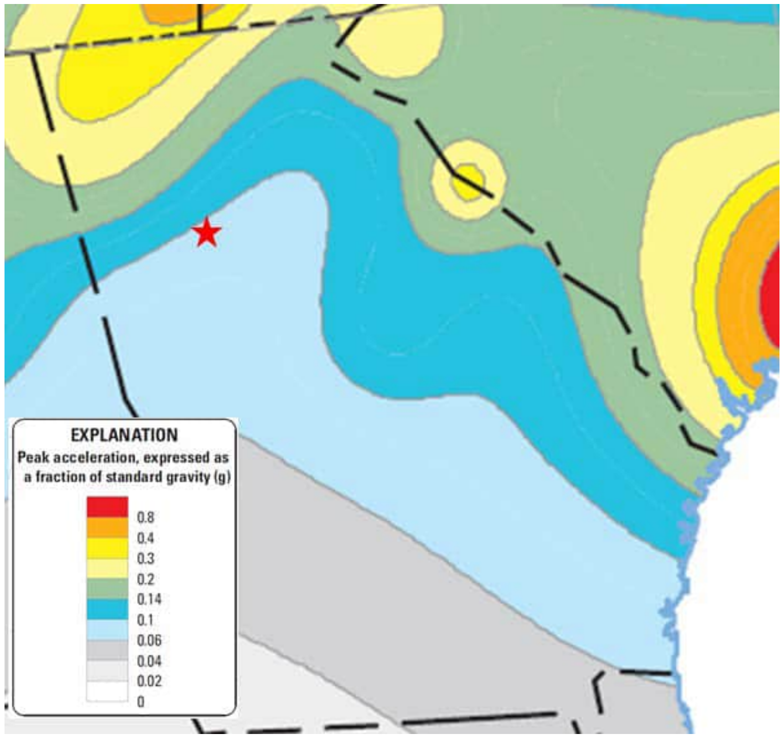


Figure 1: PGA(BC) for the 2% PE in 50 years at the project site (red star). (USGS 2014).

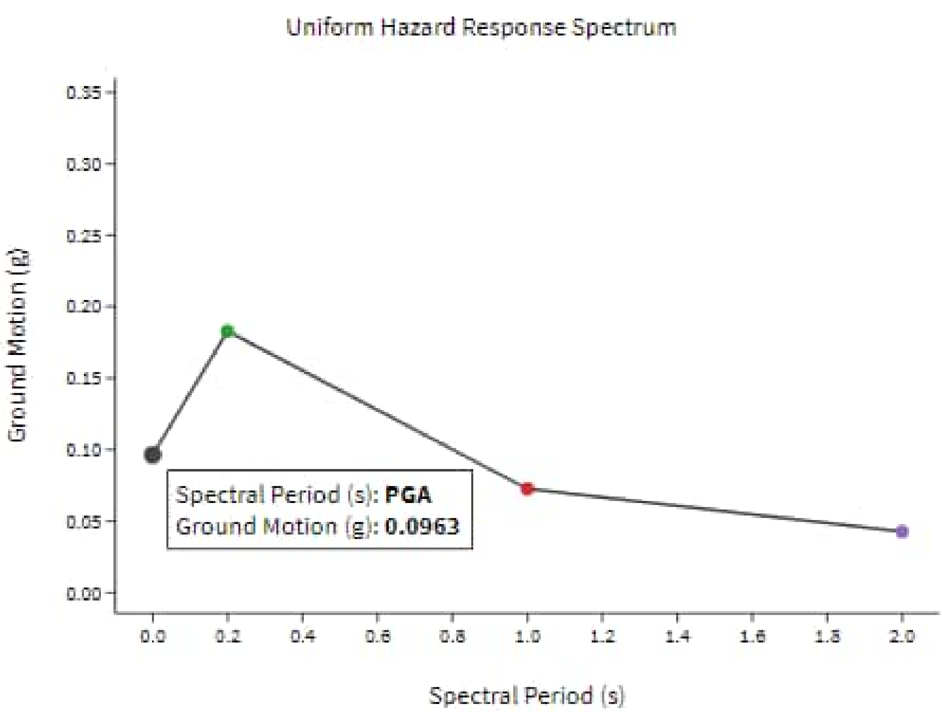


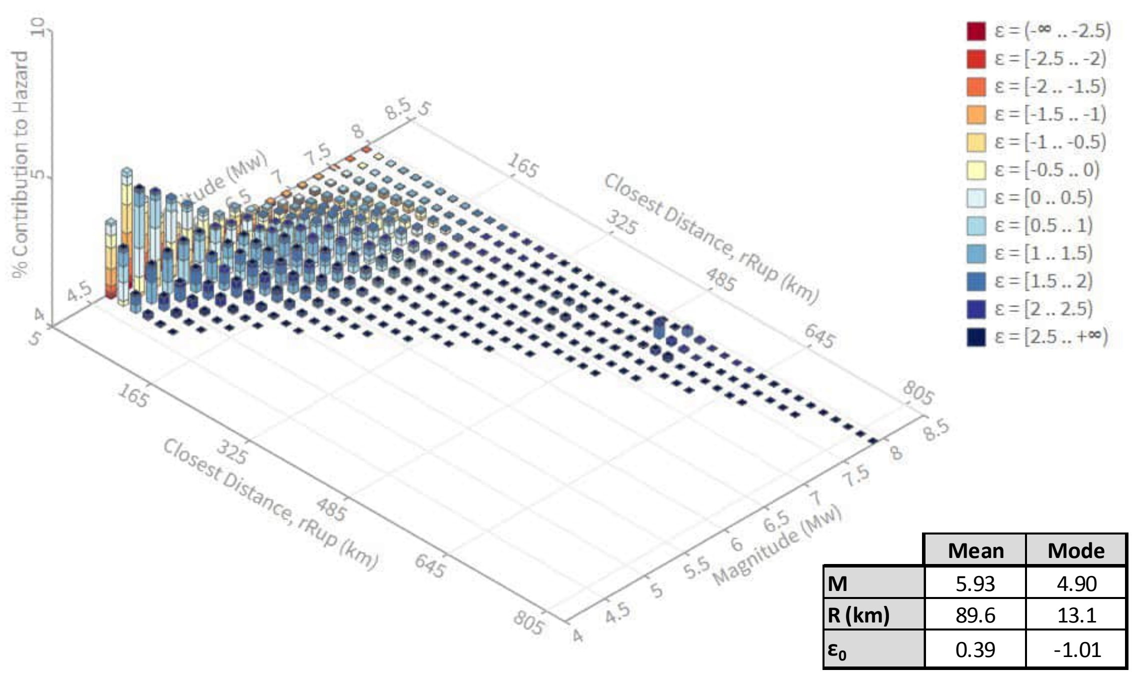
Figure 2: Uniform Hazard Response Spectrum for the 2% PE in 50 years Seismic Hazard at the Project Site (USGS 2014).

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.

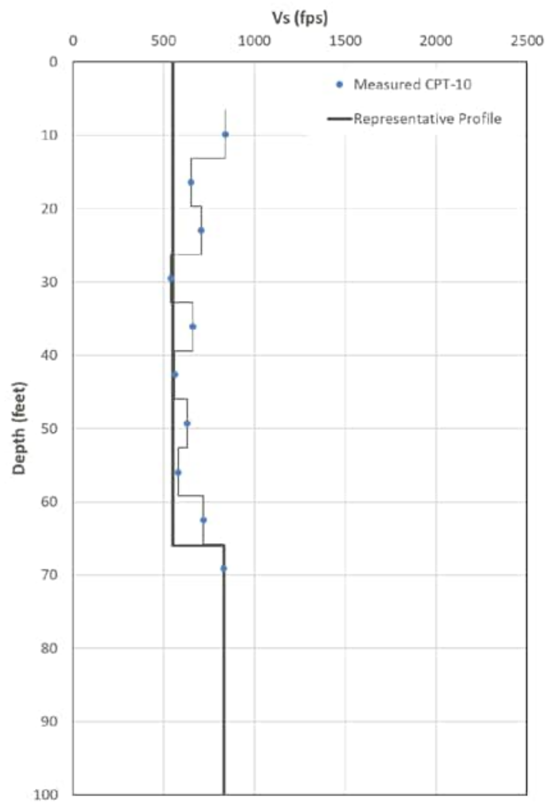


Figure 4. Representative shear wave velocity profile for Plant McDonough CCR Surface Impoundments

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}

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

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:

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

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.

Kramer, S.L. (2008). “Evaluation of Liquefaction Hazards in Washington State” Final Research report WA-RD 668.1, December 2008.

Richardson, G. and E. Kavazanjian. 1995. RCRA Subtitle D (258): Seismic Design Guidance for Municipal Solid Waste Landfill Facilities. U.S. Environmental Protection Agency, Washington, D.C., EPA/600/R-95/051.

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:	February 20, 2018	**Made by:**	LJ
Project No.:	1777449	**Checked by:**	JGM
Subject:	Liquefaction Assessment	**Reviewed by:**	GLH

PROJECT: PLANT MCDONOUGH INACTIVE CCR SURFACE IMPOUNDMENTS AP-2 AND COMBINED UNIT AP-3/4

1.0 OBJECTIVE

The objective of this calculation is to assess the liquefaction potential of the foundation soils and dikes surrounding inactive CCR surface impoundments AP-2, AP-3, and AP-4 at Georgia Power Company (Georgia Power's) Plant McDonough-Atkinson (Plant McDonough). Liquefaction potential is assessed for the final closure condition of these ponds. In the closure condition, AP-3 and AP-4 are consolidated into a single unit; thus, these ponds will be further referenced as combined unit AP-3/4.

This liquefaction assessment uses the screening-level assessment described in Youd et al. (2001). Cone Penetration Test (CPT) data is used to characterize soils for this assessment with updates suggested by Robertson (2009).

2.0 METHODOLOGY

Seismically-induced liquefaction susceptibility was evaluated using the National Center for Earthquake Engineering Research (NCEER) simplified procedure with CPT data (Youd et al., 2001). The simplified procedure is an empirical method used 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). The CRR is a measure of a soils' resistance to liquefaction and was estimated using CPT 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) as described in Golder's Seismic Hazard Calculation Package.

Factors of safety against liquefaction were calculated for six CPT soundings representative of the foundation soils and dikes of AP-2 and AP-3/4. Specifically, the following CPTs were analyzed:

- CPT-02-DAM
- CPT-04-DAM
- CPT-07-DAM
- CPT-10-AP3
- CPT-30-AP4
- CPT-44-AP4

Materials which are dry are not susceptible to liquefaction. Thus, Golder did not calculate the factor of safety against liquefaction for the CCR materials impounded in AP-3/4 since these materials are modelled to be dry in the long term due to dewatering efforts and engineering controls. While these efforts will likely dry large portions of the dike and foundation soils, the extent of drying in the dike and foundation soils was not estimated. Therefore, Golder calculated factors of safety for all dike and foundation soils measured by the CPTs to be conservative.

2.1 CSR Determination

The CSR is defined as:

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

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:

$$r_d = 1.0 - 0.00765z \quad for\ z \leq 9.15\ m$$

$$r_d = 1.174 - 0.0267z \quad for\ 9.15\ m < z \leq 23\ m$$

$$r_d = 0.744 - 0.008z \quad for\ 23\ m < z \leq 30\ m$$

$$r_d = 0.50 \quad for\ z > 30\ m$$

where z is the depth in meters (m). The determination of the a_{max} value is provided in the Golder's Seismic Hazard Calculation Package.

2.2 CRR Determination

The second major step in assessing the liquefaction susceptibility using the simplified approach is to estimate the CRR. Robertson and Wride (1998) developed the procedure for calculating CRR from the CPT as a function of the "clean sand" cone penetration resistance normalized to 1 atmosphere (atm; approximately 100 kilopascals; kPa) and given as $(q_{c1N})_{cs}$. 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 for an earthquake magnitude (M) of 7.5 is given as:

$$(q_{c1N})_{cs} < 50 \quad CRR_{7.5} = 0.833\left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.05$$

$$50 \leq (q_{c1N})_{cs} < 160 \quad CRR_{7.5} = 93\left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08$$

where $(q_{c1N})_{cs}$ is the clean sand cone penetration resistance normalized to 1 atm (approximately 100 kPa or 1 ton per square foot; tsf).

The tip resistance (q_c) is normalized to obtain q_{c1N} as:

$$q_{c1N} = C_Q\left(\frac{q_c}{P_a}\right)$$

$$C_Q = \left(\frac{P_a}{\sigma'_v}\right)^n$$

where C_Q is the normalizing factor for cone penetration resistance, P_a is 1 atm of pressure, n is an exponent that is dependent on the soil type, and q_c is the cone tip penetration resistance (q_c is replaced by q_t, the cone tip resistance corrected for geometric impacts of the pore pressure measurement in all instances).

The method adopted in this assessment calculates the exponent, n, according to a method developed by Robertson (2009) and represents a small modification to the standard NCEER approach. The exponent, n, is calculated as:

$$n = 0.381I_c + 0.05\left(\frac{\sigma'_{vo}}{P_a}\right) - 0.15 \leq 1.0$$

where

$$I_c = [(3.47 - logQ_{t1})^2 + (1.22 + logF_r)^2]^{0.5}$$

$$Q_{t1} = \left[\frac{q_c - \sigma_{vo}}{\sigma'_{vo}}\right]$$

$$F_r = \left[\frac{f_s}{q_c - \sigma_{vo}}\right] \times 100\%$$

2.2.1 Clean Sand Equivalent Cone Penetration Resistance $(q_{c1N})_{cs}$

According to the NCEER approach, the presence of fines affects the liquefaction resistance of soils. A correction factor, K_c, is applied to the normalized penetration resistance (q_{c1N}) to determine the clean sand equivalent $(q_{c1N})_{cs}$ where:

$$(q_{c1N})_{cs} = K_c q_{c1N}$$

$$for\ I_c \leq 1.64 \quad K_c = 1.0$$

$$for\ I_c > 1.64 \quad K_c = -0.403I_c^4 + 5.581I_C^3 - 21.63I_c^2 + 33.75I_c - 17.88$$

Note that in the case of CCR materials the clean sand correlation is not conservative, and in cases of applying this approach to CCR materials a modified factor is used by Golder to be more conservative.

2.2.2 Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5 (Youd et al. 2001) where the factor of safety against liquefaction is calculated as

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF$$

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:

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

Where M is the design earthquake magnitude (5.75, see more details in Seismic Hazard Calculation Package).

2.3 Factor of Safety Against Liquefaction

The factor of safety was calculated as:

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF$$

The factor of safety was calculated for every recorded depth reading in each CPT. Liquefaction calculations for each CPT including the calculated factors of safety are graphically presented in the figures attached to the end of this text.

3.0 RESULTS AND CONCLUSIONS

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 a required factor of safety of 1.2 against liquefaction for pond impoundment structures (§257.73(e)(iv)). The dikes and foundation soils at AP-2 and AP-3/4 meet this requirement as all calculated factors of safety against liquefaction for these materials are greater than 1.2 in all CPT soundings analyzed. CCR materials impounded in AP-3/4 were not analyzed for liquefaction susceptibility since these materials will be dry in the long term and, thus, not susceptible to liquefaction.

4.0 REFERENCES

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.

Robertson, P.K. and C.E. (Fear) Wride (1998) "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test," Canadian Geotechnical Journal, Vol. 35, pp. 442-459.

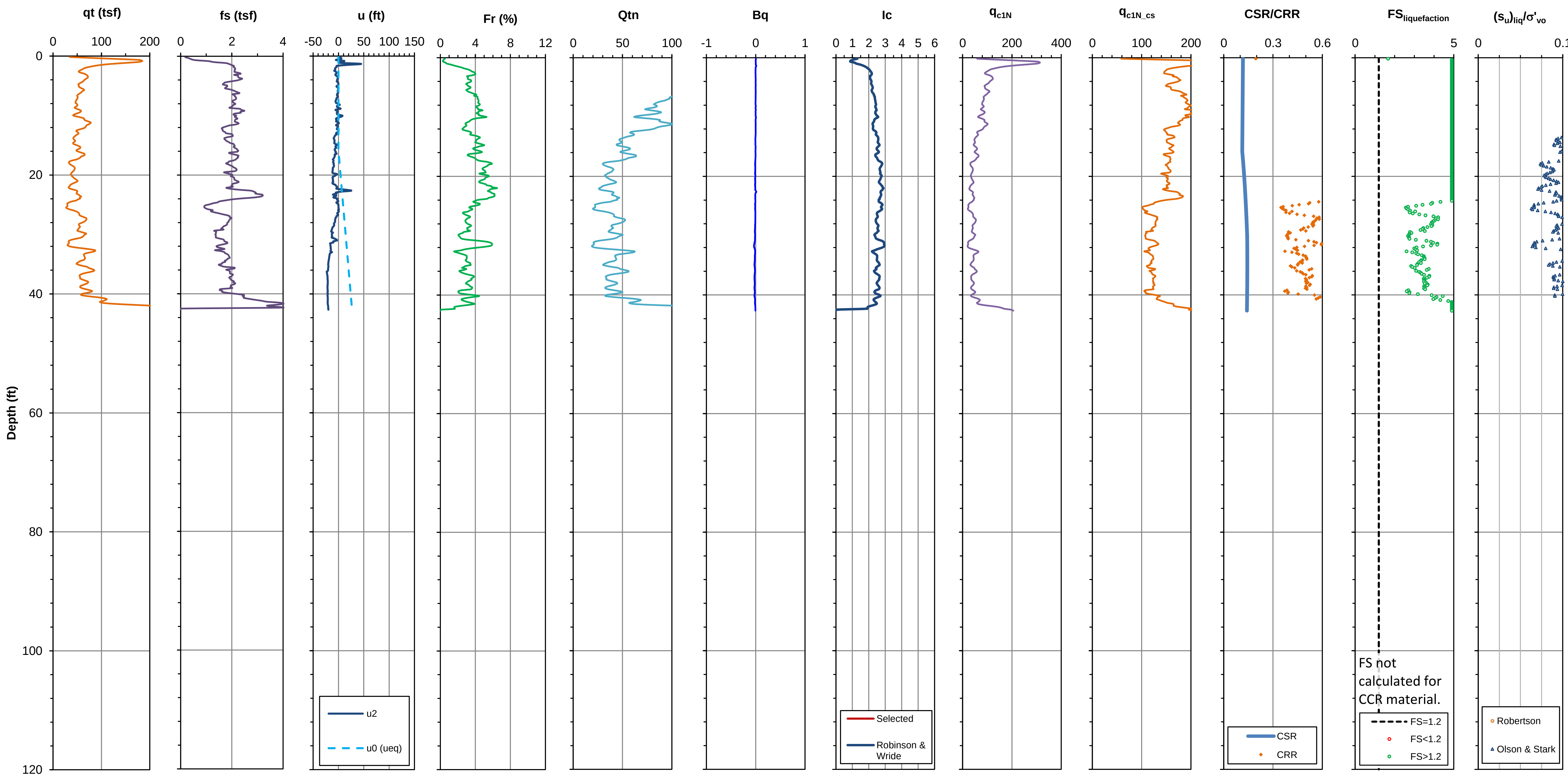
Youd, T.L. et al. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, vol. 127, No. 4, April 2001.

5.0 ATTACHMENTS

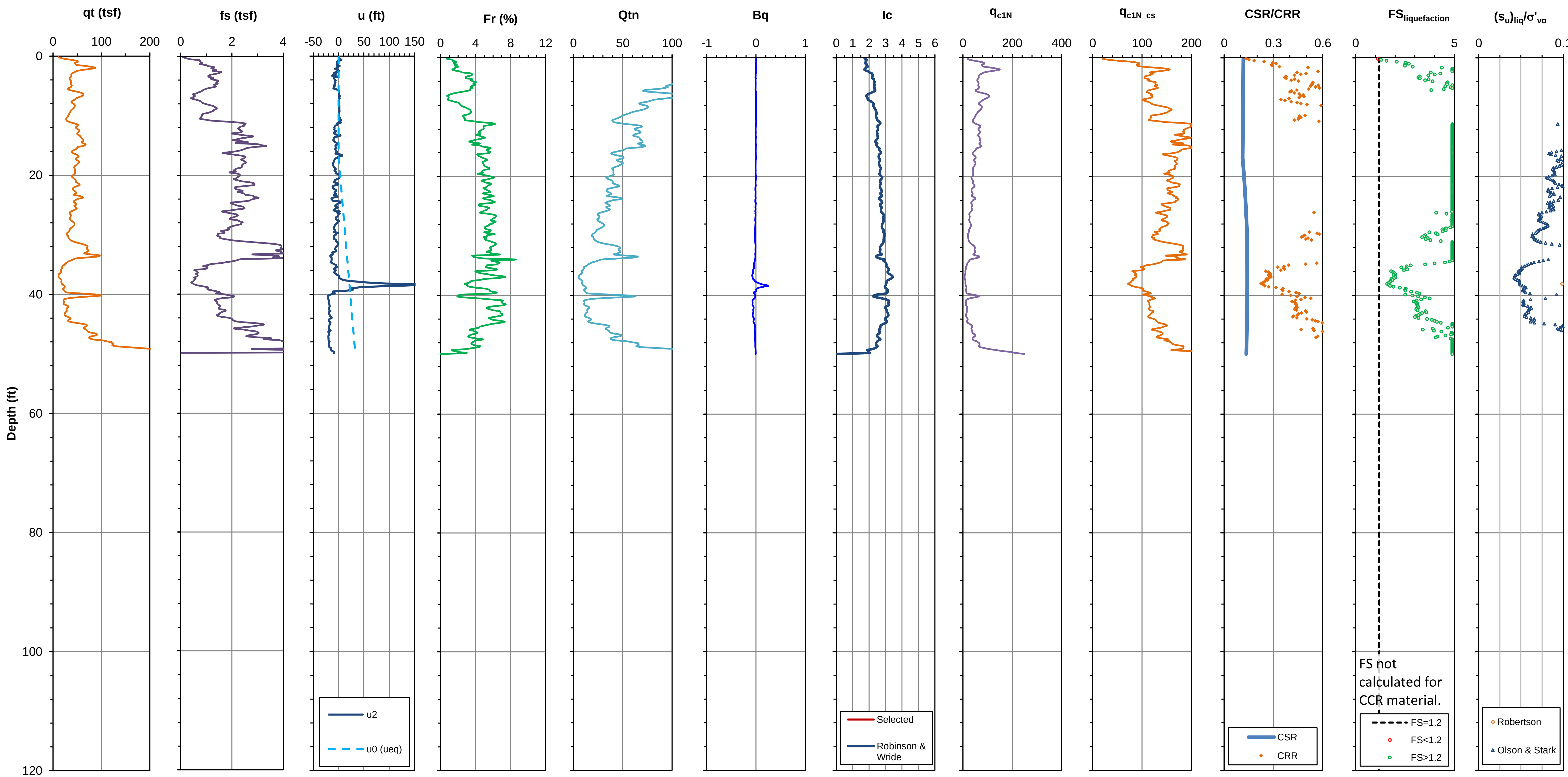
Liquefaction Factor of Safety Results

Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	15.8 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-2-DAM	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	1392584.5	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	**a_{max}:**	0.18 g
Easting	2202822.0	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	845.8 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				

GOLDER

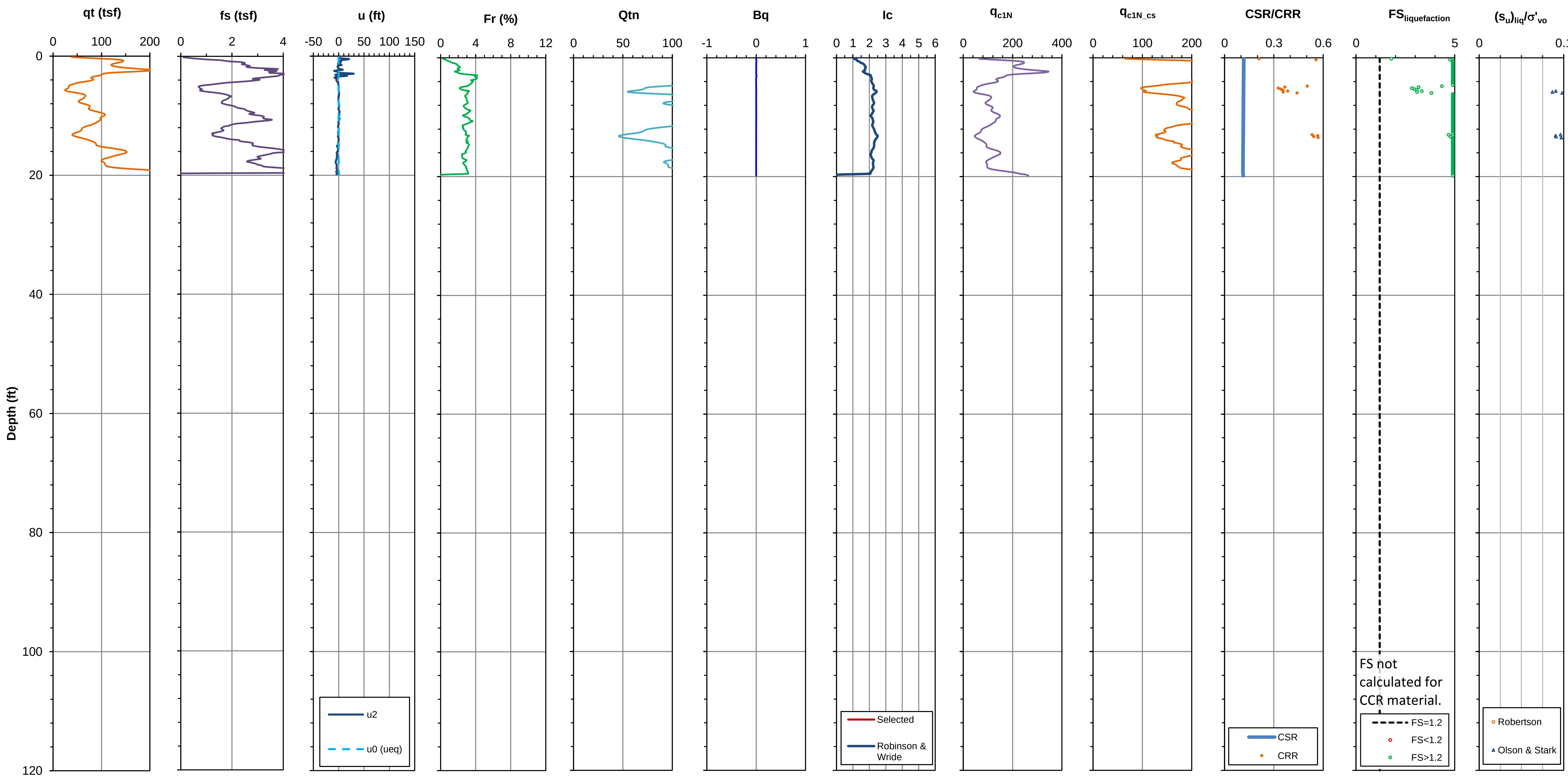


Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	16.9 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-04-DAM	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	1392271.1	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	**a_{max}:**	0.18 g
Easting	2201861.3	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	846.9 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				

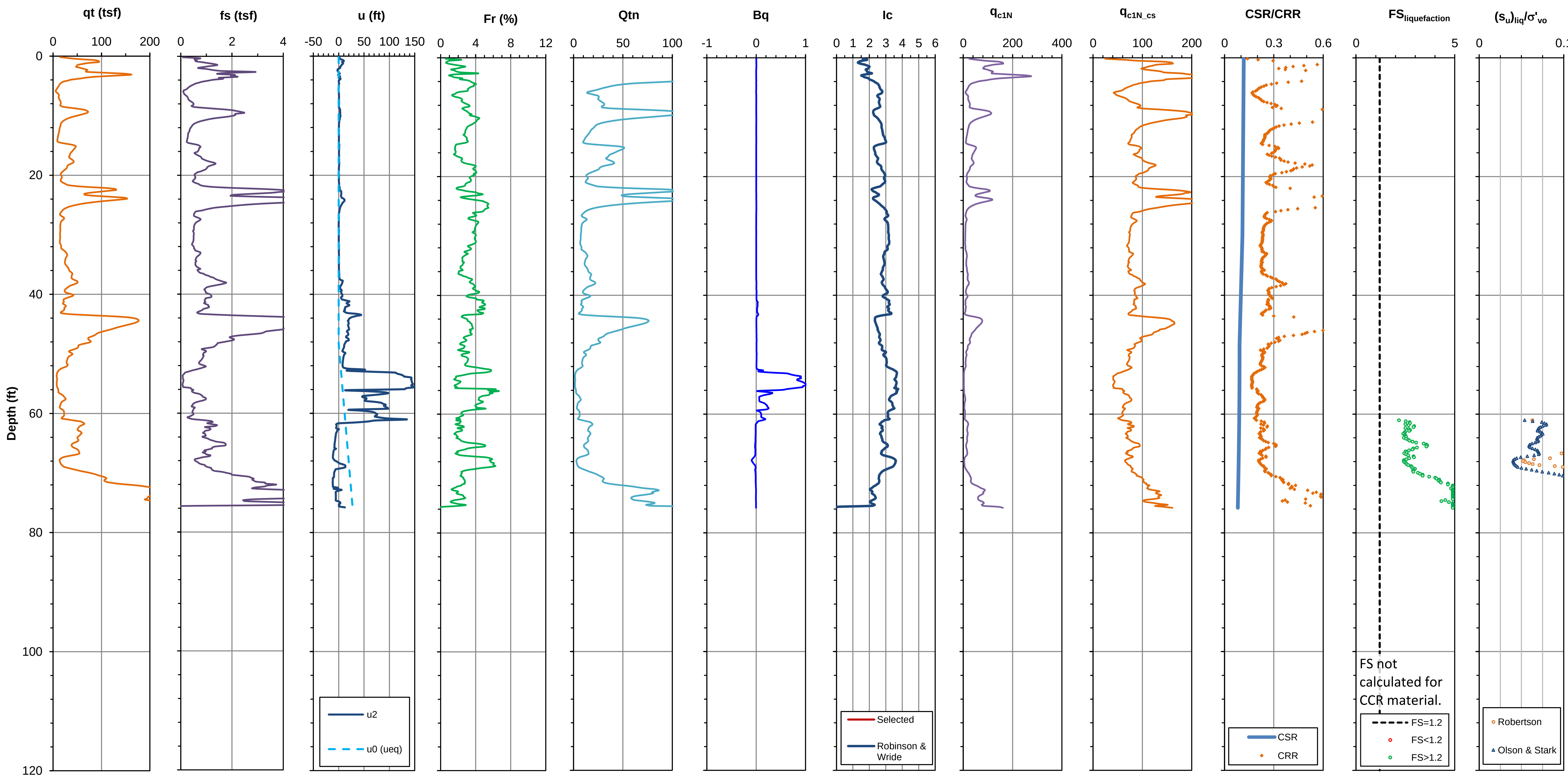


Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	19.1 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-07-DAM	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	1393082.5	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	**a_{max}:**	0.18 g
Easting	2201627.5	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	849.2 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				

GOLDER

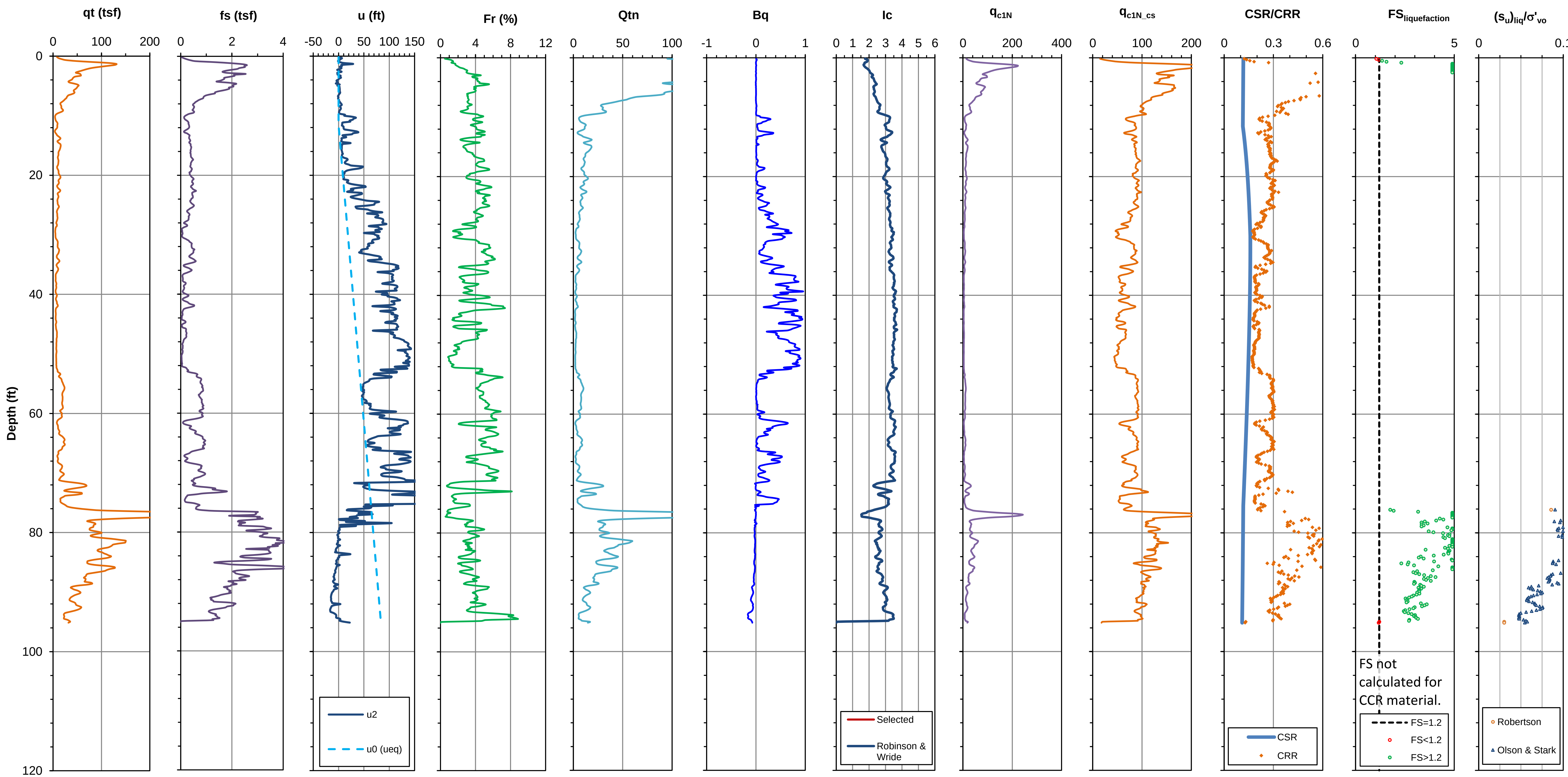


Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	48.3 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-10-AP3	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	132917.1	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	$\mathbf{a_{max}}$:	0.18 g
Easting	2202107.3	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	878.3 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				

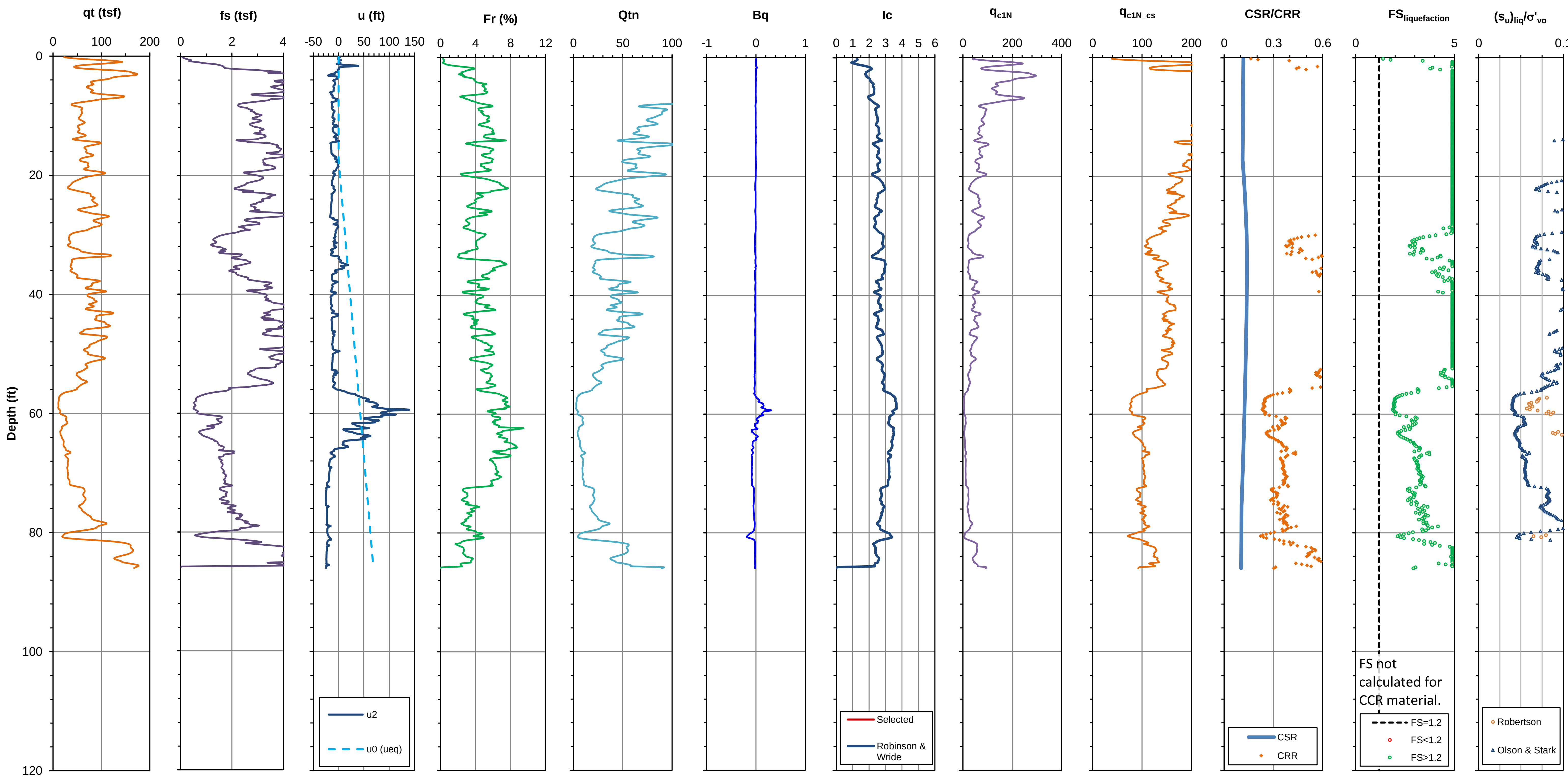


Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	11.6 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-30-AP4	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	1393796.0	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	**a_{max}:**	0.18 g
Easting	2203383.8	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	841.5 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				

GOLDER



Test Date:	11/4/2015	**Project:**	Plt McDonough Permitting	**Test Type:**	CPTU	**Water Table:**	17.3 ft	**2% PE in 50 years Seismic Hazard**	
Test ID:	CPT-44-AP4	**Location:**	Smyrna, GA	**Device:**	10 cm^2, Type 2 filter	**Golder Eng:**	L. Jin	**Magnitude:**	5.75
Northing	1394120.5	**Client:**	GPC	**Standard:**	ASTM D5778	**Check**	G. Martin	**a_{max}:**	0.18 g
Easting	2203001.0	**Proj No.:**	1777449	**Push Co.:**	ConeTec	**Review:**	G. Hebeler		
Elevation:	847.3 ft	**Termination:**	0.0 ft-bgs	**Operator:**	ConeTec				



APPENDIX E

Settlement Analysis Calculation Package

GOLDER

CALCULATIONS

Date:	June 2018	**Made by:**	LJ
Project No.:	1777449	**Checked by:**	JGM
Subject:	Settlement Analyses	**Reviewed by:**	GLH

PROJECT: PLANT MCDONOUGH-ATKINSON INACTIVE CCR SURFACE IMPOUNDMENTS AP-2 AND COMBINED UNIT AP-3/4 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-2 and Combined Unit AP-3/4 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. The closure design of AP-3/4 also includes significant active dewatering during closure which is expected to result in most dewatering induced settlements to be complete prior to final grading and capping.

Post-closure settlement of Combined Unit AP-3/4 is expected to occur based on any remaining water level lowering within the capped CCR material 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 1.0. Based on 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

As discussed above, most of the settlement due to grading and dewatering of ash is calculated to occur prior to final grading and capping. The post closure settlement analysis results for the AP-3/4 unit are presented in Figure 1 below. The maximum calculated settlement in ash pond is less than 0.1 feet (< 1 inch). Therefore, settlement is expected to have minimal impact to the final grades after closure. Should localized areas of settlement occur, these will be monitored and can be maintained as necessary through the post-closure care inspection program.

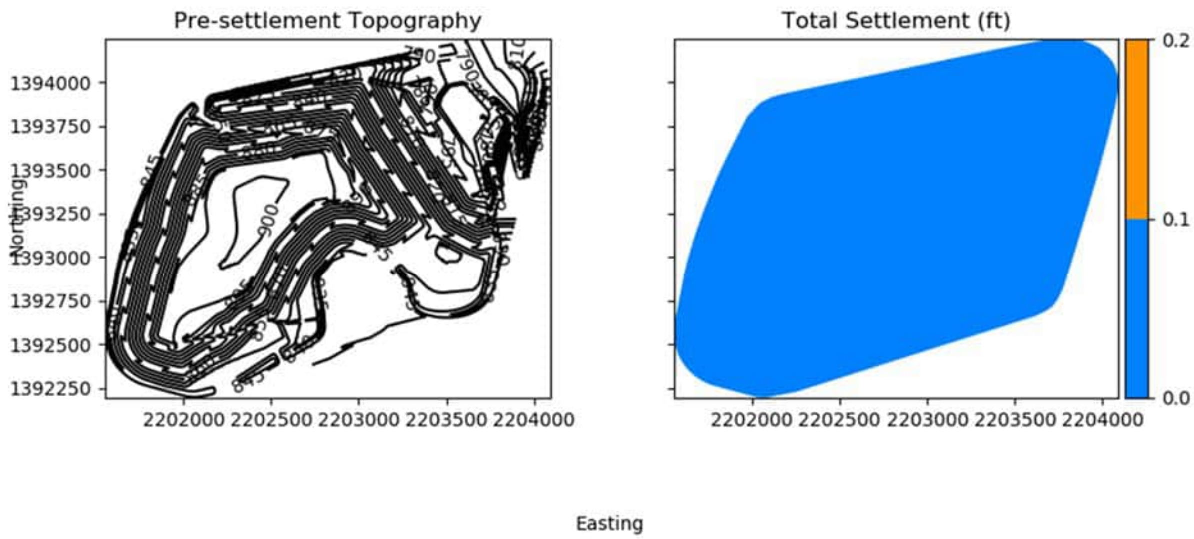


Figure 1. Settlement analysis for AP3/4.

4.0 REFERENCES

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

APPENDIX F

Under Slope Drainage System

GOLDER	**SUBJECT:** **Under Slope Drainage System Capacity** **Project Number:** **1777449** **Project Name:** **Plant McDonough Surface Impoundment Units AP-3/4 Closure** **Prepared by:** LS **Checked by:** GLH **Date:** Jul 2018 **Reviewed by:** GLH

1.0 OBJECTIVE

To evaluate the minimum capacity of the under slope drainage system of the final closure of AP-3/4 at Plant McDonough-Atkinson (Plant McDonough) located in Cobb County, GA. This includes the capacity evaluation of the geocomposite drain, sand trench drains, and drainage pipes. Component details for the under slope drainage system can be found on Sheet 8 of the Plant McDonough-Atkinson AP-2 and AP-3/4 Closure Drawings.

2.0 BACKGROUND

An under slope drainage system has been proposed for the collection and conveyance of contact water that collects along the eastern face and toe of the eastern slope of the proposed closed design for combined unit AP-3/4. The under slope drainage system (Figure 1 and also located on Closure Plan Sheet 8) has been proposed to:

- Collect potential seepage from the existing ash; and
- Serve as a drainage layer for water that has contacted the ash and relocated ash during the pond closure.

There are four main components of the under slope drainage system:

- Geocomposite strips
- 3 ft. by 3 ft. sand and gravel trench drains
- Perforated HDPE contact water collection pipes
- Under Slope Collection Sump

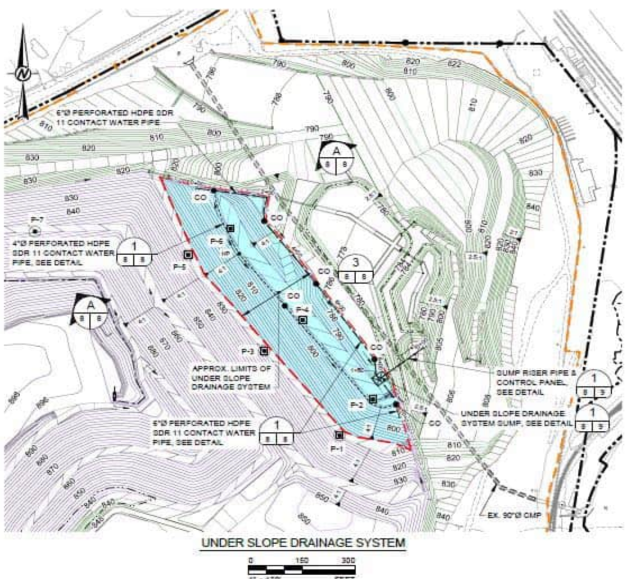


Figure 1 - Under Slope Drainage System (within blue shaded area)

3.0 GEOCOMPOSITE CAPACITY

Geocomposite strips are designed as a back up protective component of the under slope drainage system in order to convey contact water within the eastern limits of AP-3/4 that potentially migrates to the face of the ash cut slope to the under slope drainage system sump. The capacity of the geocomposite strips proposed for the under slope drainage system is based on current state of the practice and reduction factors to evaluate the proposed system. Based on the specified transmissivity of the geocomposite identified for use for the closure of Plant McDonough Closure of Ash Pond 1, 3 and 4 Technical Specifications of 9.0 x 10^{-4} meters squared per second (m^2/sec), the design transmissivity of the geocomposite strips layer under the soil buttress as part of the under slope drainage system is calculated based on the recommendations provided in GRI GC8 "Determination of the Allowable Flow Rate of a Drainage Geocomposite".

3.1 Calculations

$$T_{allow} = T_{ult} \times \frac{1}{RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}}$$

where:

T_{allow} = minimum allowable flow rate or transmissivity

T_{ult} = ultimate (design or as-manufactured) flow rate or transmissivity

RF_{IN} = 1.3 product

RF_{CR} = 1.2 Reduction Factor for Creep (based on loading)

RF_{CC} = 1.2 Reduction Factor for Chemical Clogging

RF_{BC} = 1.0 Reduction Factor for Biological Clogging

T_{ult} = 9.0E-04 m^2/sec therefore T_{allow} = 4.8E-04 m^2/sec

The equivalent hydraulic conductivity is k = T / thickness

thickness of geocomposite = 300 mil

Therefore:

k_{allow} = equivalent hydraulic conductivity = 6.31 cm/sec

3.2 Design

Geocomposite will be laid in four separate sections along the eastern slope of the unit as part of the under slope drainage system: Two sections are located along the first tier of the eastern slope, and two sections are located along the second tier of the eastern slope (Figure 2). These geocomposite sections will be approximately 60 feet apart (15 feet of vertical separation). The geocomposite strip sections will be approximately 15 feet wide. Geocomposite quantities in the underdrain system are as follows:

Table 1: Geocomposite in Under Slope Drainage System Quantities

Geocomposite Section	Length (ft)	Surface Area (ft^2)	Strip Location
Section A	645	9,675	Lower Strip - Lower Bench
Section B	730	10,950	Middle Strip - Lower Bench
Section C	780	11,700	Lower Strip - 2nd Bench
Section D	845	12,675	Middle Strip - 2nd Bench

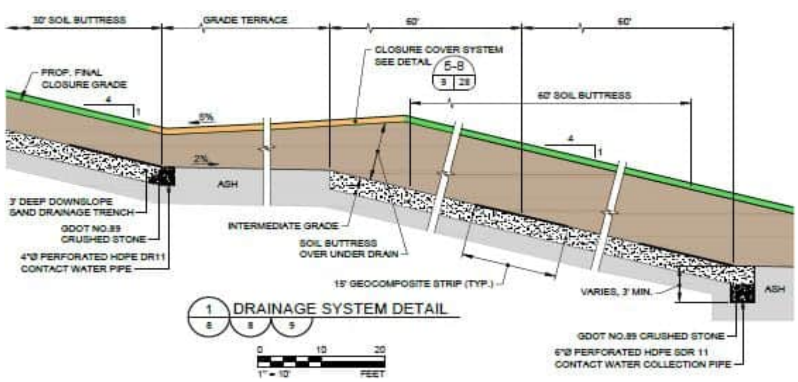


Figure 2 - Under Slope Drainage System Section & Detail

4.0 SAND TRENCH DRAINS

The under slope drainage system includes the use of downslope sand drainage trenches for collection and conveyance of contact water. These trench drains have a proposed width of 3 ft and depth of 3 ft, with a typical horizontal spacing of 25 ft between each sand trench along the slope (Detail B of Closure Plan Drawings Sheet 8). The hydraulic conductivity has been estimated for the #10 sand (GDOT) based on GDOT specifications.

Material	k (cm/sec)	k (ft/sec)
GDOT #10 Sand	0.03	9.84E-04

4.1 Calculations

The design capacity for the trench drains will be evaluated based on Darcy's Law. Darcy's Law states that the hydraulic conductivity (k) is related to the flow rate (q), cross-sectional area of the drain (A), length of flow (l) and the change in total head (DH) by the following relationship:

$$\rightarrow \quad q = \frac{k \times \Delta H \times A}{l} \qquad\qquad k = \frac{ql}{A\Delta H}$$

There are two groupings of the sand drainage trenches proposed in the design. Trenches terminating at the 6-inch dia. perforated HDPE contact water collection pipe, and trenches terminating at the 4-inch dia. perforated HDPE contact water collection pipe. The estimated cumulative lengths for sand drainage trenches are summarized in Table 2 below:

Table 2: Sand Trench Drain Capacities in Under Slope Drainage System

Trench Drains	Cumulative Length (ft)
Terminating at 6" dia. contact water pipe	1,084
Terminating at 4" dia. contact water pipe	3,704
Total Trench Drains	**4,788**

5.0 CONTACT WATER DRAINAGE PIPES

The under slope drainage system includes perforated HDPE SDR 11 contact water collection pipes (Figure 1 and Closure Permit Plan Sheet 8) to convey contact water collected by the geocomposite drains and sand drainage trenches to the under slope collection sump. As shown on the Closure Permit Plan, the 4-inch dia. perforated HDPE contact water collection pipe at the 40 ft. terrace conveys contact water from geocomposite and sand trench drain drainage components, and the northern half of the pipe conveys water to the 6-inch dia. contact water pipe. The 6-inch dia. perforated HDPE contact water collection pipe at the toe of the eastern slope conveys water from the geocomposite and sand trench drain drainage components, and the 4-inch dia. contact water pipe. The drainage pipes will capture flow from the geocomposite and sand drains via the perforations, and discharge the flow at the under slope sump.

5.1 Calculations for Contact Water Drainage Pipe Capacity

Manning's Equation was utilized for the calculation of velocity, and in turn the capacity in the pipe for the given parameters, as it is assumed that open channel flow exists in a pipe when flowing partially full.

$$v = \frac{1.49}{n} r^{2/3} s^{1/2}$$

where: r = hydraulic radius (or D/4) (ft)
s = slope of drainage pipe (ft/ft)
n = Manning's roughness coefficient, 0.008 - 0.011 for HDPE pipes
For these calculations, an n-value of 0.009 was assumed.

Table 3 below summarizes the capacities of the drainage pipes specified in the under slope drainage system. Table 4 evaluates the capacity of these pipes during the 25-year, 24-hour and 100-year, 24 hour storm events considered for the design.

Table 3: HDPE Drainage Pipe Capacities in Under Slope Drainage System

Drainage Pipe	Pipe Diameter (ft)	n	A (ft^2)	S (ft/ft)	V (ft/s)	Capacity (Q, cfs)	Capacity (GPM)	Storage in Pipe (ft^3)
6-inch dia. contact water pipe	0.5	0.009	0.20	0.009	3.9	0.77	346.0	153.2
4-inch dia. contact water pipe (draining north)	0.33	0.009	0.09	0.02	4.5	0.39	175.0	74.2
4-inch dia. contact water pipe (draining south)	0.33	0.009	0.09	0.03	5.5	0.48	214.3	
Drainage Pipe Capacity to Drainage Sump						**1.25**	**560.3**	**227.3**

Table 4: Capacity of Drainage Pipes for 24 Hours

Drainage Pipe	Duration (hr)	Capacity (ft³)	Capacity (gal)
4-inch dia. contact water pipe (draining north)	24	33,680	251,923
4-inch dia. contact water pipe (draining south)	24	41,249	308,541
6-inch dia. contact water pipe	24	66,611	498,253

6.0 DRAINAGE SUMP CAPACITY

The proposed contact water drainage sump (Closure Permit Plan Sheet 9) is the collection point where contact water from the drainage appurtenances summarized in Table 4 is routed to and collected via the contact water riser pipe and submersible pump. Contact water sump parameters are summarized below.

Table 5: Summary of Drainage Sump Parameters

	Length (ft)	Width at Base (ft)	Width at Top (ft)	Capacity of Sump (ft³)	Standing Volume (ft³)	Pumpable Volume (ft³)
Contact Water Sump	29	10	34	2,552	638	1,914

The contact water sump has been outfitted with an underdrain sump pump, capable of managing 30 gallons per minute (GPM) and able to pump a dynamic head of 90 vertical feet. Contact water sump and pump performance parameters are summarized below. The long term steady state groundwater conditions at the site were estimated using MODFLOW and estimated the long term steady state flow into the underdrain system as 7 GPM in the long term after steady state is reached.

Table 6: Summary of Drainage Sump Pump and Contact Sump Performance Parameters

Pump Location	Pipe Diameter	Pump Flow (GPM)	Head (ft)	Modelled Flow from Under Slope Drainage System in Long Term (GPM)	Capacity in Sump (Pumping) (Gal)	Capacity in Sump if Not Pumping (minutes)
Drainage Sump Pump	2 in.	30	90	7	14,317	2,045

7.0 SUMMARY AND CONCLUSION

The proposed under slope drainage system is designed to control contact water seepage from the eastern slope of the closure conditions of Plant McDonough Combined Unit AP-3/4. The maximum seepage rate able to be managed by the under slope contact water drainage pipes designed is approximately 560 GPM. The modelled groundwater collection within the under drain slope in the long term is approximately 7 GPM, with the underdrain system effectively lowering ground water levels below the bottom of ash. If the pump system were to fail or require maintenance, the under drain sump has storage capacity equal to nominally 2045 minutes within the sump itself, and an additional 100 days of capacity within the slope piping system. In addition to the contact water drainage sump and drainage pipe system, the sand trenches and geocomposite drainage layers provide additional contact water storage capacity within the under slope drainage system if needed.

9.0 REFERENCES

Koerner, R. M. Designing with Geosynthetics. 3rd ed. Englewood Cliffs, N.J.: Prentice Hall,1994.

Koerner, R.M., Koerner, G.R. Reduction Factors Used in Geosynthetic Design, GSI White Paper #4. Rev. 2007. Geosynthetic Research Institute.

Technical Specifications, Earthwork and Final Cover Installation for Closure of Ash Pond 1, 3 and 4, Plant McDonough. Georgia Power Company, 2015.

APPENDIX G

Alternative Final Cover Evaluation

CALCULATIONS

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

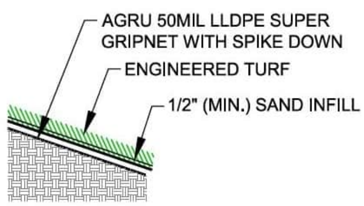
PROJECT: GEORGIA POWER COMPANY– PLANT MCDONOUGH-ATKINSON CCR UNIT AP-1 AND COMBINED CCR UNIT AP-3/4 CLOSURE

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)).

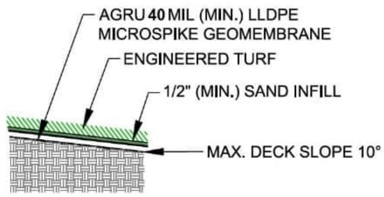
2.0 ANALYSIS PARAMETERS AND CONDITIONS

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



CLOSURETURF FINAL COVER WITH MICROSPIKE DETAIL

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.

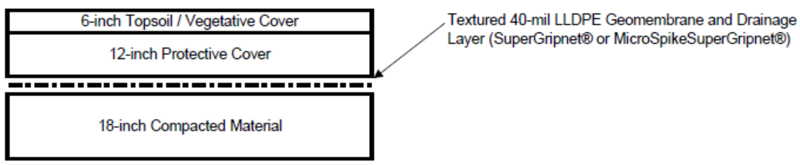


Figure 3 – Soil & Liner Closure System at Top Deck

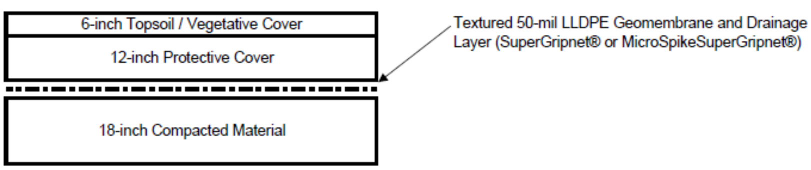


Figure 4 – Soil & Liner Closure System at Side Slopes

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.

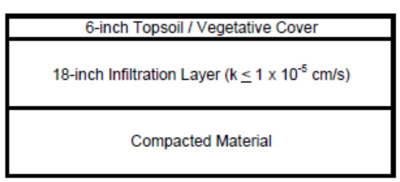


Figure 5 – CCR Unit Final Cover

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 (ft^3/day/acre)	Average Annual Percolation (ft^3/year/acre)	Maximum Percolation (ft^3/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 (ft^3/day/acre)	Average Annual Percolation (ft^3/year/acre)	Maximum Percolation (ft^3/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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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

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


     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
                              --------
```

```
             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

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	0.0403	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

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


     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
                                  --------
```

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                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

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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	0.055	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 16.16854 (  4.81010)     58691.816   33.26953
   FROM LAYER  3

 PERCOLATION/LEAKAGE THROUGH  0.04707 (  0.02786)       170.861     0.09685
   LAYER  5
```

AVERAGE HEAD ON TOP OF LAYER 4	0.222 (	0.137)		
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.04705 (	0.02790)	170.785	0.09681
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              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


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

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


     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

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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

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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

****

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

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

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
                                  --------
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           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 FROM LAYER 3	14.59292	(3.47176)	52972.297	30.02741
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.42464	(0.19292)	1541.441	0.87377

AVERAGE HEAD ON TOP OF LAYER 4	2.030 (	0.938)		
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.43162 (	0.16306)	1566.789	0.88814
CHANGE IN WATER STORAGE	-0.045 (	1.4563)	-162.32	-0.092

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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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**              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:\MCDCCR3.D10
OUTPUT DATA FILE:           C:\MCDCCR3.OUT


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


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     TITLE:  Plant McDonough CCR 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  =    0.1824 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
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 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 %
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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

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

♀

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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APPENDIX H

Veneer Stability Analyses Calculation Package

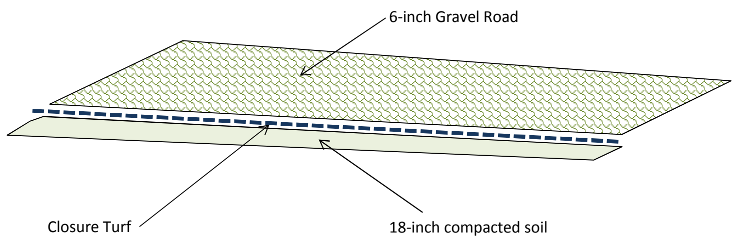
GOLDER	SUBJECT: Stability of Cover System - Veneer Stability				
	Job No. 1777449	Prepared by	DM		
	Ref. : Plant McDonough-Atkinson Closed CCR Surface Impoundment Units AP-1 and AP-3/4	Checked by	LJ / LS	Date	7/19/2018
		Reviewed by	GLH		

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 x cos β) = 578
(C + W_p x tan ϕ) = 69
cos β = 0.98
sin β = 0.17
sin β x tan ϕ = 0.13
$\sin^2$ β x tan ϕ = 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} x sin β + Fe) cos β
b = - [(Na+e x tan δ + Ca) x cos β + [(W_{a+e} x sin β + Fe) x sin β x tan ϕ] + (C + W_p x tan ϕ)]
c = (N_{a+e} x tan δ + C_a) x sin β x tan ϕ

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 = γ x h^2 x (L/h - 1/sin β - tan β / 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} x cos β
C_a = c_a x (L - h/sin β)
W_p = (γ x h^2) / sin 2β
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

C_a (lbs/ft) = 298 x c_a

W_p (lbs/ft) = 95

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

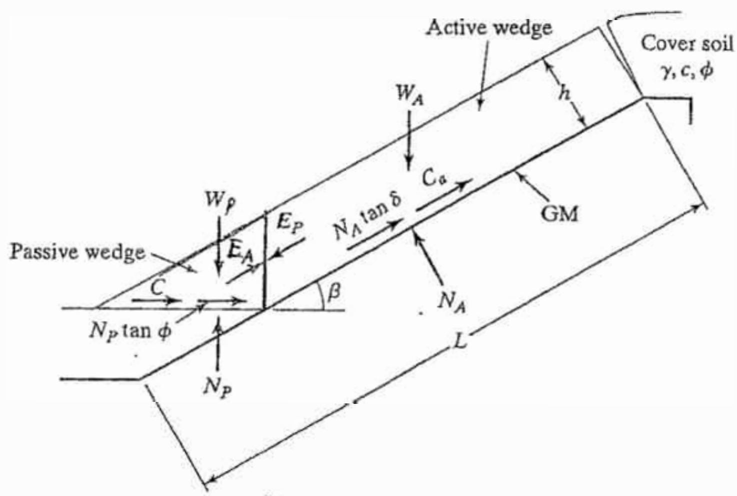


FIGURE 13.3 Limit Equilibrium Forces Involved in a Finite Length Slope Analysis for a Uniformly Thick Cover Soil

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 $a = (W_A - N_A \cdot \cos\beta) \cdot \cos\beta$

$b = -[(W_A - N_A \cdot \cos\beta) \cdot \sin\beta \cdot \tan\phi + (N_A \cdot \tan\delta + C_a) \cdot \sin\beta \cdot \cos\beta + (C + W_P \cdot \tan\phi) \cdot \sin\beta]$

$c = (N_A \cdot \tan\delta + C_a) \cdot \sin^2\beta \cdot \tan\phi$

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

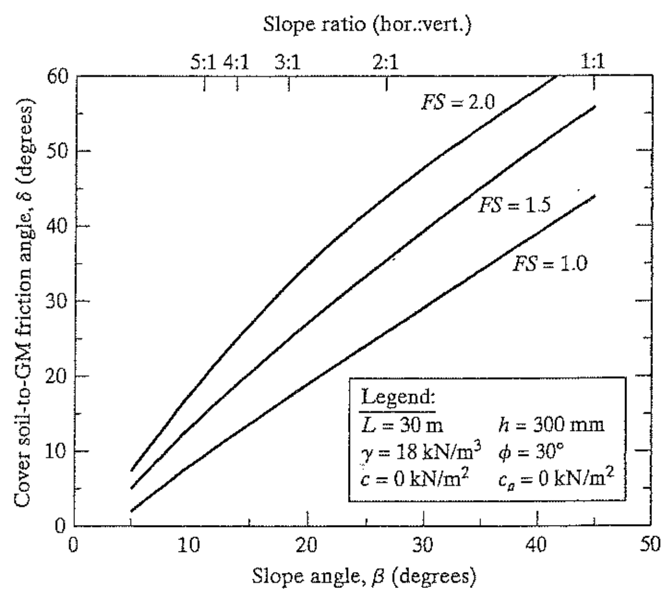


FIGURE 13.4 Design Curves for Stability of Uniform-Thickness Cohesionless Cover Soils on Linear Failure Planes for Various Global Factors of Safety

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.

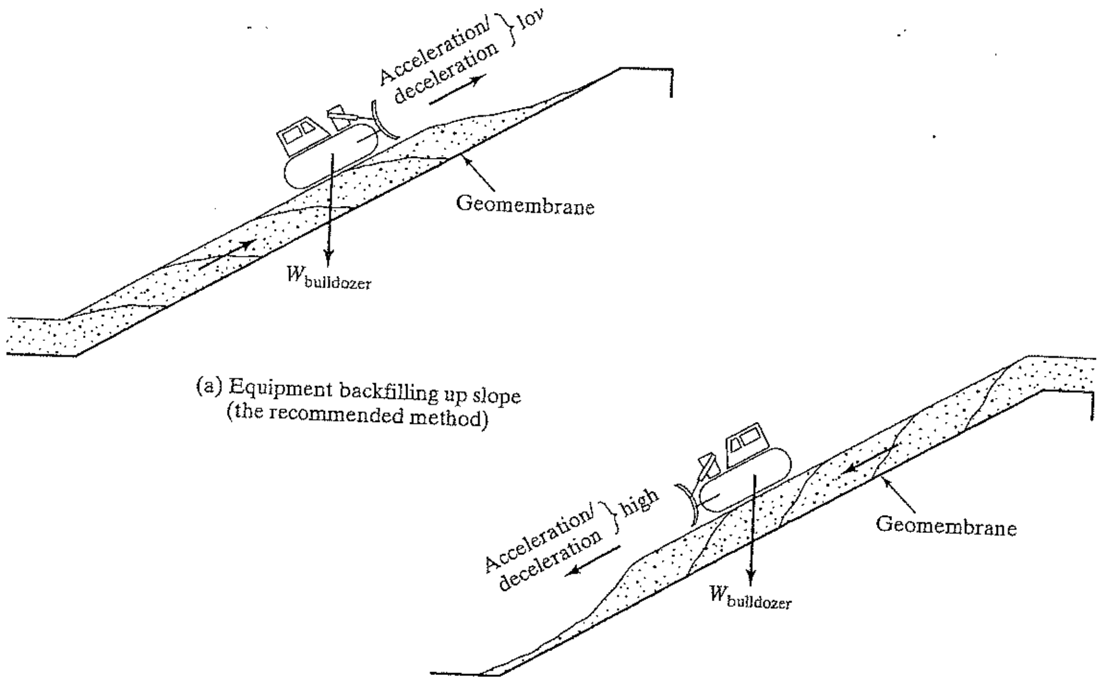
Comment In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. There are many possible options to increase the value (e.g., changing the geometry of the situation, the use of toe berms, tapered cover soil thickness, and veneer reinforcement, see Koerner 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



(a) Equipment backfilling up slope (the recommended method)

(b) Equipment backfilling down slope (method is not recommended)

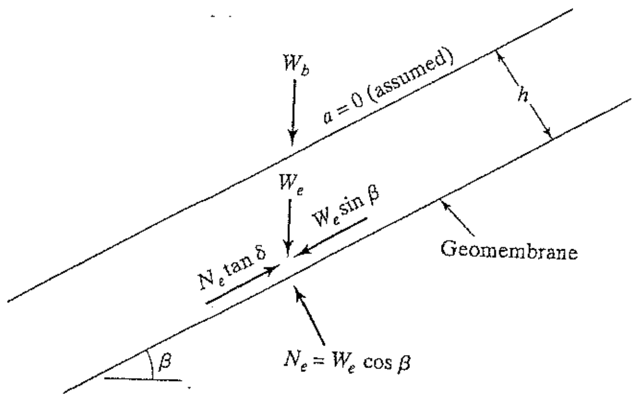
FIGURE 13.5 Construction Equipment Placing Cover Soil on Slopes Containing Geosynthetics

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

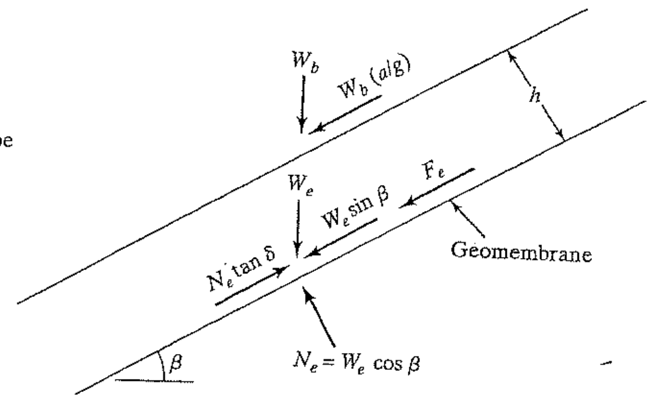
$$W_e = q \cdot w \cdot I \tag{13.10}$$

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



(a) Equipment moving up slope (load with no acceleration)



(b) Equipment moving down slope (load plus acceleration)

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.

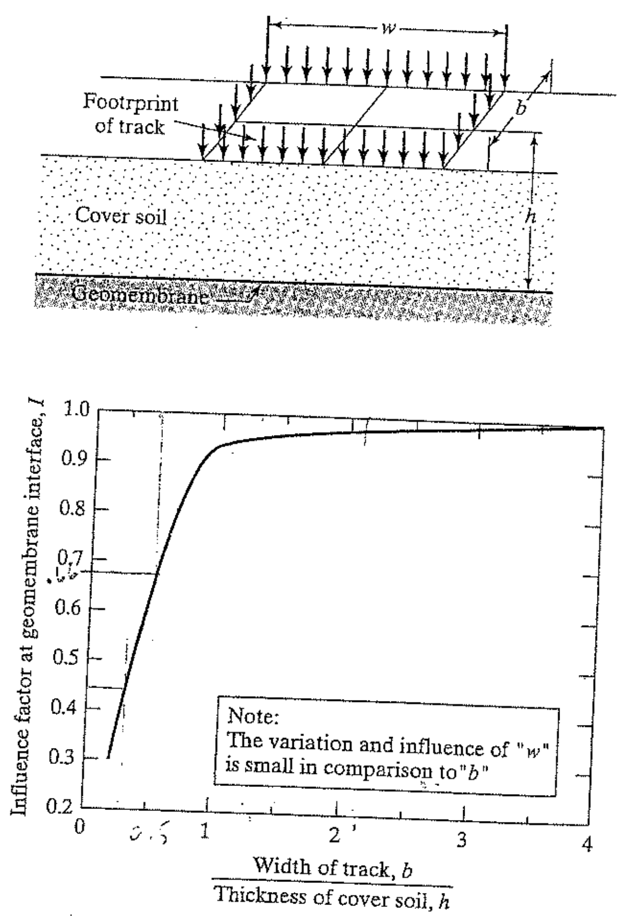


FIGURE 13.7 Values of Influence Factor, "I", for Use in Equation 13.10 to Dissipate Surface Force through the Cover Soil to the Geomembrane Interface (after Soong and Koerner, 1996)

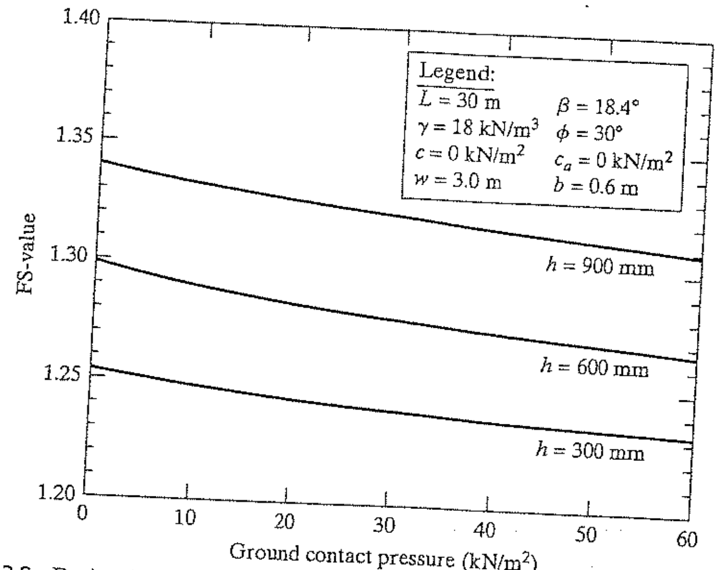


FIGURE 13.8 Design Curves for Stability of Different Thickness of Cover Soil for Various Construction Equipment Ground Contact Pressure

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

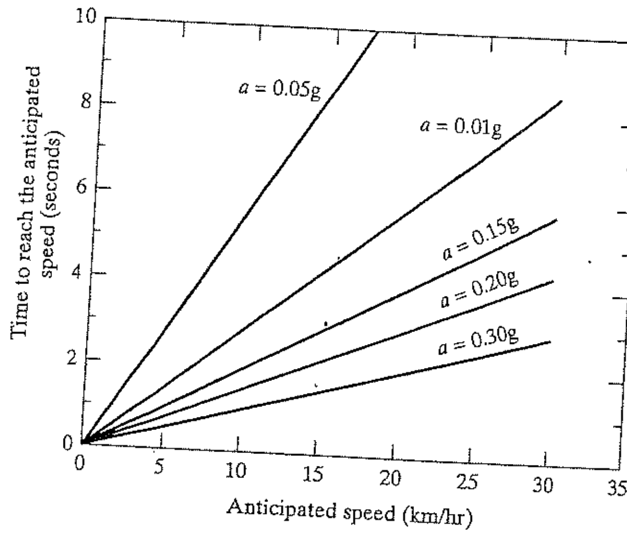


FIGURE 13.9 Graphic Relationship of Construction Equipment Speed and Rise Time to Obtain Equipment Acceleration.

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

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

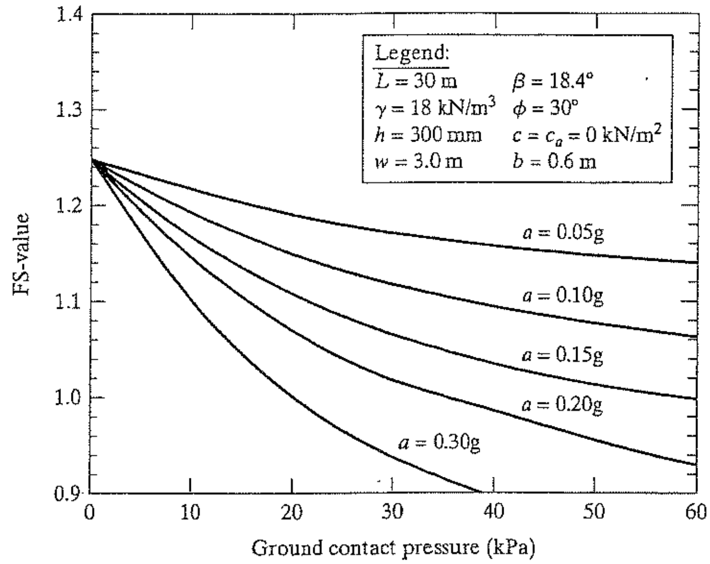


FIGURE 13.10 Design Curves for Stability of Different Construction Equipment Ground Contact Pressure for Various Equipment Accelerations

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

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

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

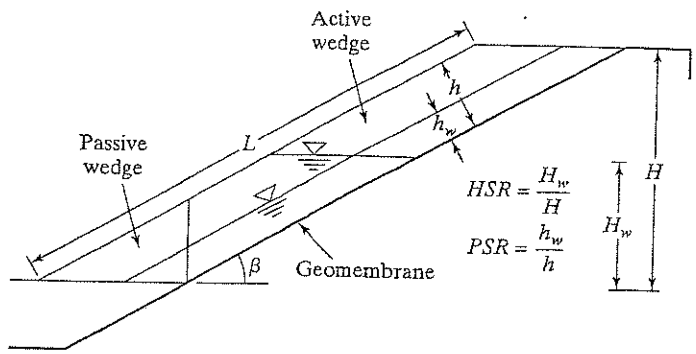


FIGURE 13.12 Cross Section of a Uniform Thickness Cover Soil on a Geomembrane Illustrating Different Submergence Assumptions and Related Definitions (Soong and Koerner, 1996)

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,

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

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

$$U_h = 0.5\cdot\gamma_w\cdot h^2 \qquad (13.16)$$

$$N_A = W_A\cdot\cos\beta + U_h\cdot\sin\beta - U_n \qquad (13.17)$$

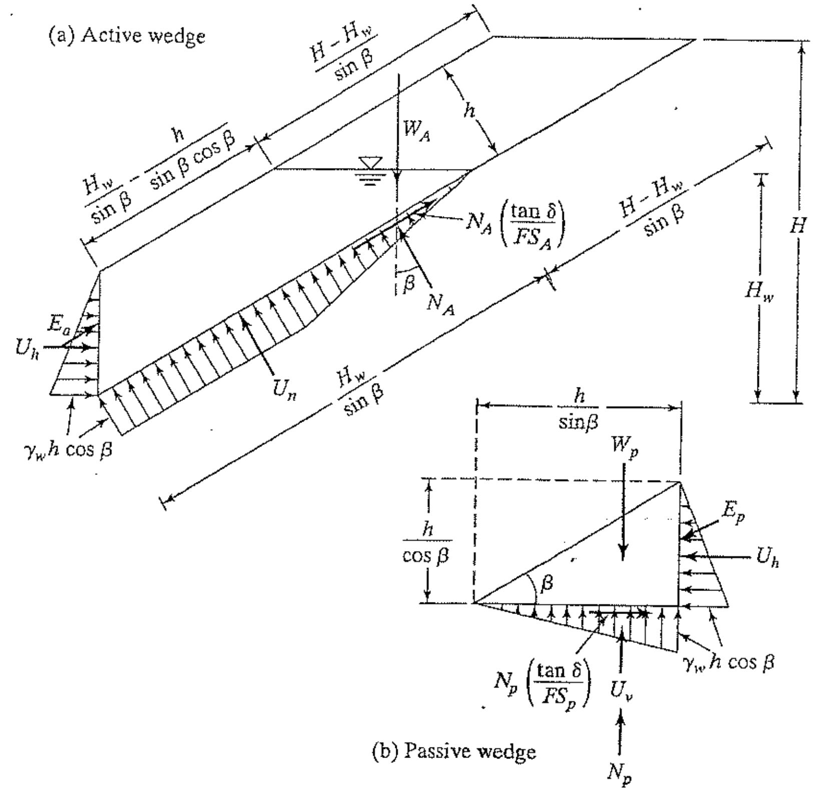


FIGURE 13.13 Limit Equilibrium Forces Involved in a Finite Length Slope of Uniform Cover Soil with Horizontal Seepage Buildup

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/

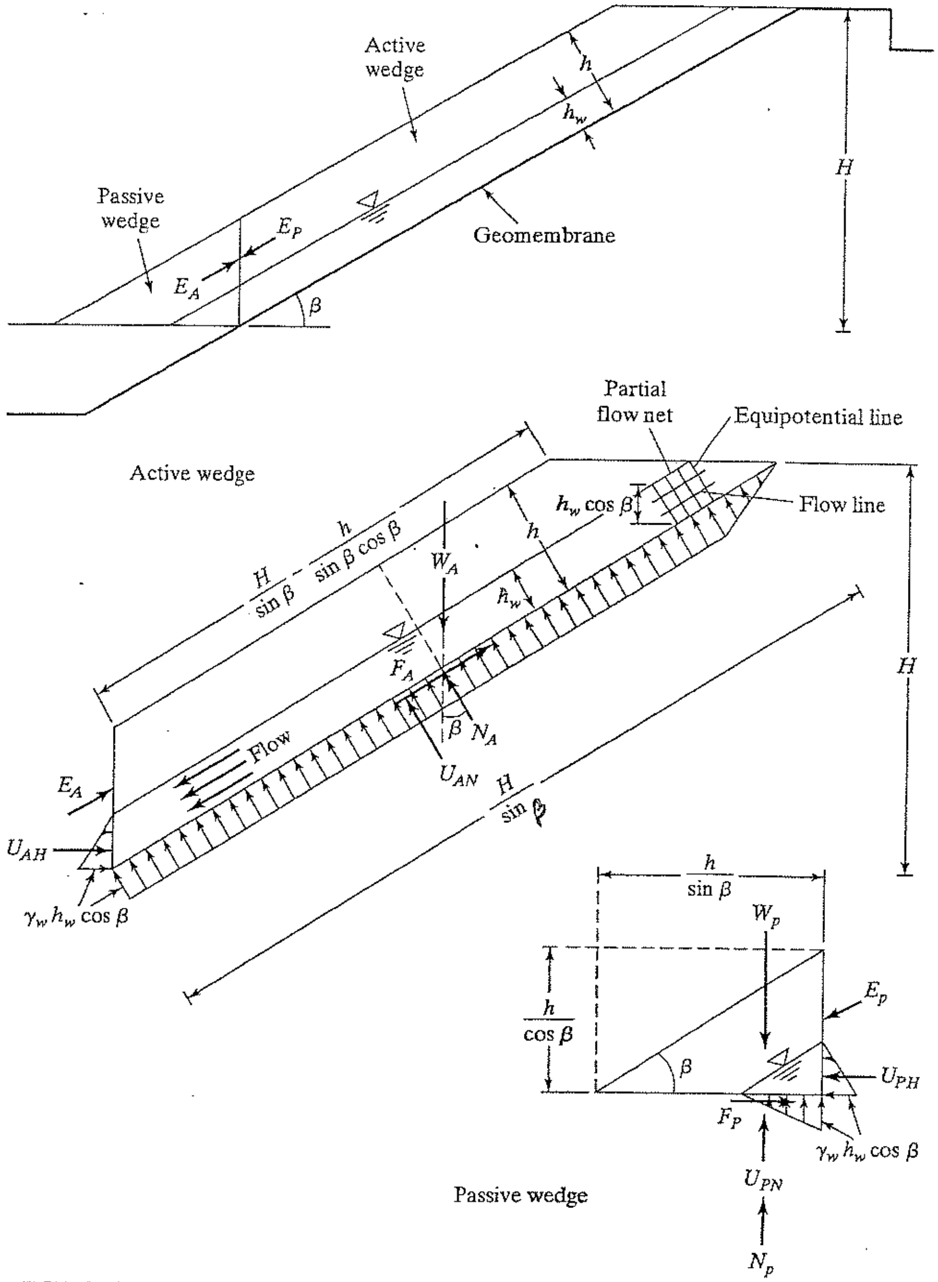


FIGURE 13.14 Cross Section of Sand Layer over Geomembrane on Side Slope with Seepage Parallel to Slope.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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,

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

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

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

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

$$11{,}766 \pm [(-11{,}766)^2 - (4)(9{,}071)(1{,}963)]^{0.5}$$

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

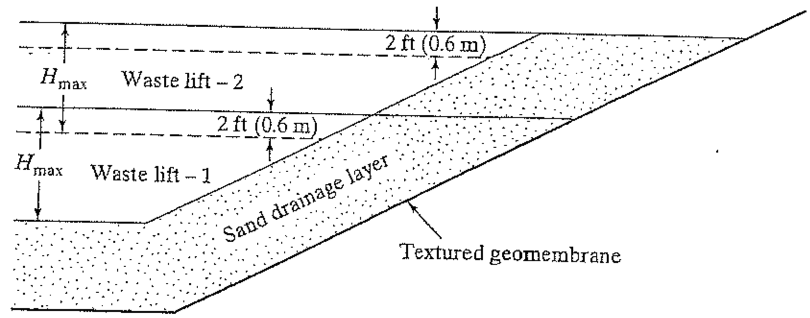


FIGURE 13.16 Incremental Placement of Soil Drainage Layer on Sideslope

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), γ = 110 lb/ft^3 (17.3 kN/m^3), γ_{sat} = 115 lb/ft^3 (18 kN/m^3), γ_w = 62.4 lb/ft^3 (9.81 kN/m^3), ϕ = 32°, δ = 22°.

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

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\ h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$
$$= (62.4)(0.5)[16 - (0.5)(0.5)(0.949)]/(0.333) = 1{,}476.9 \text{ lb/ft } (20.90 \text{ kN/m})$$

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

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

$$W_A = 0.5 \cdot [\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) + \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta) \tag{13.40}$$
$$= (0.5)\{(110)(2 - 0.5)[(2)(16)(0.949) - 2 - 0.5] + (115)(0.5)[(2)(16)(0.949) - 0.5]\}/[(0.316)(0.949)]$$
$$= (0.5)(4{,}598.22 + 1{,}717.41)/[(0.316)(0.949)] = 10{,}530.1 \text{ lb/ft } (148.9 \text{ kN/m})$$

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

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"
.7?	8'11"
.(	13'4"
.9b ...	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

ior. 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 I

Anchor Trench Calculation Package

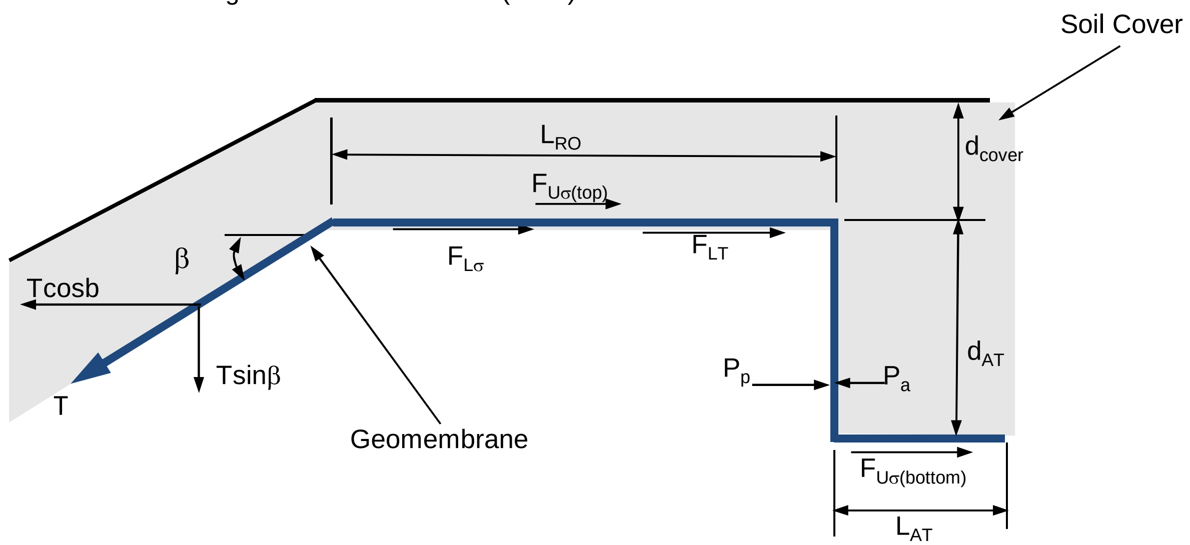
Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 4:1 Slope	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

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

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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:

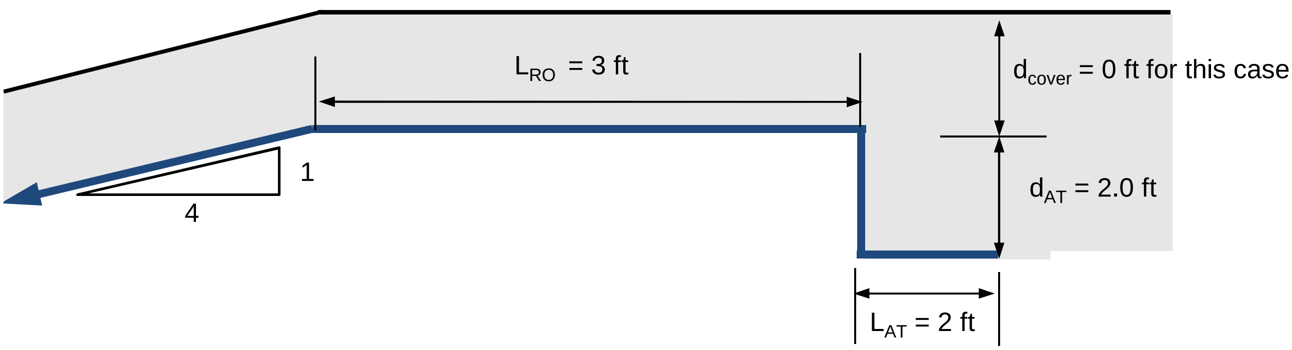
Min Calculated L_{AT} = 0.3 m 0.8 ft Minimum anchor trench width must be greater than calculated minimum

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		

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.

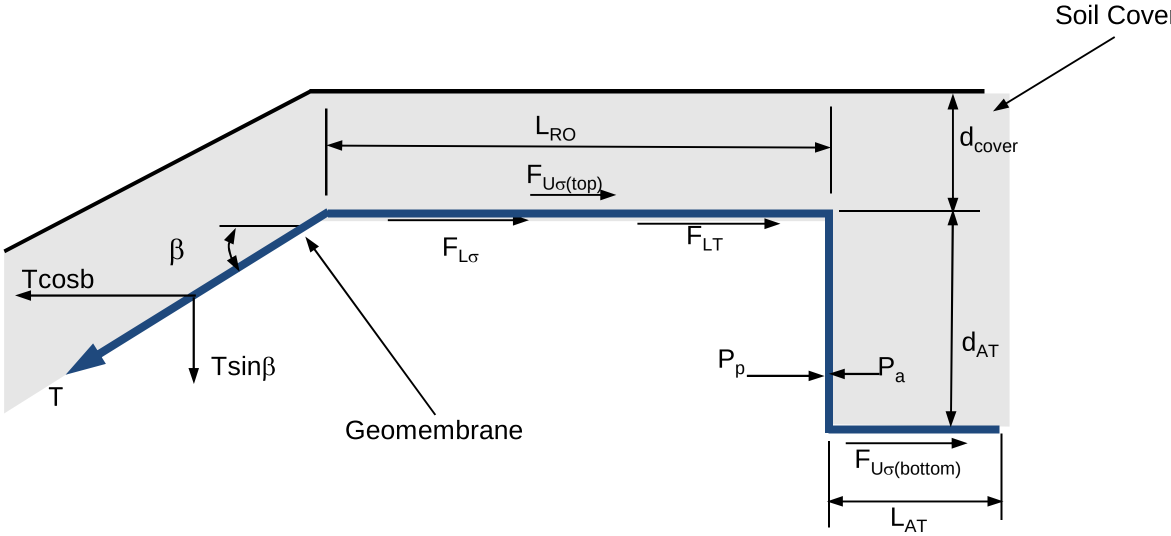
Date:	July 1, 2018	Made by:	DM
Project No.:	1777449	Checked by:	LJ / LS
Subject:	Anchor Trench Design - Top of Slope for 2.5:1 Case	Reviewed by:	GLH
Project:	Plant McDonough AP-1 and AP-3/4 Closure Design		

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

γ_{AT} = unit weight of soil in anchor trench

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

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:

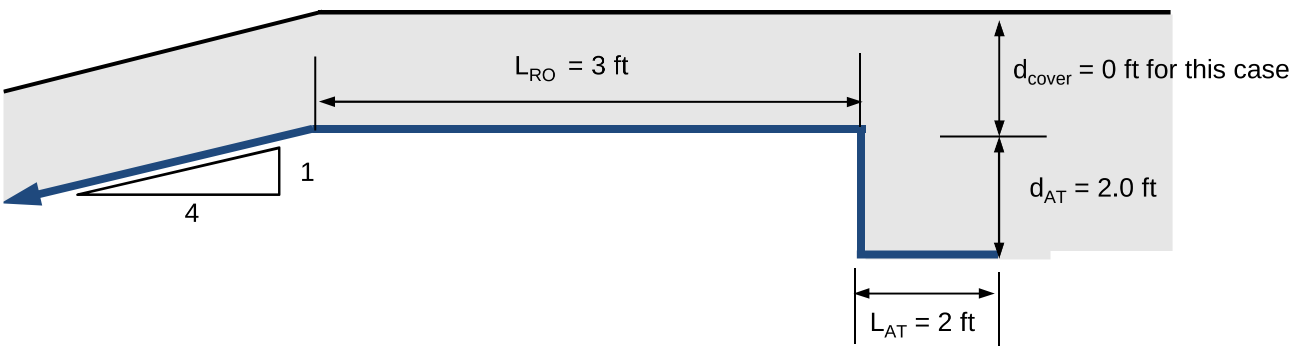
Min Calculated L_{AT} = 0.1 m 0.2 ft — Minimum anchor trench width must be greater than calculated minimum

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.

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					
Strength @ Break, lb/in width (N/mm)	2 in/minute	20,000 lb	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 mil	THICKNESS mm	WIDTH ft	WIDTH m	LENGTH ft	LENGTH m	AREA (APPROX.) ft²	AREA (APPROX.) m²
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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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 mil	THICKNESS mm	WIDTH ft	WIDTH m		LENGTH ft	LENGTH m	AREA (APPROX.) ft²	AREA (APPROX.) 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).

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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Revision Date: March 21, 2018

APPENDIX J

Hydrology and Hydraulic Design for AP-3/4

CALCULATIONS

Date:	01 November 2017	**Made by:**	Jimmy Grimes
Project No.:	1539180 – SCS Project ID MCD15017	**Checked by:**	Joshua K. Myers
Subject:	Final Closure Hydrology and Hydraulic Design	**Reviewed by:**	Gregory L. Hebeler
Project:	**SOUTHERN COMPANY / MCDONOUGH ASH PONDS 3 AND 4 CLOSURE / GA**		

1.0 OBJECTIVE

The objective of this memo is to outline the design process and present engineering calculations for the proposed storm water system of the Plant McDonough Ash Pond 3 and 4 closure landfill.

2.0 METHODOLOGY

Golder is developing a hydrologic and hydraulic model within the AutoCAD Civil 3D Storm and Sanitary Analysis (SSA) program to analyze the proposed landfill closure site. Proposed grading information has been created in order to remove all ash above the existing ash pond pipe culvert (see Figure 1) and relocate the ash to the western portion of the site. For the landfill cap, SCS/GPC have chosen the use of AgruTurf closure turf, a non-permeable liner consisting of fiber "grass" strands and a sand infill overlying an integrated geomembrane or structured geomembrane. Because the liner is non permeable, almost all rainfall on the site will be directly generated into storm runoff. Golder proposes a series of three permanent detention ponds to attenuate this runoff while leaving existing outfall infrastructure in place. The first outfall from the proposed closure site will remain the existing ash culvert, which will handle flow from the basin north of the closure site (see previous technical memo documenting the analysis of this basin) as well as flow from the landfill via an outflow pipe culvert in detention pond 2 (previous calculations had presented a riser structure which is now changed to a culvert with headwall). The second outfall from the closure site will be flow exiting the riser structure of detention pond 3.

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3.0 PRECIPITATION

NOAA's Atlas 14 is used to determine storm depths for 24 hour storms ranging from the 2 year to 100 year storm events as shown in Table 1. An SCS Type II distribution was used in all subsequent modeling efforts. The design storm used to size all storm infrastructure was taken as the 100 year, 24 hour storm event.

Table 1: 24 Hour Storm Depths

Storm Event	Depth (in)
2 year	3.73
5 year	4.45
10 year	5.09
25 year	6.00
50 year	6.74
100 year	7.52
500 year	9.47
1000 year	10.40

4.0 FINAL POND STAGE STORAGE

Golder proposes three permanent detention ponds to provide storage capacity and attenuation of floods. Detention Pond 1 and Detention Pond 3 will also serve as sediment basins during various construction phases. Figure 1 shows the location of each pond on site. Tables 2 through 4 give the stage storage curve for each pond.

Table 2: Stage-Storage Curve for Detention Pond 1

Elevation	Area (ft^2)	Area (acres)	Volume (acre-ft)
824	20110	0.46	0.0
826	25336	0.58	1.0
828	34926	0.80	2.4
830	43341	0.99	4.2
832	54936	1.26	6.5
834	65604	1.51	9.2
836	76014	1.75	12.5
838	86821	1.99	16.2

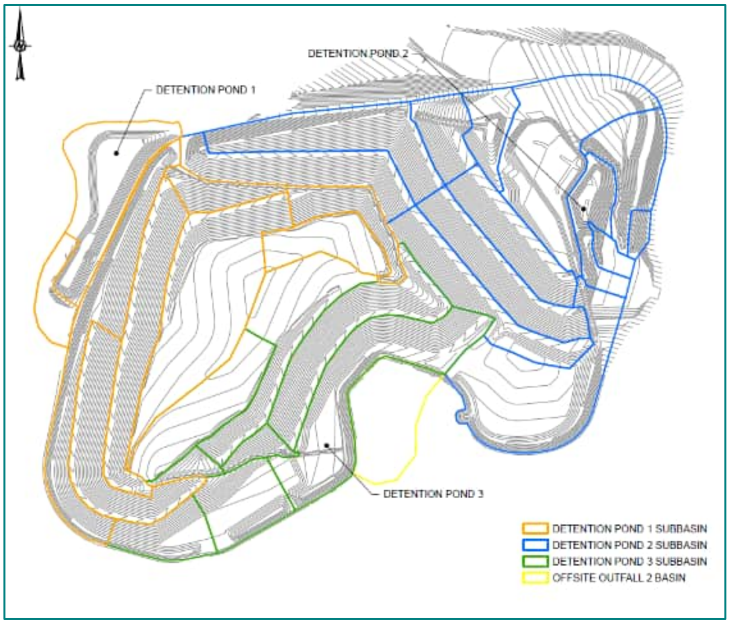
Table 3: Stage-Storage Curve for Detention Pond 2

Elevation	Area (ft^2)	Area (acres)	Volume (acre-ft)
771	0	0.00	0.0
772	1365	0.03	0.0
774	7864	0.18	0.2
776	11858	0.27	0.7
778	14533	0.33	1.3
780	17389	0.40	2.0
782	20420	0.47	2.9
784	23650	0.54	3.9

Table 4: Stage-Storage Curve for Detention Pond 3

Elevation	Area (ft^2)	Area (acres)	Volume (acre-ft)
832.8	100	0.00	0
834	2905	0.07	0.0
836	11594	0.27	0.4
838	27361	0.63	1.3
840	58791	1.35	3.2
842	80138	1.84	6.4
844	97141	2.23	10.5

5.0 HYDROLOGY

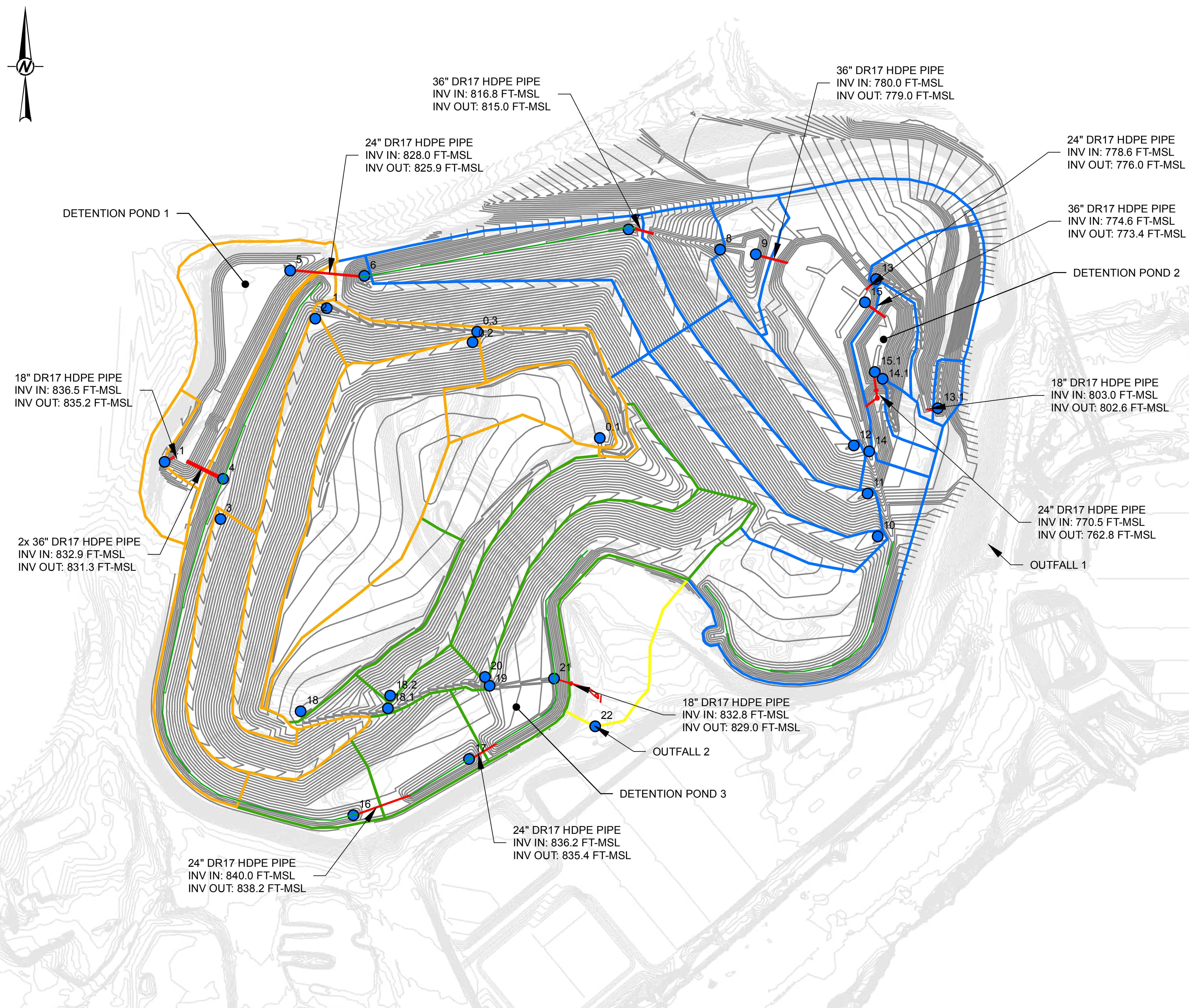


Site and Basin Layout; For Expanded View See Next Page

Golder has performed an analysis of the hydrology of the closure system. Watersheds are delineated at multiple "study points" (see Figure 1) so that each hydraulic component in the stormwater system can be sized and checked for adequate stormwater capacity. Curve numbers for each basin consisting of landfill cap are taken to be a value of 95 based on design guidelines provided by AgruTurf (see Attachment A). Areas of landfill which are not be enclosed with closure turf are also taken to be 95 in order to provide a conservative runoff estimate and to account for drainage patterns during construction before final grass cover has been established. Curve numbers for off-landfill basins are developed based on existing ground cover conditions and type B soils. Time of concentration values are calculated via the velocity method (see Attachment D). A minimum time of concentration of six minutes was used as recommended by the TR-55 manual (see Attachment B). Storm runoff values were taken directly from the SSA model.

5.1 Detention Pond 1

The total watershed contributing to Pond 1 is divided into eight sub-basins. Pond 1 is fed by a culvert at point 4 which transmits runoff from the western side of the landfill. Table 5 shows the hydrologic parameters associated with each basin.



LEGEND

- PROPOSED 2' CONTOURS
- EXISTING 2' CONTOURS
- PROPOSED CULVERT
- OFFSITE OUTFALL 2 BASIN
- DETENTION POND 1 SUBBASIN
- DETENTION POND 3 SUBBASIN
- DETENTION POND 2 SUBBASIN

300 150 0 300 Feet

NOTES

REFERENCE

THE EXISTING TOPOGRAPHY AND CONTOUR ELEVATIONS WERE PROVIDED BY GEORGIA LAND DEPARTMENT AND METRO ENGINEERING AND SURVEYING CO, INC. THE DATE OF THE SURVEY PROVIDED AND SHOWN ON THIS SET OF PLANS IS 10-16-2012. REFER TO THE SURVEY DRAWING TITLED "TOPOGRAPHIC MAP PREPARED FOR GEORGIA POWER COMPANY PLANT MCDONOUGH – GEORGIA STATE PLANE WEST SURVEY FEET – DATE OF PHOTOGRAPHY 10-26-12. PROJECT NO. 13225 – 01-13-2013."

CLIENT
GEORGIA POWER COMPANY/
SOUTHERN COMPANY SERVICES

PROJECT
PLANT MCDONOUGH
ASH POND NO. 3 & 4 CLOSURE PLAN

TITLE
WORKING STORMWATER DESIGN AND BASIN LAYOUT

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-02-02
PREPARED	JDG
DESIGN	JDG
REVIEW	JKM
APPROVED	GLH

PROJECT No.	CONTROL	Rev.	FIGURE
1539180		0	1

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

Table 5: Hydrology Parameters for Detention Pond 1

	Size (acres)	Time of Concentration (mins)	Curve Number	100 Year Storm Peak Runoff (cfs)
Basin 0.1	1.31	10.7	95	11
Basin 0.2	7.22	28.8	95	41
Basin 0.3	1.53	6.0	95	15
Basin 1	1.10	6.1	95	10
Basin 2	3.89	9.1	95	34
Basin 3	3.36	10.0	95	29
Basin 4	4.42	11.7	95	36
Basin 4.1	1.0	6.0	95	9.5
Basin 5	4.1	6.0	95	39

5.2 Detention Pond 2

The total watershed contributing to Pond 2 is divided into thirteen sub-basins. Outflow from Pond 1 flows through the northern segment of perimeter ditch and down the northern downslope channel into the pond. Flow generated from runoff on the east side of the landfill is directed into the pond via the southern downslope channel. Runoff from the northeast corner of landfill flows directly into the pond. Table 6 shows the hydrologic parameters associated with each basin.

Table 6: Hydrology Parameters for Detention Pond 2

	Size (acres)	Time of Concentration (mins)	Curve Number	100 Year Storm Peak Runoff (cfs)
Basin 6	3.22	12.8	95	26
Basin 7	4.00	6.0	95	38
Basin 8	1.71	7.8	95	16
Basin 9	2.56	6.0	95	24
Basin 10	2.82	7.4	95	26
Basin 11	2.76	7.8	95	25
Basin 12	2.45	6.9	95	23
Basin 13	2.39	6.0	95	5
Basin 13.1	0.34	6.5	95	12
Basin 14	5.61	18.8	95	40
Basin 14.1	0.47	6.0	95	4
Basin 15	5.40	13.1	95	45
Basin 15.1	1.62	6.0	95	15

5.3 Detention Pond 3

The total watershed contributing to Pond 3 is broken into eight sub-basins. Several sections of perimeter ditch and roadside channels contribute runoff from the south and west sections of landfill cap. Table 7 shows the hydrologic parameters of each sub-basin.

Table 7: Hydrology Parameters for Detention Pond 3

	Size (acres)	Time of Concentration (mins)	Curve Number	100 Year Storm Peak Runoff (cfs)
Basin 16	1.59	13.0	95	13
Basin 17	2.17	11.6	95	18
Basin 18	3.45	22.1	95	23
Basin 18.1	0.48	6.0	95	5
Basin 18.2	3.10	26.4	95	19
Basin 19	0.57	5.3	95	5
Basin 20	3.72	7.0	95	34
Basin 21	5.20	6.8	95	48

5.4 South Offsite Basin

Golder performed a hydrologic analysis of the basin contributing to the culvert beneath the road south of the pond 3 outlet. With the addition of outflow from detention pond 3, this culvert and basin are analyzed to ensure existing infrastructure could remain in place. Table 8 gives the hydrologic parameters for the south offsite basin.

Table 8: Hydrology Parameters for Offsite Basin

	Size (acres)	Time of Concentration (mins)	Curve Number	100 Year Storm Peak Runoff (cfs)
Basin 22	3.00	6.0	82	24

6.0 HYDRAULIC ANALYSES

Golder proposes a stormwater conveyance system to convey water off the landfill surface through a series of ditches, ponds, and culverts. Each component of the system was sized to meet minimum performance and freeboard criteria within the SSA model.

All culvert pipes are shown on the plans to be SDR26 HDPE pipes. For the purpose of this study, all pipes are assumed to be SDR17 pipes, as the contractor has requested to use SDR 17 pipes as conditions in the field dictate. As SDR17 pipes have reduced flow capacity, the entire system is modeled with this configuration in order to provide a conservative design and allow the contractor to use this pipe configuration as needed.

6.1 Detention Pond Outlet Structures

In detention Pond 1 and Pond 3 Golder proposes a riser structure to maximize storage potential while maintaining a minimum of 1 feet of freeboard during the 100 year storm event. The risers in Pond 1 and Pond 3 are designed to be 4 foot by 4 foot box risers. Each riser consists of a low flow conduit at the pond invert, which in each case is a 3" orifice. A mid-level weir and emergency level weir are present at heights which vary between each structure. For a detailed rating curve for each structure see Attachment C. The proposed outlet structure for detention Pond 2 consists of a concrete headwall and pipe culvert. The pipe culvert consists of a 24” SDR17 HDPE pipe which drains to a junction (manhole), after which a separate 24” HDPE pipe conveys flow into the existing culvert located beneath Ash Pond 4. The rating curve for the outlet culvert is calculated within the SSA model. Table 9 shows a summary of the characteristics and performance of each outlet structure.

Table 9: Summary of Pond Outlet Structures

	Pond 1	Pond 2	Pond 3
Low Level Conduit Elevation (ft-msl)	828.00	-	832.80
Top of Pond Elevation (ft-msl)	840.0	784.0	846.0
Weir Elevation (ft-msl)	836.0	-	841.0
Top of Riser/Emergency Weir Invert (ft-msl)	838.0	-	843.0
Weir Length (ft)	2.00	-	2.00
Emergency Weir Length (ft)	12.0	-	12.0
Outlet Pipe Size (ft)	2.0	2.0	1.5
100 Year Storm Max Inflow to Pond (cfs)	106	58	121
100 Year Storm Max Outflow (cfs)	12	30	9
100 Year Storm Max Water Level (ft-msl)	837.5	782.4	842.3
100 Year Storm Freeboard (ft)	2.5	1.6*	3.7
1000 Year Storm Max Inflow to Pond (cfs)	123	96	165
1000 Year Storm Max Outflow (cfs)	30	35	20
1000 Year Storm Max Water Level (ft-msl)	838.7	785.8	843.4
1000 Year Storm Freeboard (ft)	1.3	Overtopped*	2.6

*Pond 2 Freeboard calculated to the top elevation of the Detention Pond 2 splitter dike

6.2 Channel Capacity

Golder proposes a series of ditches to convey flow off of the landfill surface. Flow depths are taken directly from the SSA model. When a channel is not directly modeled in the SSA program flow depth was calculated using the manning equation (Equation 1) with a discharge equal to any direct discharge from the contributing basin. The manning's N values for hydroturf and armorflex are taken, respectively, from AgruTurf design guidelines (Attachment A) and factor of safety calculations provided by Armortech (Attachment E). Table 10 shows a summary of the various channel configurations in use throughout the system. A minimum freeboard of at least 1 foot is required in all perimeter ditches. In terrace channels there is no freeboard requirement (depth of terrace channel is 1 foot).

Table 10: Summary of Channel Type Geometries

	Channel Type	Base Width (ft)	Side Slope 1 (h:v)	Side Slope 2 (h:v)	Total Depth(ft)	Manning's N	Liner Type
Triangular Terrace	Type 1	N/A	4	20	1	0.030	Riprap
Trapezoidal Side Channel	Type 2	2.7	4	2.5	2	0.030	Riprap
Perimeter Channel	Type 3	4.0	2.5	4	4	0.069	Riprap
Downslope Channels	Type 4	4.0	3	3	4	0.025	Armorflex

$$Equation\ 1: Q = \frac{1.49}{n} S^{0.5} x\ A\ x\ R_h{}^{0.67}$$

Table 11: Summary of Available Freeboard in Each Channel

	Channel Type	Peak Flow (cfs)	Channel Slope (%)	Flow Depth (ft)	Freeboard (ft)
Terrace Channels					
Flow into Point 0.2	Type 1	41	3	0.6	0.4
Flow into Point 2	Type 1	34	3	0.9	0.1
Flow into Point 3	Type 1	29	3	0.5	0.6
Flow into Point 6	Type 1	26	3	0.5	0.5
Flow into Point 7	Type 1	38	3	0.6	0.4
Flow into Point 10	Type 1	26	3	0.4	0.6
Flow into Point 11	Type 1	25	3	0.4	0.6
Flow into Point 13.1	Type 1	15	6.8	0.2	0.8
Trapezoidal Side Channels					
Flow into Point 0.1	Type 2	11	1	0.3	1.7
Flow into Point 0.3	Type 2	25	5	1.2	0.8
Flow into Point 1	Type 2	97	5	1.8	0.2
Flow into Point 9	Type 2	24	8	0.6	1.4
Flow into Point 12	Type 2	23	11	0.6	1.4
Flow into Point 13	Type 2	5	0.08	1.1	0.9
Flow into Point 14.1	Type 2	4	20	.1	1.9
Flow into Point 18	Type 2	15	1	1.1	0.9
Flow into Point 18.1	Type 2	39	8	1.0	1.0
Flow into Point 18.2	Type 1	32	3	1.3	0.8
Flow into Point 19	Type 2	92	17	1.1	0.9
Perimeter Channel					
Flow into Point 4	Type 3	106	0.75	2.2	0.8
Flow into Point 7	Type 3	80	1	2.3	0.7
Downslope Channel					
Flow into Point 8	Type 4	59	16	.7	3.3
Flow into Point 14	Type 4	82	21	.8	3.2
Flow into Point 22	*	25	3	1.6	2.1

*Flow into point 22 runs through an existing paved channel and is taken directly from SSA model. Freeboard is calculated using a top of road elevation of 824.7 and channel invert elevation of 820.5 ft-msl.

6.3 Ditch Stability

Golder has checked each proposed conveyance ditch for its ability to withstand shear stress from flow within the channel with methodology from HEC-15. Equation 2 gives the shear stress for a straight channel. The hydroturf liner is reinforced with grout and is highly resistant to erosion. For a more detailed calculation regarding Armortech lined channels see Attachment E. Table 12 shows the calculated shear stress in each channel section.

$$Equation\ 2{:}\ \tau = \gamma d S$$

Table 12: Summary of Shear Stress on Each Ditch Lining

	Flow Height (ft)	Channel Slope (%)	Shear Stress (lb/ft^2)	Channel Lining	Permissible Shear Stress (lb/ft^2)	Safety Factor
Flow into Point 0.2	0.6	3.0	1.2	Riprap	4.6	3.8
Flow into Point 2	0.9	3.0	1.7	Riprap	4.6	2.7
Flow into Point 3	0.5	3.0	0.8	Riprap	4.6	5.5
Flow into Point 6	0.5	3.0	0.9	Riprap	4.6	4.9
Flow into Point 7	0.6	3.0	1.1	Riprap	4.6	4.1
Flow into Point 10	0.4	3.0	0.7	Riprap	4.6	6.1
Flow into Point 11	0.4	3.0	0.7	Riprap	4.6	6.1
Flow into Point 13.1	0.2	6.8	0.8	Riprap	4.6	5.4
Flow into Point 0.1	0.3	0.3	0.1	Riprap	4.6	81.9
Flow into Point 0.3	1.2	1.2	0.9	Riprap	4.6	5.1
Flow into Point 1	1.8	0.9	1.0	Riprap	4.6	4.8
Flow into Point 9	0.6	0.6	0.2	Riprap	4.6	20.1
Flow into Point 12	0.6	0.6	0.2	Riprap	4.6	21.2
Flow into Point 13	1.1	1.1	0.8	Riprap	4.6	6.1
Flow into Point 14.1	0.1	0.1	0.0	Riprap	4.6	921.5
Flow into Point 18	1.1	1.0	0.7	Riprap	4.6	6.7
Flow into Point 18.1	1.0	1.0	0.6	Riprap	4.6	7.1
Flow into Point 18.2	1.3	1.3	1.0	Riprap	4.6	4.5
Flow into Point 19	1.1	1.2	0.8	Riprap	4.6	5.6
Flow into Point 4	2.2	0.8	1.4	Riprap	4.6	4.3
Flow into Point 7	2.3	1.0	1.4	Riprap	4.6	4.3
Flow into Point 8	0.7	16.0	7.0	Armorflex	25	3.2
Flow into Point 14	0.8	21.0	10.5	Armorflex	25	2.4

6.4 Energy Dissipation

6.4.1 Stilling Basins

Golder proposes an energy dissipation system to remove energy from flow traveling along each downslope channel. Golder proposes a riprap basin at the end of each downslope channel in line with the methodology in Chapter 10 of HEC-14. Figure 2 shows a profile view of a typical riprap basin as proposed. The proposed design however uses armorflex articulated block the length of the basin (LS) and transitions to riprap for the apron section (LA). Equations 4-10 outline the necessary steps and information required to size a basin as outlined in Figure 2.

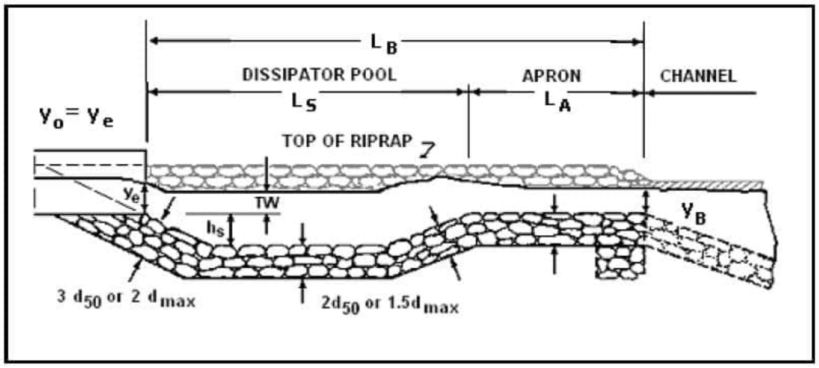


Figure 2: Profile View of Energy Dissipation Basin

$$Equation\ 3: Fr_o = \frac{v_o}{\sqrt{gy_o}}$$

$$Equation\ 4: C_o = 1.4 \qquad \frac{TW}{y_e} < 0.75$$

$$C_o = 4.0\left(\frac{TW}{y_e}\right) - 1.6 \qquad 0.75 < \frac{TW}{y_e} < 1.0$$

$$C_o = 2.4 \qquad 1.0 < \frac{TW}{y_e}$$

$$Equation\ 5: \frac{h_s}{y_e} = 0.86(\frac{D_{50}}{y_e})^{-0.55}\left(\frac{v_o}{\sqrt{gy_e}}\right) - C_o$$

$$Equation\ 6: \frac{Q^2}{g} = \frac{{A_c}^3}{T_c} = \left(y_c(W_B + zy_c)\right)^3/(W_B + 2zy_c)$$

$$Equation\ 7: L_S = 10h_S\ must\ be\ minimum\ of\ 3W_o$$

$$Equation\ 8: L_A = 5h_S\ must\ be\ minimum\ of\ W_o$$

$$Equation\ 9: W_B = 2W_o + 2(L_S + L_A)/3$$

6.4.1.1 North Downslope Channel

Entrance Flow	Q	62.4	cfs			
Initial Flow Depth	y_o	1.1	ft			
Initial Flow Velocity	v_o	14.0	ft/s			
Channel Flow Width	W_o	10.6	ft			
Froude Number	Fr	2.3				
Trial Riprap Size	D_{50}	0.4	ft			
	D_{50}/y_o	0.4		must be greater than 0.1		
Tailwater Height	TW	2.6	ft			
	TW/y_o	2.4				
	C_o	2.4		taken from Equation 4		
Stilling Basin Height	h_s	1.2	ft			
	h_s/D_{50}	3.1		must be greater than 2		
Riprap Lining Thickness	$2*D_{50}$	0.8	ft			
Dissipator Pool Length*	L_s	12.3	ft		32	ft
Apron Length*	L_A	6.1	ft		11	ft
Basin Width	W_B	38.9	ft			
*Use minimum value						
	Q^2/g	120.9				
Trial Exit Flow Depth	y_c	2.6	ft			
Basin Side Slope	z	3.0				
Confirm	Q^2/g	120.9				
Exit Area	A_c	121.3	ft^2			
Exit Velocity	V_c	0.5	ft/s			

6.4.1.2 South Downslope Channel

Entrance Flow	Q	81.8	cfs			
Initial Flow Depth	y_o	0.8	ft			
Initial Flow Velocity	v_o	14.7	ft/s			
Channel Top Flow Width	W_o	8.7	ft			
Froude Number	Fr	2.9				
Trial Riprap Size	D_{50}	0.6	ft			
	D_{50}/y_o	0.8		must be greater than 0.1		
Tailwater Height	TW	0.1	ft			
	TW/y_o	0.1				
	C_o	1.4		taken from Equation 4		
Stilling Basin Height	h_s	1.2	ft			
	h_s/D_{50}	2.0		must be greater than 2		
Riprap Lining Thickness	$2*D_{50}$	1.2	ft			
Dissipator Pool Length*	L_s	11.9	ft		26	ft
Apron Length*	L_A	6.0	ft		9	ft
Basin Width	W_B	32.0	ft			
*Use minimum value						
	Q^2/g	283.2				
Trial Exit Flow Depth	y_c	3.1	ft			
Basin Side Slope	z	3.0				
Confirm	Q^2/g	283.2				
Exit Area	A_c	128.2	ft^2			
Exit Velocity	V_c	0.7	ft/s			

6.4.1.3 Stilling Basin Summary and HEC-RAS Modelling

Table 13 gives a summary of all relevant basin dimensions as determined in sections 6.4.1.1 and 6.4.1.2. Using the values from Table 13, Golder has developed a HEC-RAS model for both the north and south downslope channels. Cross sections were developed using the proposed channel geometry. A maximum stilling basin height of 6 feet is proposed to give adequate freeboard within the basin. Table 14 gives the HEC-RAS results.

Table 13: Summary of Stilling Basin Dimensions

	North Channel	South Channel	
Riprap Size (D_{50})	0.4	0.6	ft
Initial Basin Width	10.6	8.7	ft
Final Basin Width	38.9	32.0	ft
Stilling Basin Depth	1.2	1.2	ft
Stilling Basin Length	31.8	26.2	ft
Apron Length	10.6	8.7	ft
Total Length	42.4	35.0	ft

Table 14: Summary of Jump Height in Each Stilling Basin

	Hydraulic Jump Height (ft)	Basin Height (ft)	Freeboard in Basin (ft)
South Channel	3.1	6.0	2.9
North Channel	2.7	6.0	3.3

Figure 3 shows a depiction of the southern downslope channel before and after the hydraulic jump as modelled in the HEC-RAS program.

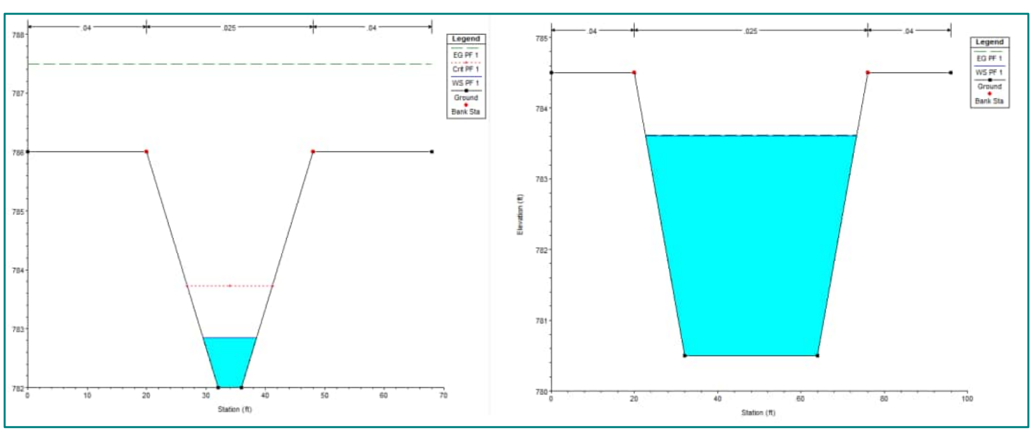


Figure 3: View of Flow in Southern Downslope Channel Before (Left) and After (Right) Hydraulic Jump

6.4.2 Riprap Aprons

Energy dissipation is also required at all pipe culvert outlets. Golder designed riprap aprons at each outlet based on the design guidelines set forth in the Georgia Stormwater Management Manual Volume 2. Figure 4 and Figure 5 show the apron sizing criteria under different tailwater conditions.

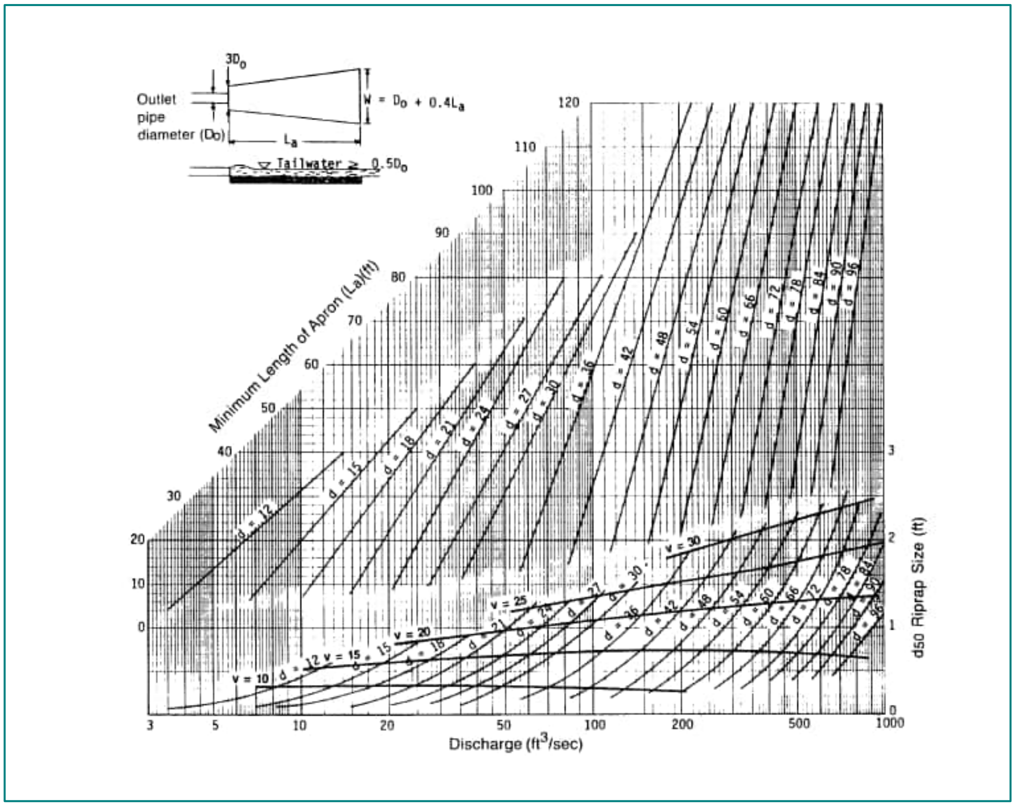


Figure 4: Riprap Apron Dimensions Under Maximum Tailwater Conditions

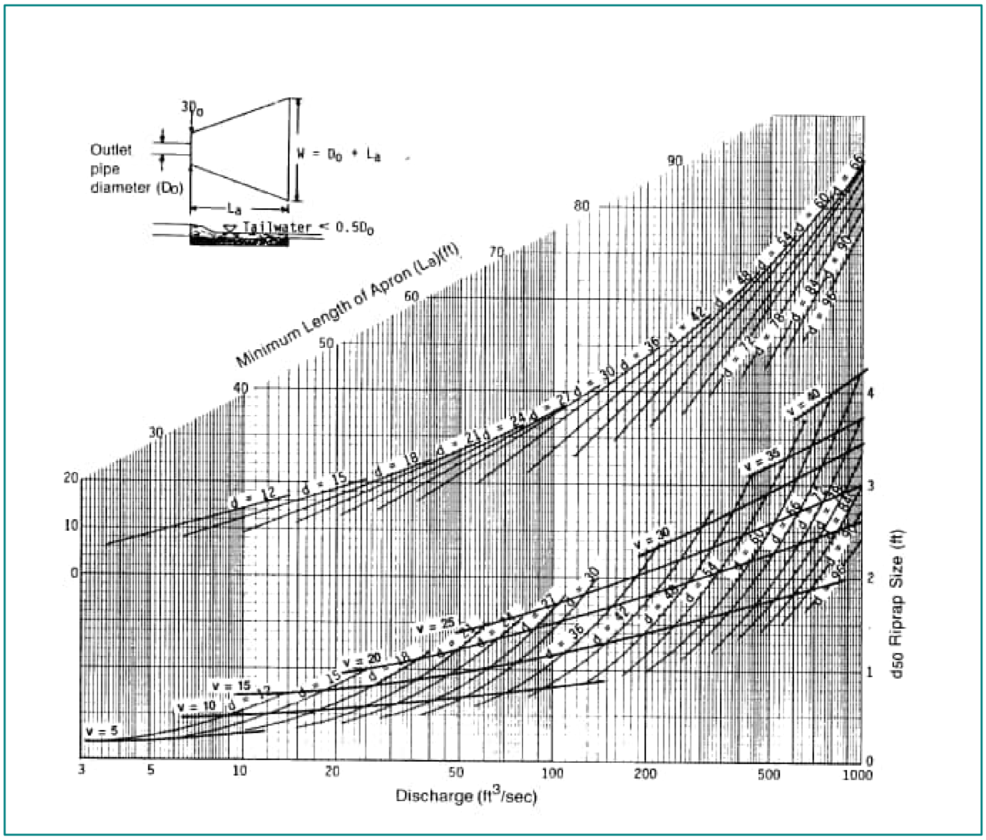


Figure 5: Riprap Apron Dimensions Under Minimum Tailwater Conditions

Table 15 shows the riprap apron dimensions at each applicable culvert outlet, meaning outlets discharging onto clean-closed, non HydroTurf, sections of landfill. Tailwater conditions at each outlet were determined individually based on results from the SSA model. All riprap aprons will use GDOT Type III riprap (D50 = .75 feet) and a riprap thickness of 1.5 feet.

Table 15: Riprap Apron Dimensions

	Pipe Diameter or Flow Depth (in)	Outlet Flow (cfs)	Outlet Velocity (ft/s)	Min Riprap Size D50 (ft)	Apron Initial Width (ft)	Apron Final Width (ft)	Minimum Apron Length (ft)
Culvert Outlet Downstream of Point 9 (Max Tailwater)	36	48	14	0.5	9	33	20
Culvert Outlet Downstream of Point 13 (Max Tailwater)	24	21	9	0.5	6	14	20
Culvert Outlet Downstream of Point 13.1 (Min Tailwater)	7	5	9	0.5	6	10	10
Culvert Outlet Into Pond 1 (Max Tailwater)	36	68	6	0.5	18	38	20
Culvert Outlet Out of Pond 3 (Min Tailwater)	16	11	6	0.5	4.5	8.5	10
Culvert Outlet Downstream of Point 4.1 (Max Tailwater)	16	10	9	0.4	4.5	16.5	12
Culvert Outlet Out of Pond 1 (Min Tailwater)	22	12	7	0.5	6	16	10

6.5 Maximum AgruTurf Drainage Length

Golder determined the maximum permissible flow length on the proposed AgruTurf liner. Based on design guidelines from Agruturf, there exists a maximum flow length before the sand infill within the liner will be displaced. Figure 6 shows the maximum permissible drainage length over Agruturf. Rainfall intensity was taken from the NOAA Atlas 14. To ensure that flow remained below the maximum flow lengths below, bench channels were added at set intervals down the 4:1 side slope of the landfill. These channels are to be lined with Hydroturf, which has no maximum drainage length.

Golder Associates

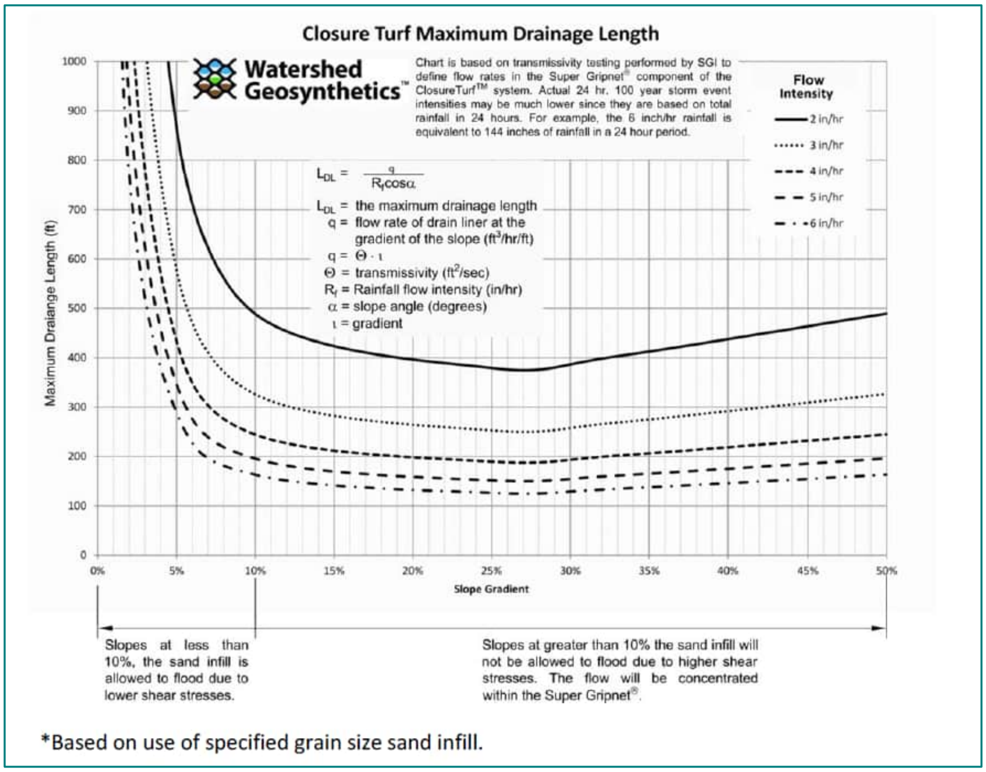


Figure 6: Maximum Drainage Lengths Over AgruTurf

Rainfall Intensity	4	in/hr	For 60 minute duration
For 4% slope (liner top)	N/A	ft maximum flow length	
For 25% slope	190	ft maximum flow length	
Maximum Proposed Sheet Flow Length	160	ft	At interface between basin 1 and basin 2

7.0 GA SAFE DAMS STORAGE REQUIREMENTS

Golder has examined the existing available site storage to determine the site's capacity to store runoff from the GA Safe Dams design storm for a large category dam. The existing conditions are seen as the worst case scenario during construction as ash will be gradually excavated from area around Detention Pond 2 increasing site storage through the construction process. The design storm for this category of dam is the 6 hour half probable maximum precipitation (PMP) storm. This storm depth as taken from the HMR 51 manual is 15.25 inches. To make a conservative estimate it was assumed that all precipitation from the storm event was converted to runoff and in need of storage capacity. The following calculation details the necessary storage requirement for this design storm under the given assumptions:

PMP Depth	30.5	in	from HMR51
1/2 PMP	15.25	in	
Drainage Area	63.6	acre	On-site basins as described in Section 5.0, without area draining to Detention Pond 3
Assuming 100% Runoff, 1/2 PMP Total Runoff Volume	80.8	acre-ft	

An existing site storage curve was generated from topography provided by the Georgia Land Department and Metro Engineering and Surveying Co, INC. from 10-16-2012, and can be seen in Table 16. With a top of dam elevation of 846 ft-msl there is approximately 7 feet of freeboard during the half PMP storm event. See Attachment F for an existing conditions plan view which shows the existing topography.

Table 16: Existing Stage-Storage

Elevation (ft-msl)	Area (sf)	Area (acres)	Volume (acre-ft)
819	3507	0.08	0.00
820	14955	0.34	0.21
822	46359	1.06	1.62
824	77147	1.77	4.45
826	103201	2.37	8.60
828	133084	3.06	14.02
830	196467	4.51	21.58
835	355750	8.17	53.28
838	687041	15.77	89.19
840	716129	16.44	121.40
842	750817	17.24	155.08
844	1049756	24.10	196.41

In order to maintain storage ash excavation must proceed to certain levels as the outer dam is lowered. At the onset of construction the prescribed storm event is the half PMP, as previously described. Once the dam height is lowered to below 35 feet, the dam transitions to a medium category dam size and the design storm transitions to the one-third PMP storm (storm depth of 10.17 inches). Table 17 shows the minimum excavation level (at the proposed grading configuration) in the pond as the outer berm is lowered. To excavate the outer dam below an elevation of 795 ft-msl the Pond 2 outlet structure must be in place and operational. In order to reach the final embankment elevation of 790 ft-msl, the outlet structure must be functioning and all ash must be excavated.

Table 17: Required Pond Excavation Levels During Outer Berm Lowering

DAM ELEVATION (FT-MSL)	DAM HEIGHT TO LOWEST DOWNSTREAM GRADE OF 763 FT-MSL (FT)	GEORGIA SAFE DAMS PROGRAM DESIGN STORM	DESIGN STORM RUN-ON VOLUME (ACRE-FT)	INTERIOR ASH MAXIMUM ELEVATION ADJACENT TO DAM REMOVAL AREAS (FT-MSL)
846.0	83.0	50% PMP	80.8	N/A
840.0	77.0	50% PMP	80.8	835
835.0	72.0	50% PMP	80.8	830.0
830.0	67.0	50% PMP	80.8	824.0
825.0	62.0	50% PMP	80.8	819.0
820.0	57.0	50% PMP	80.8	813.0
815.0	52.0	50% PMP	80.8	807.0
810.0	47.0	50% PMP	80.8	801.0
805.0	42.0	50% PMP	80.8	794.0
800.0	37.0	50% PMP	80.8	784.0
795.0	32.0	33% PMP	53.9	780.0
790.0	27.0	33% PMP	53.9	ASH REMOVED

Golder has also routed the one-third PMP storm event through the proposed final closure plan SSA model. A SITES storm distribution (as taken from the National Resources Conservation Service SITES hydrology program) is used to create a storm hyetograph as shown in Figure 7. Because all proposed stormwater infrastructure is sized for the 100 year, 24 hour storm the SSA model was updated to ensure that all stormwater at each study point is conveyed to the downstream node (not flooded out of the system). Table 18 summarizes the resulting storage during the one-third PMP storm event in Pond 2.

Table 18: One-Third PMP Required Storage in Proposed System

	Peak Elevation (ft-msl)	Top of Dam Elevation (ft-msl)	Freeboard (ft)	Peak Storage Volume (ft3)	Peak Storage Volume (acre-ft)
Pond 2	787.8	790	2.2	1,170,369	26.9

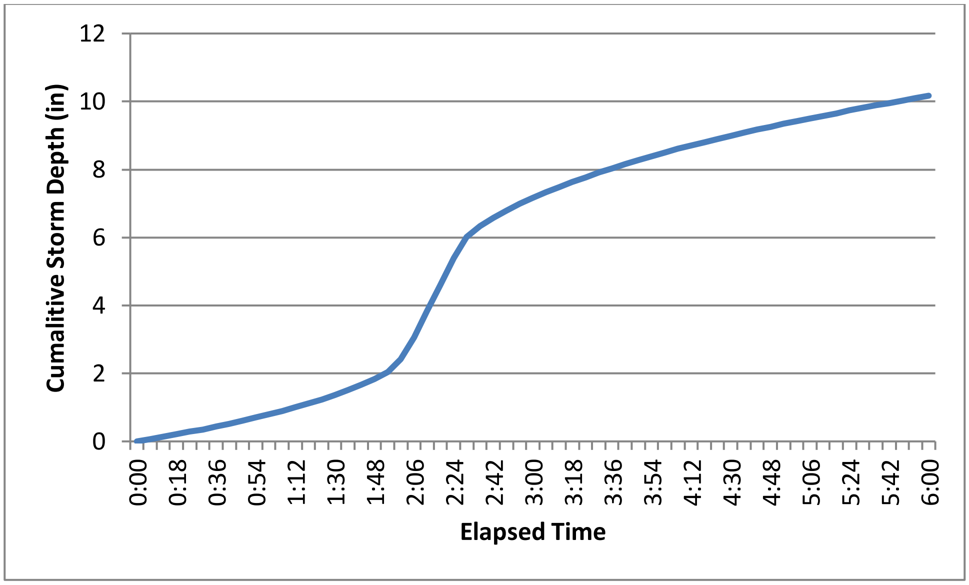


Figure 7: SITES One-Third PMP Storm Distribution

8.0 REFERENCES

Georgia Stormwater Management Manual Volume 2

NOAA Atlas 14 Online Database

Agru Closure Turf Design Guidelines

TR55 Urban Hydrology for Small Watersheds

HEC-15 Design of Roadside Channels with Flexible Linings

HEC-14 Energy Dissipators

HMR 51 Probable Maximum Precipitation Estimates, United States East of the 105th Meridian

APPENDIX A

5.0 Hydrology

5.1 ClosureTurf® Hydrology Parameters

Currently, many regulatory agencies are requiring run-off curve numbers (RCN) of 95-98 of a typical landfill closure. ClosureTurf's RCN should be calculated between 92 and 95. This number was derived by TRI Environmental, Inc. and Colorado State University Hydraulics Laboratory in separate tests. Table 2 below shows the typical TR-55 design parameters for Hydrology using ClosureTurf®.

Closure Turf® Hydrology		
	TR-55 Data	
	Curve Number Depends on Rain Intensity	92[1] - 95
Sheet Flow	Manning's n	
	Slopes >10%	0.12
	Slopes <10%	0.22
	Flow Length	100'-300' dependent on Manning's n until a depth of not more than 0.1 foot is attained in the 2yr 24hr rainfall
	2yr-24hr Rain	SCS
	Land Slope	design
Shallow Concentrated Flow	Flow Length	design
	Slope	design
	Surface (paved/unpaved)	Paved
	X-Sect Area	ft^2
Channel Flow	Wetted Perimeter	Linear Feet
	Channel Slope	ft/ft
	Manning's n	0.03[2]
	Flow Length	design

1. RCN ranging from 92 in High Intensity Rainfalls to 95 in normal rainfall events.
2. Manning's n for channel flow will vary with depth of flow.

Table 2: ClosureTurf® TR-55 Data

APPENDIX B

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{[eq. 3-4]}$$

where:

V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

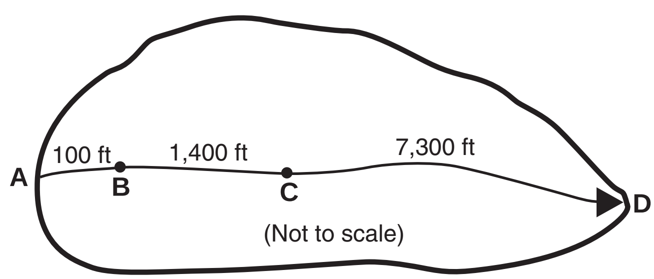
- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- The minimum T_c used in TR-55 is 0.1 hour.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft. Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1,400 ft. Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft^2; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7,300 ft.

See figure 3-2 for the computations made on worksheet 3.



APPENDIX C

Detention Pond 1

Low Flow Orifice		
Calculated Average Cpv Flow	0.48	cfs
Low Orifice Diameter, D	0.25	ft
Orifice Discharge Coefficient, C_o	0.6	
XS Area of Orifice, A	0.049087	sq. ft
Acceleration due to gravity, g	32.2	ft/s^2

High Flow Weir		
Weir Invert Elev	836	ft msl
Weir Length	2	ft
Weir Height	2	ft
Weir XS Area	4	sq ft
road-Crested Weir Coefficient, C_w	3.1	

Pipe/Barrel		
Barrel Diameter	1.750583	ft
Manning's n	0.012	SDR17 HDPE
XS Area	2.406886	sq ft
Outlet Inv	825.9	ft msl
Coefficient of minor losses, km	1	
pipe friction loss coef, kp	0.012638	
Pipe Length	214.54	ft

Emergency Spillway		
Emergency Invert	838	ft msl
Weir Length	12	ft
XS Area of Riser	16	ft^2

Riser Dimensions		
Length	4	ft
Width	4	ft

Elevation	Riser										Barrel								Total Outflow
	Low Flow		Q_{P25}/ High Flow					Q_f/ Emerg. Spilllway		Riser	Inlet					Outlet		Barrel	
			Weir		Orifice						Geometry			Orifice		Pipe			
	H	Q_{low}	H	Q	H	Q	Q_{high}	H	Q	Q_{riser}	H	θ	Area	H	Q	H	Q	Q_{barrel}	Q_{total}
ft msl	ft	cfs	ft	cfs	ft	cfs		ft	cfs		ft	radians	sq ft	ft	cfs	ft	cfs	cfs	cfs
828	-0.125	0	0	0	0	0	0	0	0	0	0	0	0	0	--	1.224708	9.85	9.85	0.00
829	0.875	0.221089	0	0	0	0	0	0	0	0.221089	1	3.427518	1.42	0.124708	2.416242	2.224708	13.27	2.42	0.22
830	1.875	0.323642	0	0	0	0	0	0	0	0.323642	2	6.283185	2.41	1.124708	12.2905	3.224708	15.98	12.29	0.32
831	2.875	0.400759	0	0	0	0	0	0	0	0.400759	3	6.283185	2.41	2.124708	16.89271	4.224708	18.29	16.89	0.40
832	3.875	0.465265	0	0	0	0	0	0	0	0.465265	4	6.283185	2.41	3.124708	20.48587	5.224708	20.34	20.34	0.47
833	4.875	0.521857	0	0	0	0	0	0	0	0.521857	5	6.283185	2.41	4.124708	23.53674	6.224708	22.20	22.20	0.52
834	5.875	0.572886	0	0	0	0	0	0	0	0.572886	6	6.283185	2.41	5.124708	26.23519	7.224708	23.92	23.92	0.57
835	6.875	0.619727	0	0	0	0	0	0	0	0.619727	7	6.283185	2.41	6.124708	28.68087	8.224708	25.52	25.52	0.62
836	7.875	0.663268	0	0	0	0	0	0	0	0.663268	8	6.283185	2.41	7.124708	30.93379	9.224708	27.03	27.03	0.66
837	8.875	0.704123	1	5.704	0	0	5.704	0	0	6.408123	9	6.283185	2.41	8.124708	33.03342	10.22471	28.45	28.45	6.41
838	9.875	0.742733	2	16.13335	0	0	16.13335	0	0	16.87608	10	6.283185	2.41	9.124708	35.00734	11.22471	29.81	29.81	16.88
839	10.875	0.779433	2	16.13335	0	0	16.13335	1	34.224	51.13678	11	6.283185	2.41	10.12471	36.87575	12.22471	31.11	31.11	31.11
840	11.875	0.814481	2	16.13335	0	0	16.13335	2	96.80009	113.7479	12	6.283185	2.41	11.12471	38.65396	13.22471	32.36	32.36	32.36
841	12.875	0.848082	2	16.13335	0	0	16.13335	3	177.8331	194.8146	13	6.283185	2.41	12.12471	40.35388	14.22471	33.56	33.56	33.56
842	13.875	0.880401	2	16.13335	0	0	16.13335	4	273.792	290.8057	14	6.283185	2.41	13.12471	41.98503	15.22471	34.72	34.72	34.72
843	14.875	0.911575	2	16.13335	0	0	16.13335	5	382.636	399.6809	15	6.283185	2.41	14.12471	43.55514	16.22471	35.84	35.84	35.84

Detention Pond 3

Low Flow Orifice

Calculated Average Cpv Flow	0.48	cfs
Low Orifice Diameter, D	0.25	ft
Orifice Discharge Coefficient, C_o	0.6	
XS Area of Orifice, A	0.049087	sq. ft
Acceleration due to gravity, g	32.2	ft/s^2

High Flow Weir

Weir Invert Elev	841	ft msl
Weir Length	2	ft
Weir Height	2	ft
Weir XS Area	4	sq ft
road-Crested Weir Coefficient, C_w	3.1	

Pipe/Barrel

Barrel Diameter	1.312917	ft
Manning's n	0.012	SDR17 HDPE
XS Area	1.35383	sq ft
Outlet Inv	829	ft msl
Coefficient of minor losses, km	1	
pipe friction loss coef, kp	0.018547	
Pipe Length	97.2	ft

Emergency Spillway

Emergency Invert	843	ft msl
Weir Length	12	ft
XS Area of Riser	16	ft^2

Riser Dimensions

Length	4	ft
Width	4	ft

	Riser										Barrel								
											Inlet					Outlet		Barrel	Total Outflow
Elevation	Low Flow		High Flow					Emergency Spillway		Riser	Geometry			Orifice		Pipe			
			Weir		Orifice			Weir											
	H	Q_{low}	H	Q	H	Q	Q_{high}	H	Q	Q_{riser}	H	θ	Area	H	Q	H	Q	Q_{barrel}	Q_{total}
ft msl	ft	cfs	ft	cfs	ft	cfs		ft	cfs		ft	radians	sq ft	ft	cfs	ft	cfs	cfs	cfs
832.8	-0.125	0	0	0	0	0	0	0	0	0	0	0	0	0	--	3.143542	9.88	9.88	0.00
833	0.075	0.064728	0	0	0	0	0	0	0	0.064728	0.2	1.603821	0.23	0	--	3.343542	10.19	10.19	0.06
834	1.075	0.245058	0	0	0	0	0	0	0	0.245058	1.2	5.092624	2.31	0.543542	8.18784	4.343542	11.61	8.19	0.25
835	2.075	0.340466	0	0	0	0	0	0	0	0.340466	2.2	6.283185	2.41	1.543542	14.39822	5.343542	12.88	12.88	0.34
836	3.075	0.414464	0	0	0	0	0	0	0	0.414464	3.2	6.283185	2.41	2.543542	18.48286	6.343542	14.03	14.03	0.41
837	4.075	0.47712	0	0	0	0	0	0	0	0.47712	4.2	6.283185	2.41	3.543542	21.81566	7.343542	15.10	15.10	0.48
838	5.075	0.532454	0	0	0	0	0	0	0	0.532454	5.2	6.283185	2.41	4.543542	24.70284	8.343542	16.09	16.09	0.53
839	6.075	0.582555	0	0	0	0	0	0	0	0.582555	6.2	6.283185	2.41	5.543542	27.28622	9.343542	17.03	17.03	0.58
840	7.075	0.628677	0	0	0	0	0	0	0	0.628677	7.2	6.283185	2.41	6.543542	29.64532	10.34354	17.92	17.92	0.63
841	8.075	0.671638	0	0	0	0	0	0	0	0.671638	8.2	6.283185	2.41	7.543542	31.83005	11.34354	18.76	18.76	0.67
842	9.075	0.712012	1	5.704	0	0	5.704	0	0	6.416012	9.2	6.283185	2.41	8.543542	33.87417	12.34354	19.57	19.57	6.42
843	10.075	0.750216	2	16.13335	0	0	16.13335	0	0	16.88356	10.2	6.283185	2.41	9.543542	35.80176	13.34354	20.35	20.35	16.88
844	11.075	0.786567	2	16.13335	0	0	16.13335	1	34.224	51.14392	11.2	6.283185	2.41	10.54354	37.63075	14.34354	21.10	21.10	21.10
845	12.075	0.821311	2	16.13335	0	0	16.13335	2	96.80009	113.7547	12.2	6.283185	2.41	11.54354	39.37487	15.34354	21.82	21.82	21.82
846	13.075	0.854643	2	16.13335	0	0	16.13335	3	177.8331	194.8211	13.2	6.283185	2.41	12.54354	41.04495	16.34354	22.52	22.52	22.52

APPENDIX D

Time of Concentration Calculations

Node 0.1 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	33 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.03 ft/ft	
Travel Time	T_{t1}	0.07 hr	

Shallow Concentrated Flow

Segment Length	l	610 ft	
Slope of Land Surface	S	0.01 ft/ft	
Short Grass Landuse			
*Flow Velocity	V	1.6 ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.11 hr	
TOTAL TIME		0.18 hr	
		10.67 mins	

Node 0.2 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	200 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.03 ft/ft	
Travel Time	T_{t1}	0.30 hr	

Shallow Concentrated Flow

Segment Length	l	740 ft	
Slope of Land Surface	S	0.01 ft/ft	
Short Grass Landuse			
*Flow Velocity	V	1.6 ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.13 hr	

Channel Flow 1

Up Invert		894.00	ft-msl
Down Invert		866.00	ft-msl
Length	l	915.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.45	ft/s
Travel Time	T_{t3}	0.05	hr
TOTAL TIME		0.48	hr
		28.76	mins

Node 0.3 Basin

Node 0.3 basin assumed to have minimum TOC of 6 mins

Node 1 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	140	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.10	hr	

Channel Flow 1

Up Invert		846.00	ft-msl
Down Invert		838.00	ft-msl
Length	l	140.00	ft
Slope	s	0.06	ft/ft
Bottom Width	a	2.70	ft
Side Slope		3.00	
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	7.32	ft
Channel Area	A	5.70	ft^2
Hydralic Radius	r	0.78	ft
Mannings Coefficient	n	0.03	
Velocity	V	10.04	ft/s
Travel Time	T_{t3}	0.00	hr
TOTAL TIME		0.10	hr
		6.11	mins

Node 2 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	50 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.25 ft/ft	
Travel Time	T_{t1}	0.04 hr	

Shallow Concentrated Flow

Segment Length	l	1367 ft	
Slope of Land Surface	S	0.04 ft/ft	
Short Grass Landuse			
*Flow Velocity	V	3.5 ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.11 hr	

TOTAL TIME		0.15 hr
		9.09 mins

Node 3 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	120 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.25 ft/ft	
Travel Time	T_{t1}	0.09 hr	

Shallow Concentrated Flow

Segment Length	l	1010 ft	
Slope of Land Surface	S	0.04 ft/ft	
Short Grass Landuse			
*Flow Velocity	V	3.5 ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.08 hr	

TOTAL TIME		0.17 hr
		10.00 mins

Node 4 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	85 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.03 ft/ft	
Travel Time	T_{t1}	0.15 hr	

Shallow Concentrated Flow

Segment Length	l	0	ft
Slope of Land Surface	S	0.25	ft/ft
Short Grass Landuse			
*Flow Velocity	V	8.5	ft/s
Travel Time	T_{t2}	0.00	hr

*Flow velocity taken from NEH Part 630, Figure 15-4

Channel Flow 2

Up Invert		841.00	ft-msl
Down Invert		834.00	ft-msl
Length	l	1115.00	ft
Slope	s	0.01	ft/ft
Bottom Width	a	5.60	ft
Side Slope		3.00	
Channel Height	h	6.00	ft
Wetted Perimeter	P_w	43.95	ft
Channel Area	A	141.60	ft^2
Hydralic Radius	r	3.22	ft
Mannings Coefficient	n	0.04	
Velocity	V	7.36	ft/s
Travel Time	T_{t3}	0.04	hr
TOTAL TIME		0.20	hr
		11.73	mins

Node 5 Basin

Node 5 basin assumed to have minimum TOC of 6 mins

Node 6 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	95	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.03	ft/ft	
Travel Time	T_{t1}	0.17	hr	

Shallow Concentrated Flow

Segment Length	l	0	ft	
Slope of Land Surface	S	0.25	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	8.5	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.00	hr	

Channel Flow 1

Up Invert		860.00	ft-msl
Down Invert		832.00	ft-msl
Length	l	912.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.46	ft/s
Travel Time	T_{t3}	0.05	hr
TOTAL TIME		0.21	hr
		12.85	mins

Node 7 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	100	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.07	hr	

Shallow Concentrated Flow

Segment Length	l	0	ft	
Slope of Land Surface	S	0.25	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	8.5	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.00	hr	

Channel Flow 1

Up Invert		832.00	ft-msl
Down Invert		822.00	ft-msl
Length	l	316.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.54	ft/s
Travel Time	T_{t3}	0.02	hr
TOTAL TIME		0.09	hr
		5.44	mins

Node 8 Basin

Sheet Flow

*Mannings coefficient	n	0.22	
Sheet Flow Length	l	200	ft
2 yr, 24 hr Rainfall	P_2	3.73	in
**Slope of Land Surface	S	0.25	ft/ft
Travel Time	T_{t1}	0.13	hr
TOTAL TIME		0.13	hr
		7.82	mins

*Mannings coefficent taken from Attachment A

Node 9 Basin

Sheet Flow

*Mannings coefficient	n	0.22	
Sheet Flow Length	l	102	ft
2 yr, 24 hr Rainfall	P_2	3.73	in
**Slope of Land Surface	S	0.25	ft/ft
Travel Time	T_{t1}	0.08	hr

*Mannings coefficent taken from Attachment A

Channel Flow 2

Up Invert		816.00	ft-msl
Down Invert		782.00	ft-msl
Length	l	360.00	ft
Slope	s	0.09	ft/ft
Bottom Width	a	2.70	ft
Side Slope		3.00	
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	7.32	ft
Channel Area	A	5.70	ft^2
Hydralic Radius	r	0.78	ft
Mannings Coefficient	n	0.03	
Velocity	V	12.91	ft/s
Travel Time	T_{t3}	0.01	hr
TOTAL TIME		0.08	hr
		5.03	mins

Node 10 Basin

Sheet Flow

*Mannings coefficient	n	0.22	
Sheet Flow Length	l	100	ft
2 yr, 24 hr Rainfall	P_2	3.73	in
**Slope of Land Surface	S	0.25	ft/ft
Travel Time	T_{t1}	0.07	hr

*Mannings coefficent taken from Attachment A

Shallow Concentrated Flow

Segment Length	l	0	ft	
Slope of Land Surface	S	0.25	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	8.5	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.00	hr	

Channel Flow 1

Up Invert		860.00	ft-msl
Down Invert		832.00	ft-msl
Length	l	950.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.35	ft/s
Travel Time	T_{t3}	0.05	hr
TOTAL TIME		0.12	hr
		7.45	mins

Node 11 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	125	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.09	hr	

Channel Flow 1

Up Invert		832.00	ft-msl
Down Invert		808.00	ft-msl
Length	l	800.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.40	ft/s
Travel Time	T_{t3}	0.04	hr
TOTAL TIME		0.13	hr
		7.84	mins

Node 12 Basin

Sheet Flow

*Mannings coefficient	n	0.22	
Sheet Flow Length	l	120	ft
2 yr, 24 hr Rainfall	P_2	3.73	in
**Slope of Land Surface	S	0.25	ft/ft
Travel Time	T_{t1}	0.09	hr

*Mannings coefficent taken from Attachment A

Channel Flow 1

Up Invert		802.00	ft-msl
Down Invert		784.00	ft-msl
Length	l	570.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.54	ft/s
Travel Time	T_{t3}	0.03	hr
TOTAL TIME		0.12	hr
		6.91	mins

Node 13.0 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	148 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.25 ft/ft	
Travel Time	T_{t1}	0.10 hr	

Channel Flow 2

Up Invert		800.00 ft-msl
Down Invert		780.00 ft-msl
Length	l	300.00 ft
Slope	s	0.07 ft/ft
Bottom Width	a	4.00 ft
Side Slope		3.00
Channel Height	h	2.00 ft
Wetted Perimeter	P_w	14.65 ft
Channel Area	A	20.00 ft^2
Hydralic Radius	r	1.37 ft
Mannings Coefficient	n	0.03
Velocity	V	15.78 ft/s
Travel Time	T_{t3}	0.01 hr
TOTAL TIME		0.11 hr
		6.46 mins

Node 14 Basin

Sheet Flow

*Mannings coefficient	n	0.22	*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	200 ft	
2 yr, 24 hr Rainfall	P_2	3.73 in	
**Slope of Land Surface	S	0.04 ft/ft	
Travel Time	T_{t1}	0.27 hr	

Shallow Concentrated Flow

Segment Length	l	21 ft	
Slope of Land Surface	S	0.04 ft/ft	
Short Grass Landuse			
*Flow Velocity	V	3.5 ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.00 hr	

Channel Flow 2

Up Invert		840.00	ft-msl
Down Invert		830.00	ft-msl
Length	l	923.00	ft
Slope	s	0.01	ft/ft
Bottom Width	a	4.00	ft
Side Slope		3.00	
Channel Height	h	2.00	ft
Wetted Perimeter	P_w	14.65	ft
Channel Area	A	20.00	ft^2
Hydralic Radius	r	1.37	ft
Mannings Coefficient	n	0.03	
Velocity	V	6.36	ft/s
Travel Time	T_{t3}	0.04	hr
TOTAL TIME		0.31	hr
		18.79	mins

Node 15 Basin

Sheet Flow 1

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	90	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.07	hr	

Sheet Flow 2

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	110	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.07	ft/ft	
Travel Time	T_{t1}	0.13	hr	

Shallow Concentrated Flow

Segment Length	l	240	ft	
Slope of Land Surface	S	0.07	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	4.5	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.01	hr	

TOTAL TIME	0.22	hr
	13.08	mins

Node 16 Basin

Sheet Flow 1

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	105	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.08	hr	

Sheet Flow 2

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	72	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.027778	ft/ft	
Travel Time	T_{t1}	0.14	hr	

TOTAL TIME	0.22	hr
	12.98	mins

Node 16 Basin

Sheet Flow 1

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	145	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.10	hr	

Sheet Flow 2

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	50	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.025	ft/ft	
Travel Time	T_{t1}	0.11	hr	

TOTAL TIME		0.21	hr
		12.52	mins

Node 17 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	142	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.10	hr	

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	53	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.04	ft/ft	
Travel Time	T_{t1}	0.09	hr	

TOTAL TIME		0.19	hr
		11.57	mins

Node 18 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	172	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.04	ft/ft	
Travel Time	T_{t1}	0.24	hr	

Shallow Concentrated Flow

Segment Length	l	740	ft	
Slope of Land Surface	S	0.01	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	1.6	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.13	hr	
TOTAL TIME		0.37	hr	
		22.13	mins	

Node 18.2 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	200	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.02	ft/ft	
Travel Time	T_{t1}	0.36	hr	

Shallow Concentrated Flow

Segment Length	l	274	ft	
Slope of Land Surface	S	0.01	ft/ft	
Short Grass Landuse				
*Flow Velocity	V	2.3	ft/s	*Flow velocity taken from NEH Part 630, Figure 15-4
Travel Time	T_{t2}	0.03	hr	

Channel Flow 1

Up Invert		894.00	ft-msl
Down Invert		870.00	ft-msl
Length	l	900.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.09	ft/s
Travel Time	T_{t3}	0.05	hr
TOTAL TIME		0.44	hr
		26.40	mins

Node 19 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	122	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.09	hr	
TOTAL TIME		0.09	hr	
		5.26	mins	

Node 20 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	70	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.06	hr	

Channel Flow 1

Up Invert		884.00	ft-msl
Down Invert		848.00	ft-msl
Length	l	1190.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.42	ft/s
Travel Time	T_{t3}	0.06	hr
TOTAL TIME		0.12	hr
		7.03	mins

Node 21 Basin

Sheet Flow

*Mannings coefficient	n	0.22		*Mannings coefficent taken from Attachment A
Sheet Flow Length	l	130	ft	
2 yr, 24 hr Rainfall	P_2	3.73	in	
**Slope of Land Surface	S	0.25	ft/ft	
Travel Time	T_{t1}	0.09	hr	

Channel Flow 1

Up Invert		852.00	ft-msl
Down Invert		840.00	ft-msl
Length	l	410.00	ft
Slope	s	0.03	ft/ft
Bottom Width	a	0.00	ft
Side Slope 1		4.00	:1
Side Slope 2		20.00	:1
Channel Height	h	1.00	ft
Wetted Perimeter	P_w	24.15	ft
Channel Area	A	12.00	ft^2
Hydralic Radius	r	0.50	ft
Mannings Coefficient	n	0.03	
Velocity	V	5.33	ft/s
Travel Time	T_{t3}	0.02	hr
TOTAL TIME		0.11	hr
		6.82	mins

APPENDIX E

Factor of Safety
Hydraulic Analysis

These calculations are an application of the Moment Stability Analysis technique presented in Julien (2010) and as illustrated in the NCMA Manual (2010), listed in the References.

The factor of safety method is used in the selection of block sizes for ACB's for revetments or bed armor.

The following assumes that hydraulic testing has been performed for the block system to quantify a critical shear stress; the use of Manning's equation conservatively assumes normal depth and critical velocity.

References

1. Julien, Pierre Y. (2010) "Erosion and Sedimentation", 2nd Edition, Cambridge University Press

2. National Concrete Masonry Association (2010), "Design Manual for Articulating Concrete Block (ACB) Revetment Systems", NCMA Publication TR220A.

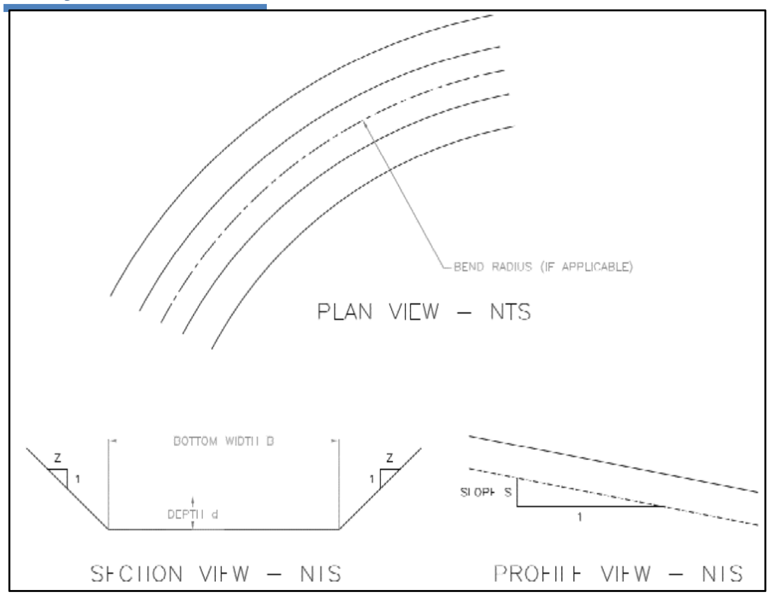
3. USDOT Federal Highway Administration Hydraulic Engineering Circular No. 15, Third Edition (2005) "Design of Roadside Channels with Flexible Linings", National Highway Institute.

4. FHWA Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance - Third Edition, Volume II, Design Guideline 8.

5. ASTM D 7276 & D7277 Testing and Analysis Compliant, See Contech Tapered Testing Report

Factor of Safety
Hydraulic Analysis

Project Data



Channel Bottom Width, B	4	ft
Bed Slope, S_o	0.21	ft/ft
Left Side Slope, Z_L	3	(_H:1V)
Right Side Slope, Z_R	3	(_H:1V)
Bend Radius, r	0	ft
Depth of Flow d	0.79	ft

The Depth of Flow is varied iteratively to obtain a given volumetric flow rate.

Top Surface Width ,T	8.76	ft

Other Constants

Density of Water, γ	62.4	pcf
Density of Concrete, Dry-Cast	130	pcf
Sp. Gr. Of Concrete, S_c	2.083	--
Gravitational Constant, g	32.2	ft/s^2

Calculated Channel Geometry Factors

Flow Area, A	5.07	ft^2
Wetted Perimeter, P	9.02	ft
Hydraulic Radius = R_H = A/P =	0.56	ft
Bend Coefficient, K_b	1	--
Froude Number, Fr	3.67	--
Flow Type	Supercritical	
Largest Side Slope Angle, θ_1	18.435	°
Bed Slope Angle, θ_0	11.860	°

Volumetric Flow Rate, Q	94.00	cfs

The Volumetric Flow Rate is determined using Manning's equation:

$Q = 1.486 / (n * A * R^{2/3} * S^{1/2})$

Velocity, V	18.55	ft/sec
Friction Slope, S	0.210	ft/ft

The Friction slope is assumed to be equal to the bed slope, which further assumes uniform flow.

ArmorFlex Block parameters

Class	**40-T**
SF	**1.97**

ϑ_1	0.198	ft
ϑ_2	0.971	ft
ϑ_3	0.317	ft
ϑ_4	0.971	ft

Weight	58.1	lbs
Width	1.292	ft
τ_c	25.0	psf
ΔZ	0.0	in
n	0.025	--

ARMORTEC
Erosion Control Solutions

Factor of Safety
Hydraulic Analysis

Detailed Calculations REFERENCE

Flow Area, $A = A_L + A_B + A_R$

$A_L = ½ * d^2 * Z_L =$	0.95	ft^2	
$A_B = B * d =$	3.18	ft^2	
$A_R = ½ * d^2 * Z_R =$	0.95	ft^2	
$A =$	5.07	ft^2	

Wetted Perimeter, $P = P_L + P_B + P_R$

$P_L = d * (Z_L^2 + 1)^{0.5} =$	2.51	ft	
$P_B = B =$	4	ft	
$P_R = d * (Z_R^2 + 1)^{0.5} =$	2.51	ft	
$P =$	9.02	ft	

Volumetric Flow Rate, Q

$Q = 1.486 / n * A * R_H^{2/3} * S^{1/2} =$	94.00	cfs	(Ref. 3 Eqn. 2.1)

Compute Factor of Safety Parameters

Submerged Weight, W_s	$W_s = W * ((S_c - 1) / S_c) =$	30.2	lb	(Ref. 2 Eqn 4.13a)
Applied Shear Stress, τ_o	$\tau_o = \gamma * d * S_o =$	10.41	psf	(Ref. 3 Eqn. 2.4)

Bend Coefficient Calculation

X = r/B = (Constrained to between 1.984 and 10)	1.984	--	
Calculated $K_b = 2.38-0.206(X)+0.0073(X)^2 =$	2.00	--	(Ref. 3 Eqn. 3.7)
Constrained K_b: $1.05 \leq K_b \leq 2 \rightarrow$	1.00		

(If no bend radius is present, $K_b = 1$)

Step 1: Compute Factor of Safety Parameters

(Design Shear Stress)	$\tau_o = K_b \gamma y \sin(\tan^{-1} S_o) =$	10.18	lbs/ft^2	(Ref. 3 Eqn 3.1 & 3.6)
(Stability Number for Horizontal Surface)	$\eta_o = \tau_0 / \tau_c =$	0.41	--	(Ref. 2 Eqn 4.12a)
	$a_\theta = (\cos^2\theta_1 - \sin^2\theta_0)^{1/2} =$	0.93	°	(Ref. 2 Eqn 4.10a)
	$\theta = \arctan((\sin\theta_0 * \cos\theta_1) / (\sin\theta_1 * \cos\theta_0) =$	32.2	°	(Ref. 2 Eqn 4.9a)
	$\beta = \arctan((\cos(\theta_0 + \theta) / ((\vartheta_4 / \vartheta_3 + 1) * (1 - a_\theta^2)^{1/2} / (\eta_o * \vartheta_2/\vartheta_1) + \sin(\theta_0 + \theta)) =$	26.15	°	(Ref. 2 Eqn 4.8a)
(Stability Number for Slope Surface)				
	$\eta_1 = ((\vartheta_4 / \vartheta_3 + \sin(\theta_0 + \theta + \beta)) / (\vartheta_4 / \vartheta_3 + 1)) * \eta_o =$	0.40	--	(Ref. 2 Eqn 4.7a)
	$\delta = 90^o - \beta - \theta =$	31.64	°	(Ref. 2 Eqn 4.6a)

Step 2: Consider Effects for Specified Projection (Assumes lift and drag forces are equal)

$F_L' = F_D' = 0.5 \Delta Z b \rho V_{des}^2 =$	0.00	lbs	(Ref. 2 Eqn. 2.2)

Step 3: Compute Factor of Safety

$SF = (\vartheta_2/\vartheta_1 * a_\theta) / ((1 - a_\theta^2)^{1/2} * \cos\beta + \eta_1 * (\vartheta_2/\vartheta_1) + (\vartheta_3 * F_D' * \cos\delta + \vartheta_4 * F_L') / (\vartheta_1 * W_s)) =$

1.97	--	(Ref. 2 Eqn 4.5a)

Modified from Julien (2010)

Factor of Safety
Hydraulic Analysis

Detailed Calculations

If H = horizontal component of side slope, then $\theta_1 = \tan^{-1}(1/H)$
If S = bed slope, then $\theta_0 = \tan^{-1}(S)$

For τ_o:

$\tan^{-1} S_o =$	11.86	$\sin(\tan^{-1} S_o) =$	0.206

For a_θ:

$\cos\theta_1 =$	0.949	$\cos^2\theta_1 =$	0.900
$\sin\theta_0 =$	0.206	$\sin^2\theta_0 =$	0.042

For θ:

$\sin\theta_0 * \cos\theta_1 =$	0.195	$(\sin\theta_0 * \cos\theta_1) / (\sin\theta_1 * \cos\theta_0) =$	0.630
$\sin\theta_1 =$	0.316		
$\cos\theta_0 =$	0.979		
$\sin\theta_1 * \cos\theta_0 =$	0.309		

For β:

$\cos(\theta_0 + \theta) =$	0.718	$(\vartheta_4 / \vartheta_3 + 1) * (1 - a_\theta^2)^{1/2} / (\eta_o * \vartheta_2 / \vartheta_1) =$	0.7677
$\vartheta_4 / \vartheta_3 + 1 =$	4.063	$(\vartheta_4 / \vartheta_3 + 1) * (1 - a_\theta^2)^{1/2} / (\eta_o * \vartheta_2 / \vartheta_1) + \sin(\theta_0 + \theta) =$	1.463
$(1 - a_\theta^2)^{1/2} =$	0.377	$\cos(\theta_0 + \theta) / ((\vartheta_4 / \vartheta_3 + 1) * (1 - a_\theta^2)^{1/2} / (\eta_o * \vartheta_2 / \vartheta_1) + \sin(\theta_0 + \theta)) =$	0.491
$\eta_o * \vartheta_2 / \vartheta_1 =$	1.996		
$\sin(\theta_0 + \theta) =$	0.696		

For η_1:

$\vartheta_4 / \vartheta_3 =$	3.063	$\vartheta_4 / \vartheta_3 + \sin(\theta_0 + \theta + \beta) =$	4.004
$\sin(\theta_0 + \theta + \beta) =$	0.941	$(\vartheta_4 / \vartheta_3 + \sin(\theta_0 + \theta + \beta)) / (\vartheta_4 / \vartheta_3 + 1) =$	0.9855
$\vartheta_4 / \vartheta_3 + 1 =$	4.063		
$\eta_o =$	0.407		

For SF:

$\vartheta_2/\vartheta_1 * a_\theta =$	4.542	$(\vartheta_3 * F_D' * \cos\delta + \vartheta_4 * F_L') / (\vartheta_1 * W_s) =$	0.000
$(1 - a_\theta^2)^{1/2} * \cos\beta =$	0.339	$(1 - a_\theta^2)^{1/2} * \cos\beta + \eta_1 * (\vartheta_2/\vartheta_1) + (\vartheta_3 * F_D' * \cos\delta + \vartheta_4 * F_L') / (\vartheta_1 * W_s) =$	2.3056
$\eta_1 * (\vartheta_2/\vartheta_1) =$	1.967		
$\cos\delta =$	0.851		
$\vartheta_3 * F_D' * \cos\delta + \vartheta_4 * F_L' =$	0.000		
$\vartheta_1 * W_s =$	5.984		

Modified from Julien (2010)

Factor of Safety
Hydraulic Analysis

Parameters for Factor of Safety Calculations								
Block Class	Submerged Weight	ϑ_1	ϑ_2	ϑ_3	ϑ_4	τ_c 0 Degrees	Width	Weight
	(lbs)	(ft)	(ft)	(ft)	(ft)	(psf)	(ft)	(lbs)
30-S	17.10	0.198	0.726	0.317	0.726	5.180	0.967	32.89
40	30.69	0.198	0.971	0.317	0.971	11.200	1.292	59.02
40-L	50.53	0.198	1.222	0.317	1.222	19.460	1.967	97.18
40-T	30.22	0.198	0.971	0.317	0.971	25.022	1.292	58.12
45	37.05	0.198	0.971	0.317	0.971	13.530	1.292	71.25
45-L	56.76	0.198	1.222	0.317	1.222	21.860	1.967	109.15
45-S	20.38	0.198	0.726	0.317	0.726	6.170	0.967	39.20
50	39.67	0.250	0.971	0.400	0.971	13.610	1.292	76.29
50-L	60.33	0.250	1.222	0.400	1.222	22.050	1.967	116.02
50-S	21.86	0.250	0.726	0.400	0.726	6.130	0.967	42.03
50-T	39.20	0.250	0.971	0.400	0.971	30.500	1.292	75.39
55	47.51	0.250	0.971	0.400	0.971	16.290	1.292	91.37
55-L	71.91	0.250	1.222	0.400	1.222	26.280	1.967	138.29
55-S	26.13	0.250	0.726	0.400	0.726	7.330	0.967	50.25
60	48.45	0.313	0.971	0.500	0.971	15.490	1.292	93.17
60-T	48.58	0.313	0.971	0.500	0.971	35.200	1.292	93.42
70	59.23	0.375	0.971	0.600	0.971	17.730	1.292	113.90
70-L	90.72	0.375	1.222	0.600	1.222	29.520	1.967	174.46
70-T	56.66	0.375	0.971	0.600	0.971	38.500	1.292	108.96
75	58.25	0.313	0.971	0.500	0.971	18.620	1.292	112.02
85	70.51	0.375	0.971	0.600	0.971	21.100	1.292	135.60
85-L	107.76	0.375	1.222	0.600	1.222	35.060	1.967	207.23

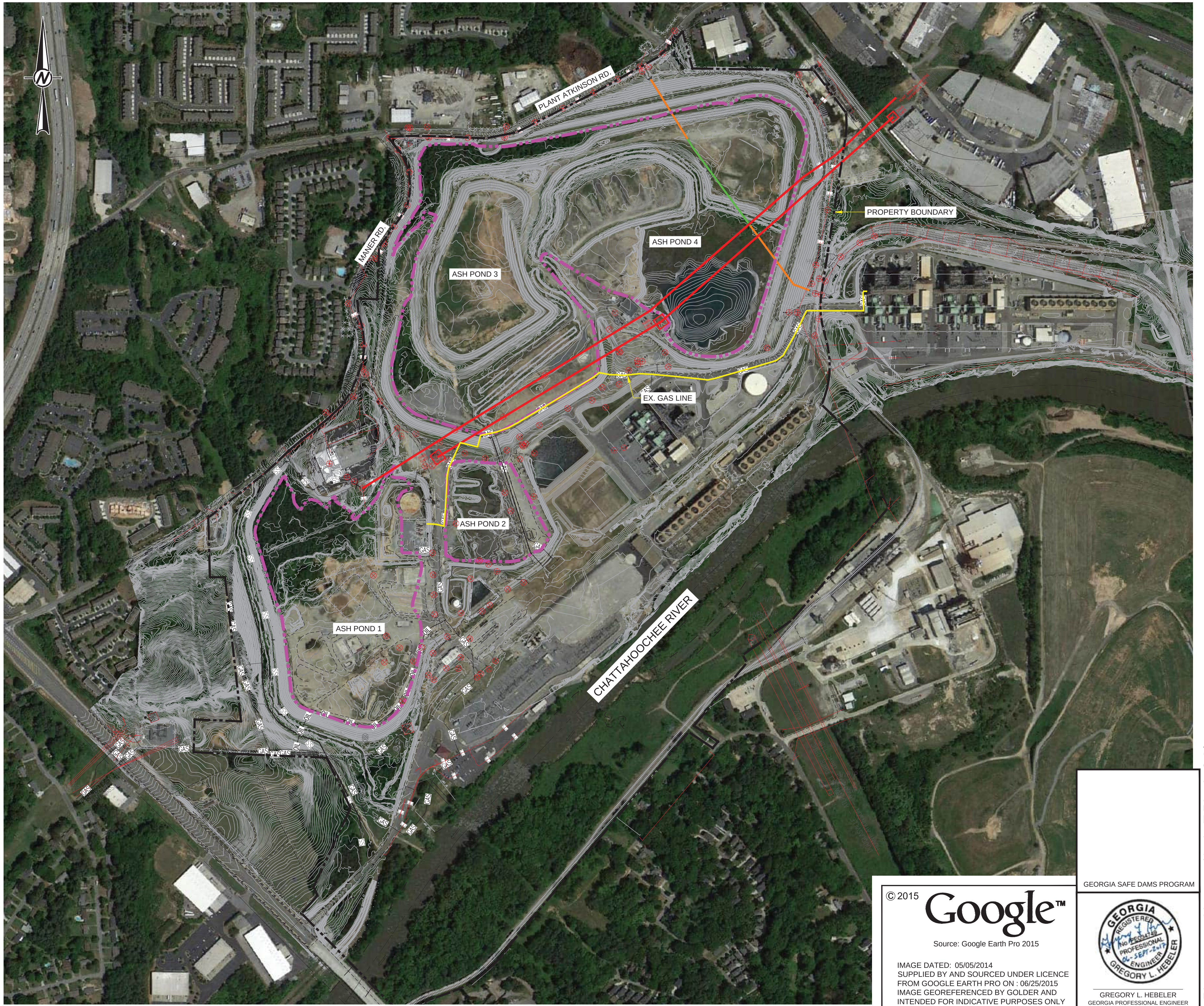
NOTE: Moment Arms and critical shear stresses assume blocks are oriented with the long axis parallel to the flow direction.

NOTE: Submerged weight assumes minimum concrete density of 130 lbs/CF and water density of 62.4 lbs/CF.

	ϑ_1	ϑ_2	ϑ_3	ϑ_4	τ_c	Width	Weight
40-T	0.198	0.971	0.317	0.971	25.022	1.292	58.120

Modified from Julien (2010)

APPENDIX F



LEGEND

	EXISTING CONTOURS
	PROPERTY BOUNDARY MARKERS/LIMITS
	EXISTING UNPAVED PLANT ROAD
	EXISTING STREAM DIVERSION CULVERT UNDER EMBANKMENTS
	EXISTING STREAM DIVERSION CULVERT-CONCRETE ENCASED SECTION
	EXISTING OVERHEAD ELECTRIC LINES IN ASH POND 3 & 4 AREA TO REMAIN & TO BE PROTECTED
	APPROXIMATE EXISTING ASH LIMITS
	EXISTING GAS LINE

REFERENCES

1. THE EXISTING TOPOGRAPHY AND CONTOUR ELEVATIONS WERE PROVIDED BY GEORGIA LAND DEPARTMENT AND METRO ENGINEERING AND SURVEYING CO, INC. THE DATE OF THE SURVEY PROVIDED AND SHOWN ON THIS SET OF PLANS IS 10-16-2012. REFER TO THE SURVEY DRAWING TITLED "TOPOGRAPHIC MAP PREPARED FOR GEORGIA POWER COMPANY PLANT MCDONOUGH - GEORGIA STATE PLANE WEST SURVEY FEET - DATE OF PHOTOGRAPHY 10-26-12. PROJECT NO. 13225 - 01-13-2013."

2. THE REVISED TOPOGRAPHY & CONTOUR ELEVATIONS WERE PROVIDED BY GEORGIA POWER LAND DEPARTMENT. THE DATA SHOWN IS AN UPDATE TO THE PLANS DONE ON 10-16-2012 & THE ONSITE CHANGES SINCE THAT 2012 SURVEY. THE REVISED SURVEY WAS DONE ON 1-12-2016 & MERGED WITH THE DATA ON 10-16-2012.
GEORGIA POWER COMPANY PLANT MCDONOUGH ASH PONDS - GEORGIA STATE PLANE WEST SURVEY FEET - DATE OF SURVEY 1-12-2016 - LAND ENG. PROJECT # 20160020.

3. IMAGE TAKEN FROM GOOGLE EARTH PRO ON JUNE 25, 2015. IMAGE DATED MAY 05. 2014.

SEPTEMBER 2017 - ISSUED FOR APPROVAL & CONSTRUCTION

0 300 600
1" = 300' FEET

GEORGIA SAFE DAMS PROGRAM

GREGORY L. HEBELER
GEORGIA PROFESSIONAL ENGINEER
CERTIFICATE NUMBER 034749

CLIENT
GEORGIA POWER COMPANY /
SOUTHERN COMPANY SERVICES

Southern Company

PROJECT
PLANT MCDONOUGH
ASH POND NO. 3 & 4 CLOSURE PLAN

TITLE
EXISTING CONDITIONS MAY 2014 SITE AERIAL

CONSULTANT
Golder Associates

YYYY-MM-DD	2016/01/21
DESIGNED	GLH
PREPARED	RMS
REVIEWED	GLH
APPROVED	WRS

PROJECT NO.	PROJECT ID	REV.	SHEET
1539180	MCD15017	2	2

Path: \\atlanta\Cadd\Southern Company\1525193 2015 Plant McDonough\PRODUCTION\DESIGN\ | File Name: 1525193 02 EX-COND - AERIAL.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D
0 1 in

APPENDIX K

Analysis of Permanent Detention Pond 3 Outlet Structure

TECHNICAL MEMORANDUM

DATE August 2020 **Project No.** 1777449

TO Mr. Morgan French, SCS; Ms. Virginia Pantano, GPC; Ms. Alex Wild, GPC

FROM Greg Hebeler, PE; James Grimes, PE; Lizmarie Steel, PE **EMAIL** lizmarie_steel@golder.com

ANALYSIS OF PERMANENT POND 3 OUTLET STRUCTURE ATTENUATION / RETENTION TIMES

Golder has conducted an analysis to determine the time required to drain the permanent Pond 3 after a variety of 24-hour storm events for a variety of outlet configurations in order to determine the optimal outlet configuration to drain the pond completely in less than a day. The pond outlet design includes a multi-stage outlet including (1) a low-level orifice (the size of the low level orifice is determined in this analysis), (2) an elevated overflow weir in one side of the outlet structure, and (3) an upper overflow weir allowing inflow through the top grating of the Pond 3 outlet structure. The low-level outlet in the Pond 3 outlet riser drains the majority of the pond storage volume over an extended period of time after a storm event. This analysis examines a variety of low-level orifice conditions and examines the effect of each on the time required to drain Pond 3.

Golder estimated the time required to fully drain Pond 3 using the AP-3/4 final closure stormwater model utilizing the Autodesk Storm and Sanitary Analysis (SSA) Program. A detailed explanation of the site hydrology/hydraulics and stormwater model is provided in the Engineering Report for Plant McDonough CCR Unit AP-2 and AP-3/4 (Part B Section 2 of Permit Application) and Appendix J of this report. The SSA model provides estimated outlet times based on a Pond 3 outlet rating curve developed by Golder, combining the total flow through the low flow orifice/overflow weir and through the 18” SDR 17 HDPE riser outlet pipe. The rating curve for each analyzed condition is provided in Table 1. The provided storage in Pond 3 is based on the Golder final closure design.

Table 1: Pond 3 Outlet Rating Curves

	Outflow (cfs)				
Elevation (ft-msl)	1 - 3” Orifice Outlet	1 - 6” Orifice Outlet	6 x 3” Orifice Outlets	Full Outlet Pipe Orifice Outlet*	Notes
832.8	0.00	0.00	0.00	0.00	Orifice Invert
833	0.06	0.46	0.39	2.40	
834	0.25	0.92	1.47	4.81	
835	0.34	1.32	2.04	8.10	
836	0.41	1.62	2.49	10.40	

	Outflow (cfs)				
Elevation (ft-msl)	1 - 3" Orifice Outlet	1 - 6" Orifice Outlet	6 x 3" Orifice Outlets	Full Outlet Pipe Orifice Outlet*	Notes
837	0.48	1.88	2.86	12.27	
838	0.53	2.10	3.19	13.89	
839	0.58	2.31	3.50	15.35	
840	0.63	2.49	3.77	16.67	
841	0.67	2.67	4.03	17.90	Overflow Weir invert
842	6.42	8.53	9.98	19.57	
843	16.88	19.12	20.35	20.35	Upper Overflow Weir Invert
844	21.10	21.10	21.10	21.10	
845	21.82	21.82	21.82	21.82	
846	22.52	22.52	22.52	22.52	

Table 2 provides the estimated times required for Pond 3 to drain for each analyzed storm event and low level orifice option. The times provided indicate the amount of time required to drain counting after the termination of the 24 hour storm event, as shown in Figure 1 for the recommended orifice condition. Golder has recommended the 6 x 3" orifice outlet configuration as it provides a drainage time under 1 day during the 100-year, 24-hour storm event while provided enough attenuation to protect the plant stormwater infrastructure downstream of the Pond 3 stormwater outlet.

Table 2: Pond 3 Outlet Drain Time Periods 24 Hour Storms

	Time Required to Fully Drain Pond 3 (Days After Rain Event Ends)			
	1 - 3" Orifice Outlet	1 - 6" Orifice Outlet	6 x 3" Orifice Outlets	Full Outlet Pipe Orifice Outlet*
0.75 in. Rain Event	0.4	0.0	0.0	0.0
1.5 in. Rain Event	1.5	0.0	0.0	0.0
2 year, 24 Hour (3.73 in)	4.3	0.7	0.3	0.0

	Time Required to Fully Drain Pond 3 (Days After Rain Event Ends)			
	1 - 3” Orifice Outlet	1 - 6” Orifice Outlet	6 x 3” Orifice Outlets	Full Outlet Pipe Orifice Outlet*
5 year, 24 Hour (4.45 in)	5.0	0.8	0.4	0.0
10 year, 24 Hour (5.06 in)	5.1	1.1	0.5	0.0
25 year, 24 Hour (6.00 in)	5.1	1.1	0.6	0.0
50 year, 24 Hour (6.74 in)	5.1	1.2	0.7	0.0
100 year, 24 Hour (7.52 in)	5.2	1.2	0.8	0.0

*Current outlet modelled as inner diameter of the SDR17 18” culvert pipe (~15.8”) – This change would require modification to the Pond 3 outlet armoring.

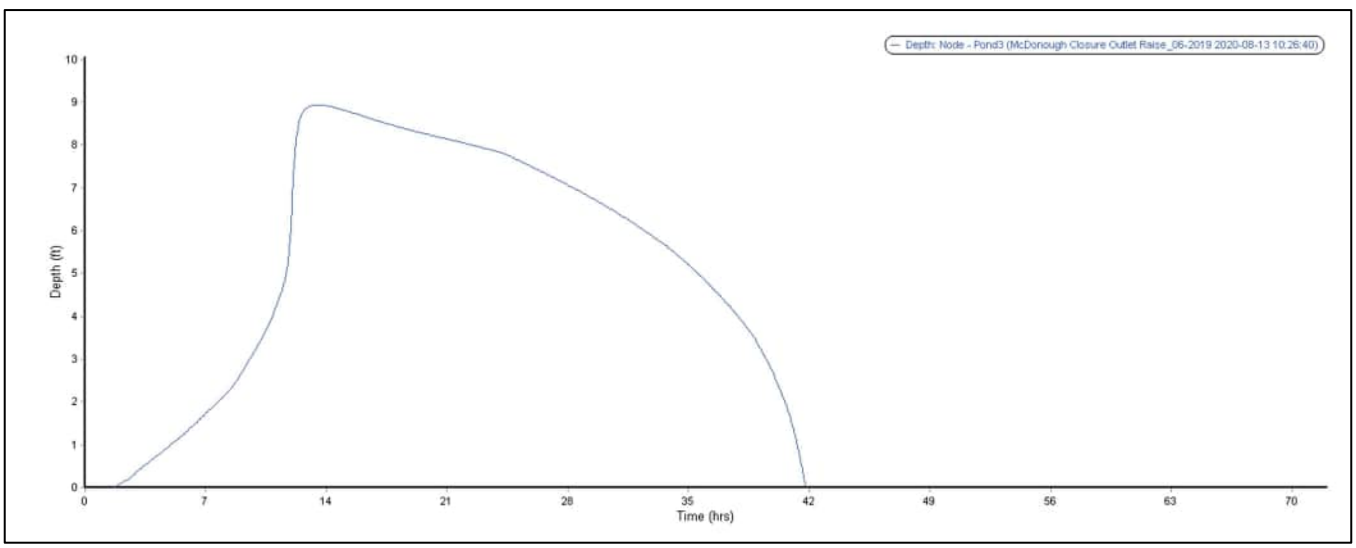


Figure 1: Pond 3 Water Depth During the 100 Year, 24 Hour Storm Event (6 x 3" Low Level Orifice Configuration)