Prepared for

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2016 SURFACE IMPOUNDMENT PERIODIC SAFETY FACTOR ASSESSMENT REPORT

ASH POND A

WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Prepared by

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engineers | scientists | innovators

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CERTIFICATION STATEMENT

This initial safety factor assessment meets the requirements of §257.73(e) of the Code of Federal Regulations Title 40, Part 257, Subpart D, and was prepared in accordance with current practices and the standard of care exercised by scientists and engineers performing similar tasks in the field of civil engineering, and no other warranty is provided in connection therewith. The contents of this report are based solely on the observations of the conditions observed by Geosyntec personnel and information provided to Geosyntec by Santee Cooper. Consistent with applicable professional standards of care, our opinions and recommendations were based in part on data furnished by others. Although we were not able to independently verify such data, we found that it was consistent with other information that we developed in the course of our performance of the scope of services. The information contained in this report is intended for use solely by Santee Cooper and their subconsultants.

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10/13/2016
Date

EXECUTIVE SUMMARY

The Winyah Generating Station (WGS or "Site") is a coal-fired, electric generating facility owned and operated by Santee Cooper and is located approximately four miles southwest of Georgetown, South Carolina (SC). Historically, WGS has utilized six surface impoundments designated for disposal of coal combustion residuals (CCR): Slurry Pond 3&4 (Slurry Pond), West Ash Pond, Unit 2 Slurry Pond, Ash Pond A, Ash Pond B, and the South Ash Pond.

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR (Code of Federal Regulations) Parts 257 and 261, regulating the design and management of existing and new CCR units (commonly referred to as the "CCR Rule"). The CCR Rule became effective on 17 October 2015. The CCR Rule requires owners and operators of existing CCR surface impoundments to conduct periodic safety factor assessments in accordance with §257.73(e) of each impoundment and publish the results to the facility's operating record.

Ash Pond A at WGS is classified as an "existing CCR surface impoundment" by the CCR Rule. This *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* (Safety Factor Assessment Report) presents the first periodic (i.e., initial) safety factor assessment in accordance with the CCR Rule for Ash Pond A at WGS prepared by Geosyntec Consultants (Geosyntec) on behalf of Santee Cooper.

A hydrologic and hydraulic analysis (Attachment 1) of Ash Pond A and its appurtenances was conducted to demonstrate that the inflow design flood (IDF) can be managed and conveyed safely (i.e., without overtopping the perimeter dikes) during and after the rainfall event. Since Ash Pond A has been classified as a "Low Hazard Potential" surface impoundment, the 100-yr rainfall event with a rainfall duration of 72 hours was selected as the IDF. Ash Pond A drains stormwater through a culvert system southward into Ash Pond B. The free water level within Ash Pond B is maintained at an elevation of 34.9 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a concrete riser structure which discharges westward into the Discharge Canal. The peak water level during and after the IDF within Ash Pond A was computed as 38.2 ft NGVD29, which is below the minimum dike crest of 38.8 ft NGVD29. Thus, Ash Pond A will adequately manage inflows during and following the peak discharge from the IDF in accordance with §257.73(d)(1)(v) of the CCR Rule.

In support of the periodic safety factor assessment, Geosyntec developed and performed a geotechnical subsurface investigation and laboratory testing program to characterize the dike fill and subsurface soils for Ash Pond A in 2013 and 2016. Boring logs, Cone Penetration Test (CPT) sounding data, and laboratory testing results have been provided in Attachments 2, 3, and 4 of this Safety Factor Assessment Report, respectively. The interpretation of the in-situ and laboratory data is described and presented in Attachment 5.

Since WGS resides within the Charleston Seismic Zone, a seismic hazard evaluation was performed to select the "maximum horizontal acceleration of lithified material" at the Site corresponding to an earthquake with a probability of exceedance of 2 percent in 50 years (i.e., 2,475 year return period) as defined in §257.53. Site response analyses (Attachment 6) were performed to evaluate the influence of the local subsurface conditions on the maximum horizontal acceleration and to compute the maximum cyclic shear stresses anticipated to occur within the soil profile during the design earthquake.

The potential of the dike fill to liquefy during the design earthquake was evaluated at each soil boring and CPT sounding location (Attachment 7) based on the cyclic shear stresses computed during the site response analyses, in-situ testing data, and laboratory index testing results. The evaluation results did not show that the dike fill soils within the Ash Pond A perimeter dikes or the foundation soils underlying the perimeter dikes of Ash Pond A were susceptible to liquefaction during the design earthquake. It is noted that the liquefaction potential of the foundation soils near the downstream dike toe (i.e., outside the perimeter dike footprint) of Ash Pond A will be evaluated separately as part of an evaluation of "Unstable Areas" in accordance with §257.64 at a later time.

A safety factor assessment (Attachment 8) was performed on five selected cross sections of the perimeter dikes of Ash Pond A to demonstrate that minimum required safety factors provided in §257.73(e)(1) of the CCR Rule are met. Static slope stability was evaluated considering the calculated "Maximum Normal Storage Pool" level (i.e., 34.9 ft NGVD29) and "Maximum Surcharge Pool" level (i.e., 38.2 ft NGVD29) under the anticipated long-term "steady-state" conditions according to the CCR Rule. The minimum safety factors required by the CCR Rule for "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions are 1.50 and 1.40, respectively. Additionally, seismic slope stability with a minimum safety factor of 1.00 was also evaluated for the perimeter dikes of Ash Pond A during "Maximum Normal Storage Pool" conditions. The safety factor assessment results indicated that the selected cross sections of the Ash Pond A perimeter dikes met the minimum required safety factors provided in §257.73(e)(1) of the CCR Rule. It is noted that the safety factor considering post-liquefaction conditions of the dike fill was not evaluated in this Safety Factor Assessment Report, because the dike fill and the foundation soils directly under the perimeter dike were not found to be susceptible to liquefaction. However, the post-liquefaction conditions of the foundation soils outside the footprint of Ash Pond A involving the perimeter dikes may be evaluated as part of the assessment of "Unstable Areas" performed at a later time, depending on the liquefaction potential evaluation results of the foundation soils near the downstream perimeter dike toe.

1. INTRODUCTION

1.1 Project Background

The Winyah Generating Station (WGS or "Site") is an electric generating facility owned and operated by Santee Cooper. WGS is located between Pennyroyal and Turkey Creeks, tributaries to Sampit River, and is situated approximately four miles southwest of Georgetown, South Carolina (SC) (see Figures 1a and 1b for Site Location and Site Vicinity Maps). WGS has historically utilized six surface impoundments (Figure 2) designated for disposal of coal combustion residuals (CCR): Slurry Pond 3&4 (Slurry Pond), West Ash Pond, Unit 2 Slurry Pond, Ash Pond A, Ash Pond B, and the South Ash Pond.

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR Parts 257 and 261, regulating the design and management of existing and new CCR units (commonly referred to as the "CCR Rule"). The CCR Rule became effective on 17 October 2015. Within the CCR Rule, §257.73(e) outlines the safety factor criteria for existing CCR surface impoundments.

Ash Pond A is situated east of the power block and west of the Site's Cooling Pond. Ash Pond A manages CCR in the form of fly ash, boiler slag, and bottom ash as well as process water resulting from power generating activities. Ash Pond A is considered as an existing surface impoundment under the CCR Rule. The *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* (Safety Factor Assessment Report) has been prepared by Geosyntec Consultants (Geosyntec) on behalf of Santee Cooper to demonstrate that Ash Pond A meets criteria for periodic safety factor assessments in accordance with §257.73(e) of the CCR Rule.

1.2 Project Site and Construction History

Ash Pond A, an unlined surface impoundment spanning approximately 90 acres, is located east of the power block and immediately west of the Cooling Pond. It was commissioned in the early 1970s and is designated for the disposal of fly ash, bottom ash, and boiler slag. Ash Pond A is bounded by the Intake Canal to the north, the Discharge Canal to the west, Ash Pond B to the south, and the Cooling Pond to the east. Ash Ponds A and B were constructed simultaneously and are separated by a recompacted, earthen divider dike spanning west to east from the Discharge Canal to the Cooling Pond.

Ash Pond A was constructed by recompacting excavated soils from the impoundment interior to form the perimeter dikes and a divider dike. The Ash Pond A perimeter dikes are approximately 12 ft to 15 ft in height along the north and west sides and approximately 20 ft to 24.5 ft in height along the east side adjacent to the Cooling Pond (Thomas and Hutton, 2012). The upstream and downstream slopes of the perimeter dikes range from 2 Horizontal to 1 Vertical (2H:1V) to 3H:1V. The Ash Pond A dike crest is approximately 12- to 15-ft wide with an approximate elevation between 38.8 ft and 44.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) (Thomas and Hutton, 2012).

Historically, free water within Ash Pond A has been routed southward via rim ditches and a series of culverts into Ash Pond B and subsequently into the Discharge Canal. Ash Ponds A and B are hydraulically connected through a 30-inch (in.) diameter corrugated metal pipe (CMP), a 48-in. diameter smooth steel pipe, and a 42-in. diameter smooth steel pipe (Thomas and Hutton, 2016; Thomas and Hutton, 2012). Ponded water within Ash Pond B is regulated by a concrete riser structure, which discharges into the Discharge Canal through a 24-in. diameter high density polyethylene (HDPE) pipe. Ash Pond A receives low volume wastewater, hydroveyor water, and bottom ash sluice water from electric generating Units 1 and 2. Bottom ash sluice water from Units 3 and 4 is also conveyed into Ash Pond A. Additionally, Ash Pond A receives contact water from the Unit 2 Slurry Pond after a rainfall event, which is pumped across the Intake Canal.

1.3 Report Organization

This Safety Factor Assessment Report presents the first (i.e., initial) periodic safety factor assessment for Ash Pond A at WGS based on the results of subsurface investigations, hydrologic and hydrology (H&H) analysis, geotechnical engineering analyses, and a review of Site information. The remainder of this Safety Factor Assessment Report is organized as follows:

- Descriptions of the hazard potential classification of Ash Pond A and corresponding performance of the hydraulic structures are presented in Section 2;

- Geotechnical subsurface investigations performed by Geosyntec are presented in Section 3;

- Subsurface conditions, geology, and geotechnical properties are discussed in Section 4;

- Selection of the seismic hazard parameters for WGS and the site response analysis of the Ash Pond A perimeter dikes performed by Geosyntec are presented in Section 5;

- Assumptions and results of the liquefaction potential evaluation of the Ash Pond A perimeter dikes are presented in Section 6;

- Slope stability analyses performed for the safety factor assessment are discussed in Section 7; and

- A summary and the general conclusions of the safety factor assessments are presented in Section 8.

2. HYDROLOGIC AND HYDRAULIC EVALUATION

2.1 Hydrologic and Hydraulic Analysis

The following section discusses the regulatory framework, the methodology and assumptions, and the results of the H&H analysis for Ash Pond A and its appurtenances.

2.1.1 Regulatory Framework

The CCR Rule (§257.73(d)(1)) requires that the periodic stability assessment:

"...at minimum, document whether the CCR unit has been designed, constructed, and maintained with:

...

(v) a single spillway or a combination of spillways configured as specified in paragraph (d)(1)(v)(A) of this section. The combined capacity of all spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge event specified in paragraph (d)(1)(v)(B) of this section."

§257.73(d)(1)(v)(B)(3) states that the spillway or spillways must manage the peak discharge from the *"100-year flood for a low hazard potential CCR Surface Impoundment"*. Additionally, §257.73(d)(1)(v)(A) indicates that *"All spillways must be either:*

(1) Of non-erodible construction and designed to carry sustained flows; or

(2) Earth- or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected."

Meanwhile, §257.73(e)(1) of the CCR Rule indicates:

"(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40."

Considering the requirements of §257.73(d)(1) listed above, this Safety Factor Assessment Report utilizes the maximum water elevation within Ash Pond A as computed during the H&H analysis to select the "maximum surcharge pool" elevation to demonstrate that the requirements of §257.73(e)(1)(ii) are met.

The culverts hydraulically connecting Ash Ponds A and B through the divider dike are effectively considered spillways, which manage the discharge during and after the Inflow Design Flood (IDF). The IDF was selected as the 100-year rainfall event because Ash Pond A was assigned a "Low Hazard Potential" classification (Geosyntec, 2016) since a potential failure would be contained within the property boundary and is not anticipated to migrate offsite. H&H analyses were performed to demonstrate that the Ash Pond A culverts are able to adequately manage flow during and following the 100-yr design rainfall (i.e., peak discharge event) without overtopping of perimeter dikes, meeting the criteria in §257.73(d)(1)(v). The results of the H&H analyses are utilized in this Safety Factor Assessment Report to calculate the maximum surcharge pool elevation in support of the safety factor assessment per §257.73(e)(1)(ii).

2.1.2 Methodology and Assumptions

Details of the H&H analysis are provided in a calculation package titled *"Hydrologic and Hydraulic Analysis for Ash Pond A"*, which is included as Attachment 1 of this Safety Factor Assessment Report. The remainder of this section describes the assumptions, conditions, and results of the H&H analysis for Ash Pond A.

The culverts connecting Ash Pond A to Ash Pond B consist of: (i) a 30-in. diameter CMP with an upstream invert at 37.50 ft NGVD 29; (ii) a 48-in. diameter smooth steel pipe with an upstream invert at 35.49 ft NGVD 29; and (iii) a 42-in. diameter smooth steel pipe with an upstream invert at 36.20 ft NGVD 29 (Thomas and Hutton, 2016; Thomas and Hutton, 2012). These culverts allow for the southward conveyance of stormwater and process water from Ash Pond A to Ash Pond B.

Ash Pond A receives contact water from the Unit 2 Slurry Pond after rainfall events. The Unit 2 Slurry Pond is equipped with a 6JSVE Thompson pump operating at a maximum capacity of 2,600 gallons per minute (gpm) (5.79 ft^3/s), which was considered a base flow into Ash Pond A during this evaluation. Low volume wastewater, hydroveyor water, and bottom ash sluice water from Units 1 and 2 and bottom ash sluice water from Units 3 and 4 were considered to have a combined base inflow to Ash Pond A totaling 6,099 gpm (13.59 ft^3/s).

The operating level in Ash Pond B is maintained by a 4-ft by 4-ft concrete riser structure (or spillway) with a top stop log elevation of 34.9 ft NGVD 29 (Thomas and Hutton, 2016) and a 24-in. diameter smooth interior, corrugated HDPE pipe discharging to the Discharge Canal. The tailwater conditions associated with discharge from Ash Pond B into the Discharge Canal were modeled using a fixed water surface elevation within the Discharge Canal and Cooling Pond estimated by conservatively assuming

2.5-ft of free water overtopping the Cooling Pond emergency spillway during a significant rainfall event. The top of the 4-ft by 4-ft stop log bolted to the top of the concrete spillway of the Cooling Pond is at elevation 21.65 ft NGVD 29 (Thomas and Hutton, 2015). The water surface of the Discharge Canal and Cooling Pond was assumed to be at 24.15 ft NGVD 29 (21.65 ft NGVD 29 plus an additional 2.5 ft of water) during the IDF.

HydroCAD® (HydroCAD, 2011) software was utilized to apply the Soil Conservation Service (SCS) Technical Release 20 (TR-20) method (SCS, 1982) to compute the stormwater volume and to model the performance of the hydraulic structures of Ash Pond A during the 100-yr rainfall event. The 100-yr rainfall event was selected with a 72-hour (hr) duration precipitation event resulting in a rainfall depth of 12.8 inches (NOAA, 2006), and modeled within HydroCAD® using a SCS Type III rainfall distribution. The analysis was performed under the following assumptions, which were confirmed by WGS personnel:

- The Site will construct a 100-ft wide emergency spillway with an invert elevation of 37.0 ft NGVD 29 in the divider dike between Ash Ponds A and B by October 2016. The emergency spillway will be constructed with 10H:1V side slopes and will be located between the 48-in. diameter smooth steel pipe and the 42-in. diameter smooth steel pipe.

- Ash Ponds A and B effectively operate as a single surface impoundment with respect to hydraulic performance (i.e., the two ponds are "hydraulically connected").

2.1.3 Analysis Results

Under the conditions and assumptions described in Section 2.1.2, the maximum free water level or "maximum surcharge pool" level during and following the 100-yr rainfall event was computed as 38.2 ft NGVD29 occurring 36.2 hours into the rainfall event.

3. GEOTECHNICAL SUBSURFACE INVESTIGATIONS

This section summarizes the geotechnical subsurface investigation programs performed in the vicinity of the Ash Pond A perimeter dikes at WGS. In the fall of 2013, Geosyntec conducted a focused geotechnical subsurface investigation program to obtain geotechnical data necessary to evaluate closure alternatives for the surface impoundment. Geosyntec returned to the Site in the spring of 2016 and performed an additional geotechnical subsurface investigation to collect subsurface information along the Ash Pond A perimeter dikes and within the interior of Ash Pond A. Historically, soil borings were performed in the vicinity of Ash Pond A prior to construction of the surface impoundment; however, records (i.e., locations, soil boring logs, laboratory testing results, etc.) pertaining to these subsurface investigations were not available during the preparation of this Safety Factor Assessment Report. Figure 3 presents the locations of soil borings and Cone Penetration Test (CPT) soundings performed during these geotechnical subsurface investigations.

The geotechnical data obtained from the 2013 and 2016 geotechnical subsurface investigation programs, including soil borings, CPT sounding data, and laboratory test results, are included in Attachments 2, 3, and 4, respectively. The interpretation of the subsurface stratigraphy and material properties is presented in Attachment 5. The following sections provide summaries of each of the geotechnical subsurface investigations in the vicinity of Ash Pond A.

3.1 Geosyntec Investigations

3.1.1 Fall 2013 Subsurface Investigation

In October 2013, Geosyntec mobilized to WGS to collect geotechnical subsurface data through additional soil borings and CPT soundings in support of evaluating preliminary and conceptual closure alternatives for each CCR surface impoundment at WGS. The subsurface investigation was focused in the vicinity of the South Ash Pond, Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B. In the Ash Pond A area, Geosyntec advanced seven soil borings using the mud rotary wash drilling method and sixteen CPT soundings. Soil Consultants, Inc. (SCI), of Charleston, South Carolina, was the drilling contractor during this investigation. Mid-Atlantic Drilling, Inc. (MAD) from Wilmington, North Carolina performed the CPT soundings. One soil boring and four CPT soundings were advanced within the interior of Ash Pond A and were terminated once native or foundation soils were encountered. The remaining soil borings and CPT soundings were performed on the perimeter and divider dikes and were terminated once refusal was encountered, which was defined as a SPT blow count of 50 blows per foot over an advancement of 6" or the inability to further advance the cone.

During each soil boring, split spoon samples were collected and SPT blow counts (i.e., N-values) were recorded typically in 5-ft depth intervals. Three Shelby tubes were pushed to collect samples in the cohesive foundation soils located in the northwest corner of Ash Pond A. Several other Shelby tubes were pushed to attempt to collect samples within the Ash Pond A perimeter dikes; however, the recompacted dike fill soils were found to be dense and cohesionless and thus, undisturbed samples were unable to be collected. In one soil boring (SPT-117), SCI utilized a tri-cone rotary wash drill bit instead of the side discharge flat drilling bit once the Chicora Member stratum was encountered to penetrate the unit and advance into the underlying formation (i.e., Williamsburg Formation Clay). In SPT-117, a Shelby tube was pushed to collect a sample of the underlying stiff clay for geotechnical laboratory testing. During this geotechnical subsurface investigation, shear wave velocities (V_S) were measured in 5-ft depth intervals at seven CPT soundings (CPT-135, 137, 140, 144, 145, 147, and 150). Additionally, dissipation tests were performed at five CPT soundings (CPT-138, 143, 146, 155, and 157) to evaluate the phreatic surface through the perimeter dikes and within Ash Pond A at the time of the investigation. Soil boring logs and CPT sounding data, including V_S and dissipation tests, are provided in Attachment 3.

In November 2013, Geosyntec installed piezometers as part of the development of a hydrogeological model at WGS. Two piezometers (PPZW-8D and PPZW-9D) were installed by South Atlantic Environmental Drilling and Construction Co. Inc. (SAEDACCO) adjacent to Site monitoring wells (WAP-8 and WAP-9). Prior to installing these piezometers, subsurface soils were collected using a split spoon sampler and logged by a Geosyntec geologist. SPT N-values measured during this installation were interpreted and utilized as a part of this subsurface assessment.

3.1.2 Spring 2016 Subsurface Investigation

In the spring of 2016, Geosyntec performed a geotechnical subsurface investigation predominantly within the interior of Ash Pond A, Ash Pond B, and the Unit 2 Slurry Pond to collect information in support of the design of closure options for each surface impoundment. Within the Ash Pond A interior and along the divider dike, Terracon was subcontracted and performed twelve CPT soundings to evaluate the subsurface stratigraphy underlying the surface impoundments. Three additional CPT soundings (CPT-228, CPT-229, and CPT-229A) were advanced at the perimeter dike crest and dike toe adjacent to the Cooling Pond (east side of Ash Pond A). Additionally, Terracon advanced three soil borings (SPT-304, SPT-305, and SPT-306) within the Ash

Pond A interior to collect soil samples for laboratory testing. The laboratory testing program for soil samples collected during this investigation consisted of particle size distribution analysis, moisture content tests, and Atterberg limits tests.

3.1.3 Laboratory Testing

During these geotechnical subsurface investigations, Geosyntec subcontracted Excel Geotechnical Testing, Inc. (EGT) of Roswell, Georgia (fall 2013) and Terracon (spring 2016) to conduct a geotechnical laboratory testing program on representative disturbed (i.e., bulk or split spoon) and undisturbed (i.e., Shelby tube) samples. The 2013 geotechnical laboratory testing program on dike fill and foundation soils included fourteen grain size distribution tests (four with hydrometer tests), fifteen fines content tests (to supplement the grain size distribution tests), nine Atterberg limits tests, twenty-nine natural water content tests, three shear strength tests (2- to 3-point consolidated-undrained (CU) triaxial tests), and two one dimensional (1-D) consolidation tests. Additionally, two CU triaxial tests and two 1-D consolidation tests (with index tests included) were performed on thin-walled Shelby tube samples of impounded fly ash collected from the interior of Ash Pond A. Several grain size distribution tests and one hydraulic conductivity test were performed on the Williamsburg Formation Clay collected from SPT-117. Samples collected during the spring 2016 geotechnical subsurface investigation were predominantly tested to evaluate select samples for particle size distribution, Atterberg limits, and natural moisture content. Laboratory testing results from each geotechnical subsurface investigation are provided in Attachment 4 and the interpretation of the laboratory testing results is discussed in Attachment 5.

4. SUBSURFACE CONDITIONS AND GEOTECHNICAL PROPERTIES

This section presents subsurface conditions, phreatic surface and free water levels, and material properties for Ash Pond A based on the geotechnical subsurface investigation programs discussed in Section 3. A summary of the regional geology is also provided as a framework to develop the subsurface stratigraphy model. Additional information on the subsurface conditions and the material properties is presented in Attachment 5 of this Safety Factor Assessment Report.

4.1 Regional Geology

Georgetown County, SC is located in the Atlantic Coastal Plain physiographic province, which is characterized by Quaternary terrace deposits produced by fluctuating sea levels. Coastal Plain sediments are underlain by Tertiary and late Cretaceous sediments to a depth of approximately 2,200 ft below ground surface (bgs) in the Georgetown area. Descriptions of geologic units of interest in the area have been referenced from Campbell and Coes (2010) and are summarized below from top to bottom. The approximate thicknesses of each unit were estimated from several borings referenced in Campbell and Coes (2010). The specific borings used for this estimation include: 1) CHN-0820 located approximately 12 miles to the south of WGS; 2) GEO-0088 located approximately 7 miles to the southeast of WGS; and 3) GEO-0185 located less than 1.5 miles to the northwest of WGS.

- Undifferentiated Quaternary sediments consist of yellowish-brown and reddish-orange poorly sorted, very fine to very coarse, clayey sand and gravel. Accessory minerals include opaque heavy minerals, mica, and feldspar. The reported thickness of Undifferentiated Quaternary sediments ranges between 20 and 42 ft in the area.
- The Williamsburg Formation (Williamsburg) consists of gray to black interbedded clay and coarse quartz sand overlying shelly clay and calcareous clay. The Williamsburg can include sandy shale, fuller's earth, fossiliferous clayey sand (Lower Bridge Member), and fossiliferous clayey sand and mollusk-rich, bioclastic limestones (Chicora Member). The reported thickness of the Williamsburg in the vicinity of the site ranges between 30 and 90 ft.
- The Lang Syne Formation (Muthig and Colquhoun, 1988) was described as consisting of red and yellow (where weathered) or white, gray, and black (where freshly exposed) interbedded sand, silt, and clay and thin beds of silicified shell debris. Opaline clay stone is the most characteristic lithology.

- The Rhems Formation which consists of light-gray to black shale interlaminated with thin seams of fine-grained sand and mica.
- The Peedee Formation which consists of a dark-green to gray, fossiliferous, glauconitic clayey sand and silt. The combined thickness of the Lang Syne and Rhems and Peedee Formations ranges between 185 and 378 ft in the vicinity of the WGS.

Additional late Cretaceous Formations are present to a depth of approximately 2,200 ft bgs in the area. These Formations, in descending order, include: Donoho Creek, Bladen, Coachman, Cane Acre, Caddin, Sheppard Grove, Pleasant Creek, Cape Fear and undifferentiated Cretaceous sediments. The most important geologic units for this report are the undifferentiated Quaternary and Williamsburg Formations, which are encountered within 60 to 100 ft bgs as described in detail by Doar (2012).

4.2 <u>Perimeter Dike Subsurface Conditions and Water Levels</u>

4.2.1 Subsurface Stratigraphy

The subsurface stratigraphy at the Site was developed from information obtained from geotechnical investigations at WGS and from regional geologic data. The information indicates that the subsurface soils primarily consist of four geotechnical units, within the depths of interest for the analyses presented in this Safety Factor Assessment Report. A brief description on each unit is presented as follows:

- Dike Fill: Dike fill soils for the Ash Pond A perimeter dikes were generally observed to be medium dense to very dense, poorly graded silty sands with uncorrected SPT blow counts typically ranging between 7 and 66 blows per foot and measured CPT tip resistances typically ranging between 100 and 450 tsf. Grain size distribution analyses indicated that these dike fill soils typically consist of 72 percent to 87 percent sand–sized particles (smaller than No. 4 sieve but greater than No. 200 sieve) and 6 percent to 28 percent silt and clay-sized particles (i.e., "fines" with diameters smaller than a No. 200 sieve), with most samples containing less than 15 percent fines.

- Foundation Soils: Foundation soils were observed to be variable across the Ash Pond A footprint. The foundation materials consist primarily of poorly graded silty sands with shells and a few isolated seams of clayey sand or high plasticity clay. Uncorrected SPT blow counts within foundation soils ranged between 0 and 61 blows per foot, with clayey material generally having a lower measured blow count then sandy material. Tip resistances generally ranged between 25 and 300 tsf (generally below 50 tsf).

- Chicora Member: A layer of dense to very dense soil consisting of partially cemented to heavily cemented shells was encountered beneath the foundations soils during subsurface investigations at WGS. SPT blow counts in this layer exceeded 50 blows over less than 6 in. of advancement with minimal sample recovery. The thickness of this layer, particularly the cemented layers of the material, varied across the Site. Based on review of historical and existing data (Doar, 2012), this layer is the upper portion of the overall Williamsburg Formation and is referred to as the "Chicora Member", "Coquina", or "Shell Hash". The term "Chicora Member" or "Chicora" is used to refer to this soil unit throughout this Safety Factor Assessment Report. Boring and CPT refusal was typically encountered at the top of this stratum, though two borings within the Ash Pond A area penetrated this stratum.

- Williamsburg Formation Clay: The Williamsburg Formation Clay was encountered beneath the Chicora Member. The Williamsburg Formation Clay is described as stiff to very hard, dark gray to black, medium to high plasticity clay or silt with sand. The Williamsburg Formation Clay has historically been referred to as "Black Mingo Clay" or the "Black Mingo Formation" at the Site. The term "Williamsburg Formation Clay" is the most recent geological term for this stratum and is used throughout this Safety Factor Assessment Report. The Williamsburg Formation Clay was found to be between 30-ft and 90-ft thick in the vicinity of WGS based on a review of the regional geology.

4.2.2 Water Levels

As described within the H&H analysis for Ash Pond A provided in Attachment 1 of this Safety Factor Assessment Report, the free water level within Ash Pond B, located to the south of Ash Pond A, is maintained at an elevation of 34.9 ft NGVD29 by a 4-ft by 4-ft concrete riser structure. Ash Pond A does not typically contain free water, but conveys stormwater and process water through a series of rim ditches and a series of culverts into Ash Pond B. A 30-in. diameter CMP, a 48-in. diameter smooth steel pipe, and a 42-in. diameter smooth steel pipe convey free water from the rim ditches through the northeast corner of the divider dike into Ash Pond B. The concrete riser structure in

Ash Pond B maintains free water at an operating elevation of 34.9 ft NGVD29 and discharges free water westward into the Discharge Canal.

The phreatic surface through the Ash Pond A perimeter dikes to the downstream toe at the time of this Safety Factor Assessment Report was predominantly developed based on water levels collected from results of porewater pressure dissipation tests conducted during CPT soundings, depth to water measurements within boreholes, and the Cooling Pond free water elevation. A temporary piezometer installed within the interior of Ash Pond A (PPZ-AS-1) indicates that the phreatic elevation within the CCR at the center of Ash Pond A has ranged between 36.0 and 37.2 ft NGVD29 since installation. Thus, the phreatic surface elevation within the center of the surface pond was selected as 37.2 ft NGVD29 and assumed to transition to 34.9 ft NGVD29 adjacent to the perimeter dikes. The water level of the Cooling Pond was selected as 19.1 ft NGVD29 based on the operating level of the Cooling Pond required to manage runoff from the 25-yr, 24-hr rainfall event. The maximum free water elevation during the IDF within Ash Pond A was computed as 38.2 ft NGVD29 (Section 2), which was used to represent the "Maximum Surcharge Pool" level within this Safety Factor Assessment Report.

In 2015, WGS installed supplementary groundwater monitoring wells (WAP-12, WAP-17, WAP-18, and WAP-19) at the downstream dike toe and perimeter dike crest of Ash Pond A. On 21 June 2016, the phreatic surface elevation was measured as 23.8 ft NGVD29 at the dike toe and between 25.9 ft and 26.7 ft NGVD29 through the dike crest.

4.3 Coal Combustion Residuals (CCR)

As noted in Sections 3.1, four soil borings and several CPT soundings have been advanced within the interior of Ash Pond A during geotechnical subsurface investigations. Numerous geoprobe borings have been advanced by Geosyntec within the interior of Ash Pond A to evaluate the location of the bottom of the surface impoundment and to estimate the volume of CCR contained within the surface impoundment. Ash Pond A contains predominantly fly ash, which was found to be soft, black, sandy silt with SPT blow counts between 0 (i.e., weight of hammer) or 2 blows per foot. The measured CPT tip resistance of ponded fly ash ranged between 5 tsf and 75 tsf with the higher tip resistance values observed in the upper 5 ft bgs.

4.4 Material Parameters

Representative parameters of subsurface materials were selected based on in-situ and laboratory testing results, as discussed in Attachment 5. Additionally, correlations based on in-situ testing methods were applied to supplement laboratory testing, in particular, the shear strength testing results for the dike fill and foundation soils. Shear strength parameters were selected from these results, which correspond to the current range of overburden stresses experienced in the vicinity of Ash Pond A. A summary of the material parameters selected for the safety factor assessment are presented in Table 1.

Representative shear wave velocity (V_S) profiles were developed based on direct measurements from seismic CPT (SCPT) soundings and empirical correlations using the CPT sounding results. The development of these V_s profiles is presented in Attachment 5 and subsequently applied in the site response analysis discussed in Section 5.2 of this Safety Factor Assessment Report.

5. SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

This section presents the results of seismic hazard evaluation and site response analysis of the Ash Pond A perimeter dikes. Seismic hazard evaluation includes the selection of an appropriate hazard level and associated hazard parameters (e.g., Peak Ground Acceleration [PGA]). Site response analysis was performed to evaluate the local site effects on selected time history records propagated from the hypothetical firm ground outcrop to the ground surface at the Site. Details and results for these analyses are presented in Attachment 6 and summarized herein.

5.1 Seismic Hazard Evaluation

A seismic hazard evaluation typically consists of the selection of appropriate hazard level and associated earthquake parameters, which include the target acceleration response spectra and PGA and the controlling earthquake magnitude. The seismic hazard analysis also involves the selection of ground motions that envelop the target response spectrum.

5.1.1 Seismic Hazard Level

The appropriate hazard level is often expressed in probabilistic terms as a specific hazard level that has a certain probability of exceedance in a given time period. The CCR Rule states in §257.63(a) that:

"New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones, unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site."

§257.53 defines the maximum horizontal acceleration in lithified earth material as:

"... the maximum expected horizontal acceleration at the ground surface as depicted on a seismic hazard map, with a 98 percent or greater probability that the acceleration will not be exceeded in 50 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment."

In accordance with the CCR Rule, the analysis presented in this Safety Factor Assessment Report was based on establishing seismic design parameters (i.e., PGA) consistent with a 98 percent or greater probability that the PGA will not be exceeded in 50 years. This results in a PGA with return period of 2,475 years, which is commonly referred to as the 2,500-year event PGA.

5.1.2 Peak Ground Acceleration (PGA)

PGA values corresponding to different hazard levels and different site conditions, including firm ground outcrops, are published as seismic hazard maps. While United States Geological Survey (USGS) national seismic hazard maps are the most commonly used resources for the selection of PGA, regional seismic hazard maps developed by local experts consider regional geologic setting and seismicity and are often the preferred alternatives.

USGS national seismic hazard maps for a 2 percent probability of exceedance in 50 year ground motion (i.e., 2,475-year return period event) provide the PGA and spectral accelerations for a hypothetical firm ground outcrop at the Site. The software available at the USGS website (USGS, 2008) uses pre-calculated hazard values at nearby grid locations and interpolates the hazard value for a given site location. As discussed within Attachment 6, the USGS interpolated PGA is 0.469g for the Site.

The South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (GDM) (SCDOT, 2010) also provides seismic hazard maps for "geologically realistic" site conditions as well as for the hypothetical "hard-rock" conditions. The SCDOT seismic hazard maps were developed by Chapman and Talwani (2006) to incorporate their local experience and research over several decades for the Charleston Seismic Zone. The "geologically realistic" site condition is a hypothetical site condition that was included via a depth-dependent transfer (i.e., site amplification) function for Coastal Plain and non-Coastal Plain regions of SC. According to these hazard maps, the Site PGA is 0.16g for "geologically realistic" conditions.

As mentioned above, the SCDOT (2010) hazard maps were developed by local experts who have spent several decades studying the Charleston Seismic Zone. A review of V_S profiles developed for WGS site indicates that use of "geologically realistic" conditions is more appropriate for the seismic analysis and site response. Therefore, the SCDOT hazard maps for "geologically realistic" conditions were used to select the PGA (i.e., 0.16g) for this Safety Factor Assessment Report. Additional discussion with respect to the selection of the PGA is provided in Attachment 6.

5.1.3 Earthquake Magnitude

In a probabilistic seismic hazard analysis, the PGA cannot be associated with a single earthquake event due to the hazard contribution from multiple possible events. An earthquake moment magnitude (M_W) value is required to conduct liquefaction potential analyses and to select earthquake time histories. A process called deaggregation can be performed for sites that have multiple hazard sources using the USGS (2002) deaggregation tool. As discussed within Attachment 6, a 7.3 moment magnitude was selected for liquefaction potential analyses and time history selection for WGS by applying this deaggregation tool.

5.1.4 Target Acceleration Response Spectra and Time History Selection

A target acceleration response spectrum was selected using the SCDOT seismic hazard maps for a "geologically realistic" site at different spectral periods (or frequencies). The "geologically realistic" target acceleration response spectrum has a PGA (represented by a spectral period of 0.01 seconds) of 0.16g and a peak spectral acceleration of 0.48g at a spectral period of 0.2 seconds. As stated previously, the "geologically realistic" condition target acceleration response spectrum was selected for WGS.

Time histories of ground motions are used as input for site response analysis and are selected such that their response spectrums match or envelope the target spectrum. While use of recorded ground motion time histories from earthquakes with similar source characteristics is preferred, synthetic motions may be used if recordings are not available for a particular seismic zone. Earthquake events with a moment magnitude, M_W, 7.0 or greater have not occurred in the stable continental tectonic environment of the Central and Eastern United States since the Charleston earthquake in 1886, so ground motion time history records matching the seismic source characteristics for WGS are generally not available. Two synthetic acceleration time histories were selected from the six synthetic acceleration time histories developed for the Site using the USGS Interactive Deaggregation tool (USGS, 2002). These time histories are referred to herein as Winyah1 and Winyah2, and provide a reasonable match to the short-period portion of the "geologically realistic" target acceleration response spectrum. Three time histories, BOS-T1, DEL090, and YER360, developed by McGuire et al. (2001) as part of a study for the Nuclear Regulatory Commission to provide time histories representative of expected earthquake events in the Central and Eastern United States were selected to provide a reasonable match with the long-period portion of the "geologically realistic" target acceleration response spectrum. One time history, RSN8529-HNE, from the Next Generation Attenuation – East database (Goulet et al., 2014), which provides a database of time histories recorded for earthquake events in the Central and Eastern United States, was selected to also provide a reasonable match with the "geologically realistic" target acceleration response spectrum for longer periods.

5.2 Site Response Analysis

Site response analysis performed during the seismic evaluation computed the cyclic shear stresses within representative soil profiles located along the perimeter dike centerline. Computed cyclic shear stresses were applied for the liquefaction potential analysis, and were also utilized to evaluate the seismic safety factor as a part of the safety factor assessment.

5.2.1 Analysis Model Setup

Site response analyses presented herein were conducted using DEEPSOIL® (Hashash et al., 2015), a one-dimensional, nonlinear site response analysis program. The program assumes that all the soil layers are perfectly horizontal (i.e., "layer cake") and that ground response is mainly caused by vertically-propagating, horizontally polarized shear waves. This assumption is valid for many geotechnical cases including the site response analyses at the Site. Under these assumptions, the subsurface stratigraphy is modeled as a one-dimensional column of soil layers for the analyses. Two representative profiles were developed for the Ash Pond A perimeter dikes and are shown on Figure 4 and in Attachment 6.

DEEPSOIL® employs a viscoelastic material model, described by its shear modulus (G), mass density (ρ) or unit weight (γ), and damping (D). Preliminary equivalent-linear site response analyses yielded calculated maximum cyclic shear strains greater than 5 percent in some layers, which is greater than the cyclic shear strains for which equivalent-linear analyses are considered applicable (i.e., 1 to 2 percent). Therefore, nonlinear site response analyses were performed. Additional discussion of input parameters, such as the V_S profile, soil plasticity, and shear modulus reduction/damping curves applied in the DEEPSOIL® program, are discussed in Attachment 6. The six selected ground motions used within these analyses are also provided within Attachment 6.

5.2.2 Site Response Analysis Results

Maximum horizontal accelerations, maximum shear strains, and maximum shear stresses within the representative soil profiles were computed, as presented in Attachment 6.

The maximum cyclic shear stresses at selected depths for each profile (Table 2) were calculated and used to calculate Cyclic Stress Ratios (CSR) in the evaluation of liquefaction potential, presented in Section 6 of this Safety Factor Assessment Report. The maximum cyclic shear stresses were also used to calculate the horizontal seismic coefficient (k_h) as presented in Section 7 of this Safety Factor Assessment Report.

6. EVALUATION OF LIQUEFACTION POTENTIAL

This section presents the liquefaction potential evaluation for the Ash Pond A perimeter dikes and foundation soils underlying the perimeter dikes. The evaluation applies the cyclic shear stress computed as part of the site response analysis (Section 5) and the interpretation of the in-situ testing data (Section 3). Further details of the liquefaction potential evaluation are presented in Attachment 7.

6.1 Regulatory Framework

A periodic safety factor assessment is required by the CCR Rule to evaluate whether the existing CCR surface impoundments meet minimum safety factors (also referred to as "factors of safety") in §257.73(e)(1). Specifically, §257.73(e)(1)(iv) requires that:

"embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20."

The purpose of this Section is to discuss the methodology, analysis, and results of the liquefaction potential analysis in order to evaluate if the Ash Pond A dike fill and foundation soils are susceptible to liquefaction. If soils are not found to be liquefiable within the dike, then the liquefaction factor of safety is not required and is not evaluated as a part of this periodic safety factor assessment.

6.2 Methodology

Liquefaction potential analysis was performed based on the Simplified Procedure recommended by Seed and Idriss (1971) and the subsequent update by Idriss and Boulanger (2008). This approach is based on comparing in-situ test results with case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. The analyses presented herein were conducted for both the soil borings and the CPT soundings performed during the Geosyntec geotechnical subsurface investigations presented in Section 3. The criteria recommended by Bray and Sancio (2006) were applied to evaluate the susceptibility of fine-grained soils to cyclic softening. All of the tested samples were found to be "Not Susceptible" to cyclic softening by these criteria.

6.2.1 Dike Phreatic Surface Conditions

The phreatic surface through the Ash Pond A perimeter dikes to the downstream dike toe at the time of the liquefaction potential analysis was developed based on water levels collected from water levels measured from borehole and CPT porewater pressure

(u_o) signatures and porewater pressure dissipation tests. Operations of Ash Pond A (i.e., CCR disposal and sluicing rates) have not changed significantly since the fall 2013 geotechnical subsurface investigation (Section 4.2.2), so these water level measurements were considered representative of steady state and anticipated phreatic surface conditions.

6.2.2 Age Correction Factor

Correlations associated with liquefaction potential analysis were developed based on case histories of relatively young soil deposits (i.e., Holocene age). As described in SCDOT (2010), liquefaction resistance, as modeled by the Cyclic Resistance Ratio (CRR), may be adjusted to account for aging effects in older soils based on time from deposition (i.e., geologic age) and time from last occurrence of liquefaction (i.e., geotechnical age). As described in Attachment 7, an age correction factor (K_{dr}) of 1.3 was applied for the Pleistocene age soils at the WGS site (typically foundation soils). The transition of dike fill into foundation soils was selected as the surveyed surface elevations of borings and soundings performed at the base of the Ash Pond A perimeter dikes or based on the elevation of the Cooling Pond when data was not available.

6.3 Evaluation Results

The factor of safety against liquefaction (FS_{liq}) was computed at each interval where in-situ data was collected for each soil boring (2-ft or 5-ft intervals) and each CPT sounding (0.16-ft intervals) advanced in the vicinity of the Ash Pond A perimeter dikes. FS_{liq} values computed for dike fill soils were calculated to exceed 1.0 for the conditions described within this Safety Factor Assessment Report. Analysis results for each boring and CPT sounding analyzed are provided as figures within Attachment 7 of this Safety Factor Assessment Report. Based on these analyses, the dike fill and foundation soils underlying the Ash Pond A perimeter dikes were not found to be susceptible to liquefaction during the design earthquake and thus, the liquefaction safety factor of the perimeter dike is not required to be evaluated during the periodic safety factor assessment. It is noted that the post-liquefaction conditions of the foundations soils outside the footprint of the Ash Pond A perimeter dikes may be evaluated as part of the assessment of "Unstable Areas" performed at a later time, depending on the liquefaction potential evaluation results of the foundation soils near the downstream perimeter dike toe.

7. SAFETY FACTOR ASSESSMENT

This section presents the first (i.e., initial) periodic safety factor evaluation for the Ash Pond A perimeter dikes. This evaluation is presented in detail in Attachment 8 and summarized herein.

7.1 Regulatory Framework

Slope stability analyses were conducted to assess whether Ash Pond A meets the safety factor (also referred to as "factor of safety") requirements of §257.73(e)(1) of the CCR Rule. Specifically, §257.73(e)(1) requires that:

> *(i) "The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.*
> *(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*
> *(iii) The calculated seismic factor of safety must equal or exceed 1.00.*
> *(iv) For embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20."*

The remainder of Section 7 describes the geometric model, methodology, and analysis results for each case.

7.2 Analysis Models

Subsurface cross sections were developed through the perimeter dikes of Ash Pond A based on the information obtained from several sources: (i) recent topographic surveys (Thomas and Hutton, 2012; Thomas and Hutton, 2016); (ii) surveyed Cooling Pond transects performed by Parker Land Surveying, LLC.; (iii) available engineering reports and drawings for WGS; (iv) subsurface stratigraphy developed from geotechnical subsurface investigations (Section 4); and (v) water level measurements (Section 4.2.2). Five representative cross sections (Cross Sections A through E) were selected based on geometric and subsurface conditions. Figures 5 through 10 present the locations of the cross sections within Ash Pond A and the geometry of each selected cross section, respectively.

7.3 Methodology

7.3.1 Static Slope Stability

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE®, version 6.037 (Rocscience, 2015). Spencer's method, which satisfies vertical and horizontal force equilibrium as well as moment equilibrium, is considered to be more rigorous than other methods, such as the simplified Janbu method (Janbu, 1973) and the simplified Bishop method (Bishop, 1955).

Both the rotational mode (i.e., the circular slip surfaces) and the non-rotational mode (i.e., the block slip surfaces) were considered during the factor of safety assessment analyses, and the slip mode resulting in the lowest calculated FS was reported. SLIDE® generates potential slip surfaces, calculates the FS for each of these surfaces, and identifies the most critical slip surface with the lowest calculated FS. Information required for these analyses includes the slope geometry, the subsurface soil stratigraphy, the phreatic surface elevation, the external loading conditions, and the properties of subsurface materials.

7.3.2 Seismic Slope Stability

Pseudo-static slope stability analyses were performed utilizing Spencer's method as described in Section 7.3.1 to evaluate the seismic performance of the perimeter dike structures and a procedure consistent with a guidance document prepared for the USEPA (USEPA, 1995) and recommendations made by Hynes-Griffin and Franklin (1984). The seismic factor of safety was evaluated by applying a seismic horizontal force coefficient (k_h) and an additional horizontal force ($F = k_h \times W$) to each slice during a seismic event based on the weight of the slice. The k_h for each evaluated cross section was developed from the Maximum Horizontal Equivalent Acceleration (MHEA) computed during the site response analysis (Section 5) at the depth of the anticipated critical slip surface for each cross section. The k_h value is dependent on the allowable displacement (u) for an embankment or dike structure. For the purpose of this Safety Factor Assessment Report, the allowable displacement of the Ash Pond A perimeter dikes was selected as 12 inches (30.48 cm). Based on this allowable displacement and the upper bound relation, Hynes-Griffin and Franklin (1984) was applied to adjust the MHEA at the target depth by 0.5 to compute the k_h applied within SLIDE®.

7.4 Static Safety Factor – Maximum Normal Storage Pool

§257.73(e)(1)(i) requires that the static factor of safety meets or exceeds 1.50 for the maximum normal storage pool conditions within the surface impoundment. The static safety factor was evaluated for Cross Sections A through E assuming that the free water level within Ash Pond A was approximately the same as Ash Pond B, maintained at 34.9 ft NGVD29 by a concrete riser structure. Rim ditches adjacent to the north and east perimeter dikes convey surface and sluice water southward into Ash Pond B. It was assumed that the phreatic surface within the interior of Ash Pond A gradually increased to a peak elevation of 37.2 ft NGVD29 based on measurements collected from PPZ-AS-1.

7.5 Static Safety Factor – Maximum Surcharge Pool

§257.73(e)(1)(ii) requires that the static factor of safety meets or exceeds 1.40 for the maximum surcharge pool conditions within the surface impoundment. The static safety factor was evaluated for Cross Sections A through E assuming that the free water level within the Ash Pond A was maintained at 38.2 ft NGVD29 and steady state conditions had been established within the perimeter dikes. The maximum surcharge pool elevation of 38.2 ft NGVD29 was computed as the peak free water level within Ash Pond A during and following the 100-yr rainfall event (Section 2).

7.6 Seismic Safety Factor – Maximum Normal Storage Pool

§257.73(e)(1)(iii) requires that the seismic factor of safety meets or exceeds 1.00 for the maximum normal storage pool conditions within the surface impoundment. The seismic safety factor was evaluated for Cross Sections A through E by applying a computed seismic horizontal force coefficient of 0.025 to 0.030 for Cross Sections A, B and E and 0.035 to 0.0375 for Cross Sections C and D to each slice within SLIDE®. As described in Section 7.4, the Ash Pond A free water level was assumed to be approximately the same as Ash Pond B (i.e., maintained at 34.9 ft NGVD29 by a concrete riser structure). Rim ditches adjacent to the north and east perimeter dikes convey surface and sluice water southward into Ash Pond B. It was assumed that the phreatic surface within the interior of Ash Pond A gradually increased to a peak elevation of 37.2 ft NGVD29 based on measurements collected from PPZ-AS-1. During the evaluation of the Seismic Safety Factor, the undrained shear strength of cohesive soils was reduced by 20% to account for the influence of cyclic degradation (Hynes-Griffin and Franklin, 1984).

7.7 Liquefaction Safety Factor - Maximum Normal Storage Pool

257.73(e)(1)(iv) requires that the liquefaction factor of safety meet or exceed 1.20 for the maximum normal storage pool conditions within the surface impoundment if embankment soils are potentially liquefiable. As described in Section 6 of this Safety Factor Assessment Report, the perimeter dike fill and underlying foundation soils of Ash Pond A were not found to be liquefiable. Thus, a liquefaction safety factor assessment is not required to be evaluated for these conditions.

7.8 Summary of Results

The calculated minimum safety factor for each analysis case and each of these Cross Sections A through E are summarized in Table 3. Cross Section A was calculated to have the lowest safety factor for the seismic safety factor case; while Cross Section D was calculated to have the lowest safety factor for the static safety factor cases. The results corresponding to the lowest calculated safety factor for the three evaluated scenarios are provided in Figures 11 through 13. These results indicate that the perimeter dikes of Ash Pond A at WGS meet the periodic safety factor assessment criteria required by §257.73(e)(1) of the CCR Rule. Further details of the safety factor assessment for Ash Pond A can be found in Attachment 8.

8. SUMMARY AND GENERAL CONDITIONS

The following section provides a summary and general conclusions of the safety factor assessment presented in this Safety Factor Assessment Report:

- The hydrologic and hydraulic performance of Ash Pond A during the 100-yr rainfall event was evaluated and the calculated maximum surcharge pool elevation within the surface impoundment was used in the safety factor assessment.

- A desktop review of site history and engineering reports (when available), geotechnical subsurface investigations, and laboratory testing programs was carried out to evaluate the construction history, characterize the dike and subsurface soils, and understand the existing conditions of Ash Pond A.

- The seismic hazard evaluation resulted in the selection of the design "bedrock" PGA as 0.16g at the Site. This bedrock PGA corresponds to a seismic event with a probability of exceedance of 2 percent in 50 years as required by the CCR Rule and represents a peak ground motion corresponding to "geologically realistic" conditions. Site response analyses were performed to compute the maximum cyclic shear stresses and maximum horizontal equivalent accelerations, which were applied to evaluate the liquefaction potential and seismic safety factors of the Ash Pond A perimeter dikes.

- The evaluation of liquefaction potential indicated that the dike fill soil and foundation soils underlying the Ash Pond A perimeter dikes were not liquefiable and the evaluation of the liquefaction safety factor was not required during the periodic safety factor evaluation. Further evaluation of liquefaction within foundation soils near the downstream perimeter dike toe (i.e., outside the perimeter dike footprint) will be presented in a subsequent evaluation of "Unstable Areas" in accordance with §257.64 at a later time.

- Based on the safety factor assessment of five representative cross sections of the Ash Pond A perimeter dikes, Ash Pond A meets the required safety factors presented in §257.73(e)(1).

Based on the evaluations presented within this Safety Factor Assessment Report, Ash Pond A satisfies the periodic safety factor criteria for existing surface impoundments described within §257.73(e) of the CCR Rule.

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TABLES

Table 1. Summary of Selected Geotechnical Material Properties

Material	Total Unit Weight (pcf)[3]	Drained Parameters		Undrained Parameters[1]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill	125	38 to 40[4]	0	-	-
Clayey Foundation Soils	100	18	250	Varies[5]	100
Sandy Foundation Soils	115	31 to 34[4]	0	-	-
Loose Foundation Soils	110	20[4]	0	-	-
Chicora	130	50[3]	0	-	-
Williamsburg Formation Clay	105	50[3]	-	-	-
Fly Ash	100	34[3]	0	-	-

Notes:

1. pcf = pounds per cubic feet; ϕ' = effective friction angle; c' = cohesion intercept; S_u/σ_v' = undrained shear strength ratio; and $S_{u,\,min}$ = minimum undrained shear strength.
2. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only. Dike fill soils were observed to consist primarily of poorly graded to silty sands in the vicinity of Ash Pond A.
3. The selection of shear strength parameters for Chicora, Williamsburg Formation Clay, and Fly Ash, as well as total unit weights for all materials, is explained in the Attachment 5.
4. These drained shear strengths (ϕ') vary by location. Interpretation of in-situ results applied in the selection is provided in Figures 7 through 10 of Attachment 8.
5. The selected undrained strength ratio (S_u/σ'_{vo}) varies between locations and ranges from 0.25 to 0.40 for the selected cross section. Interpretation of in-situ results applied in the selection is provided in Figure 7 through 10 of Attachment 8. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in Attachment 5.

Table 2. Summary of Calculated Cyclic Shear Stresses

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	31.4	2.5	40.3
7.5	60.3	7.5	98.7
12.5	94.6	12.5	140.9
17.5	130.7	17.0	176.0
22.5	163.1	20.5	205.8
27.5	192.5	24.5	238.9
32.5	211.7	29.5	269.7
37.5	224.6	34.5	291.5
42.5	226.7	39.5	305.4
47.5	235.3	44.5	314.2
52.5	316.9	49.5	327.4
60.0	391.9	54.5	417.6
70.0	523.4	62.0	525.1
80.0	582.9	72.0	631.5
90.0	659.9	82.0	735.0
100.0	782.6	92.0	827.3
-	-	102.0	908.2

Notes:

1. Profiles were developed in the Site Response Package provided as Attachment 6.
2. For calculation points located in between the depth intervals listed above, the average τ_{max} was linearly interpolated for liquefaction potential computations.
3. Profile 1 corresponds to the Ash Pond A perimeter dikes adjacent to the Intake and Discharge Canals; while, Profile 2 corresponds to the Ash Pond A perimeter dikes adjacent to the Cooling Pond.

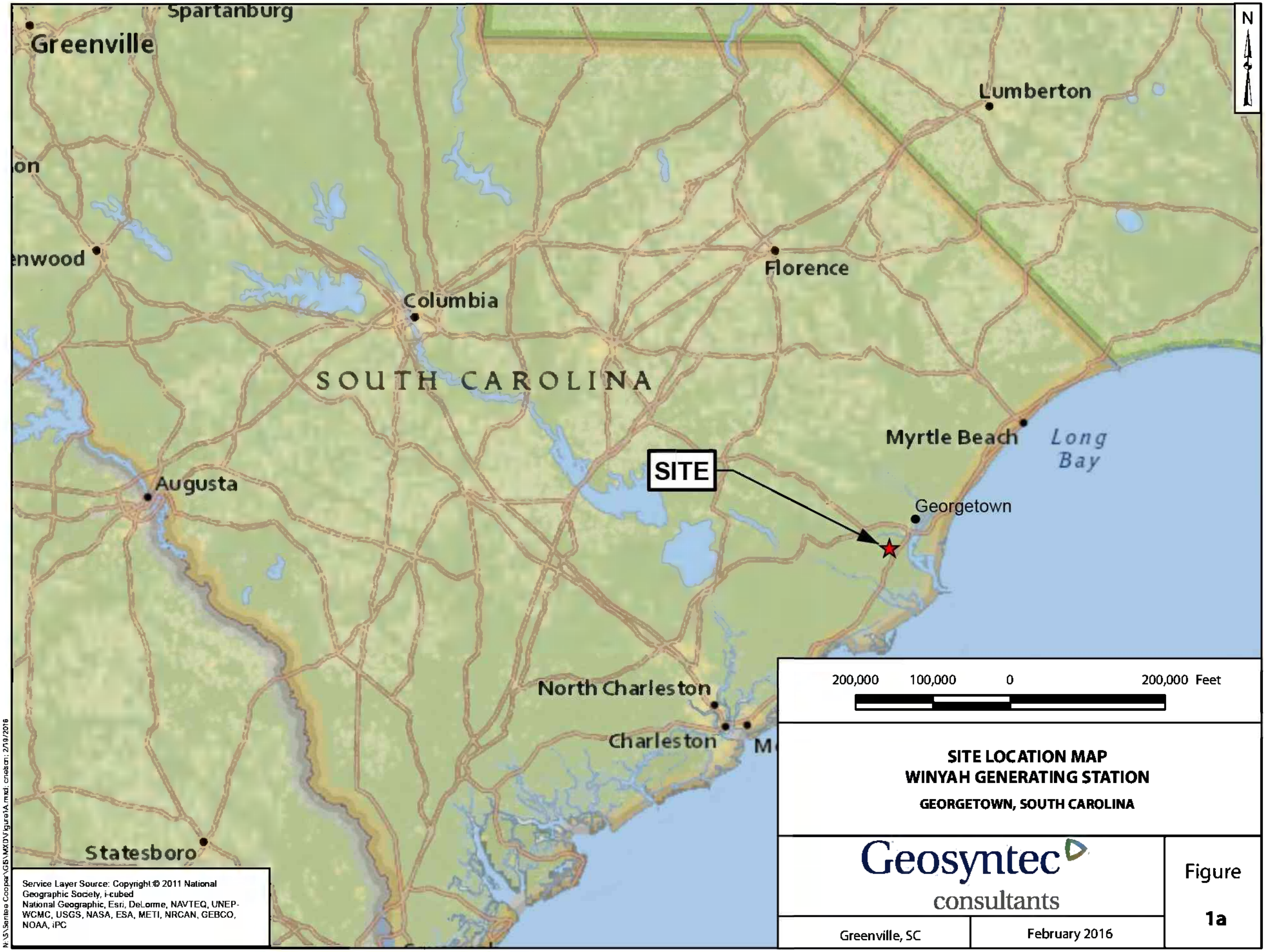
Table 3. Summary of Calculated Safety Factors

Factor of Safety Case	**Target FS**	**Cross Section A**	**Cross Section B**	**Cross Section C**	**Cross Section D**	**Cross Section E**
Static FS- Maximum Normal Storage Pool	1.50	2.07	3.16	2.08	*1.56*[2]	2.16
Static FS- Maximum Surcharge Pool	1.40	1.92	2.88	1.89	*1.45*[2]	2.01
Seismic FS- Maximum Normal Storage Pool	1.00	*1.20* [2]	2.58	1.43	1.25	1.30
Liquefaction FS[1]	1.20	N/A	N/A	N/A	N/A	N/A

Notes:

1. The liquefaction safety factor was not evaluated since embankment soils were not found to be liquefiable.
2. The lowest computed safety factors for each analysis case was italicized and are shown on Figures 11 through 13.

FIGURES



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SITE LOCATION MAP
WINYAH GENERATING STATION

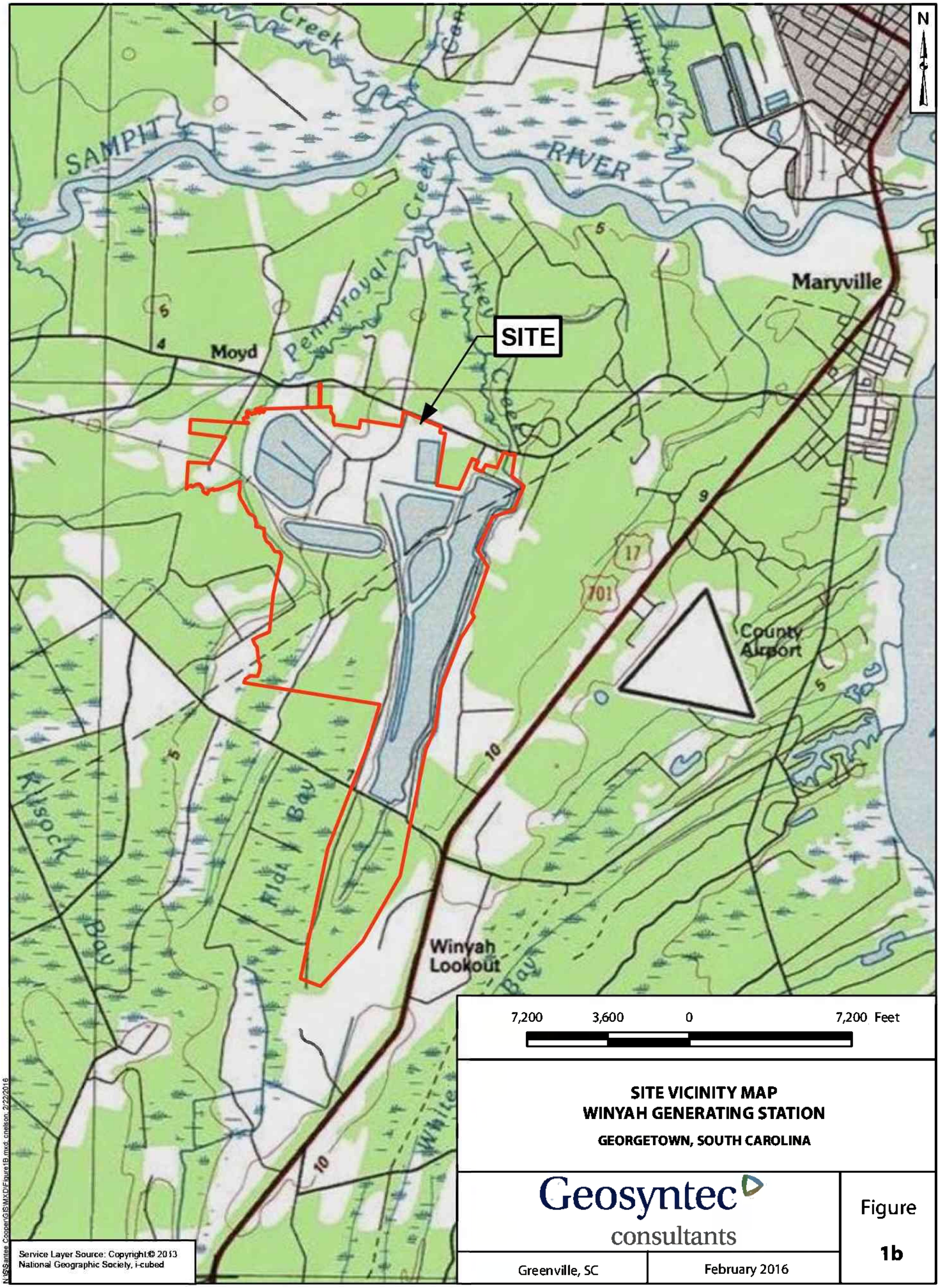
GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Greenville, SC

February 2016

Figure 1a



Service Layer Source: Copyright© 2013
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SITE VICINITY MAP
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

Geosyntec
consultants

Greenville, SC

February 2016

Figure

1b

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, and the GIS User Community

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SITE LAYOUT MAP
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

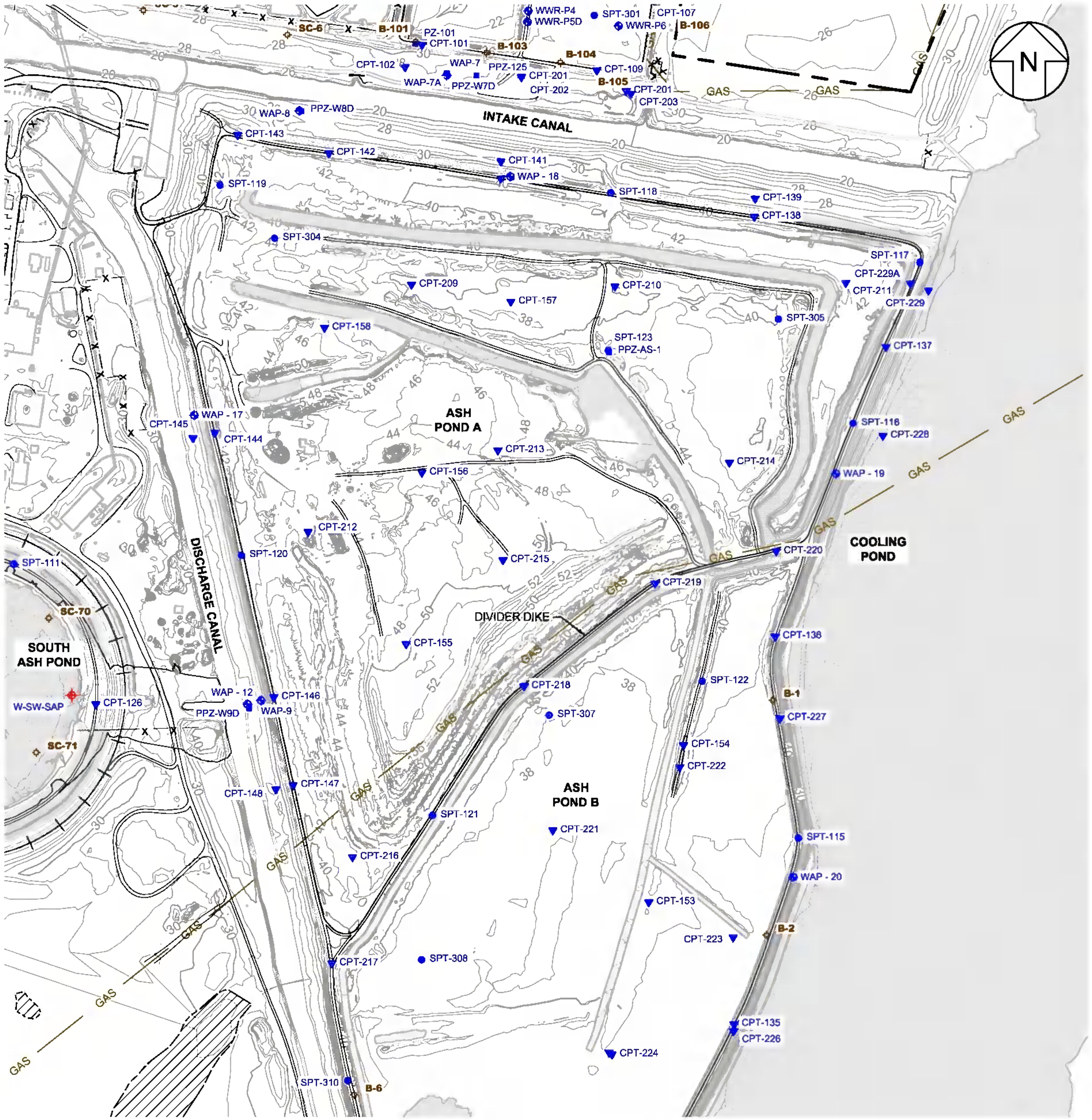
Geosyntec consultants

Greenville, SC

February 2016

Figure

2



LEGEND

——— GAS ——— GAS ———	EXISTING GAS LINE
——— 20 ———	EXISTING MAJOR GRADE CONTOUR
—+—+—+—+—+—+—	EXISTING RAILROAD
▨	EXISTING PONDED WATER
⌖ W-SW-SAP	EXISTING STAFF GAUGE
▼ CPT-101	GEOSYNTEC CONE PENETRATION TEST
● SPT-111	GEOSYNTEC SOIL BORING
⌖ B-1	HISTORICAL BORING
◆ WAP-7, WWR-P4	MONITORING WELL
■ PPZ-125, PPZ-AS-1, PZ-101	PIEZOMETER

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
3. THE POSITION OF UNDERGROUND UTILITIES SHOWN ON THIS DRAWING IS BASED UPON THE LOCATION OF SURFACE APPURTENANCES AND/OR SURFACE MARKINGS AND SHOULD BE CONSIDERED APPROXIMATE.

0 400' 800'

SCALE IN FEET

ASH POND A
BORING LOCATION MAP

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PROJECT NO: GSC5242

OCTOBER 2016

FIGURE
3

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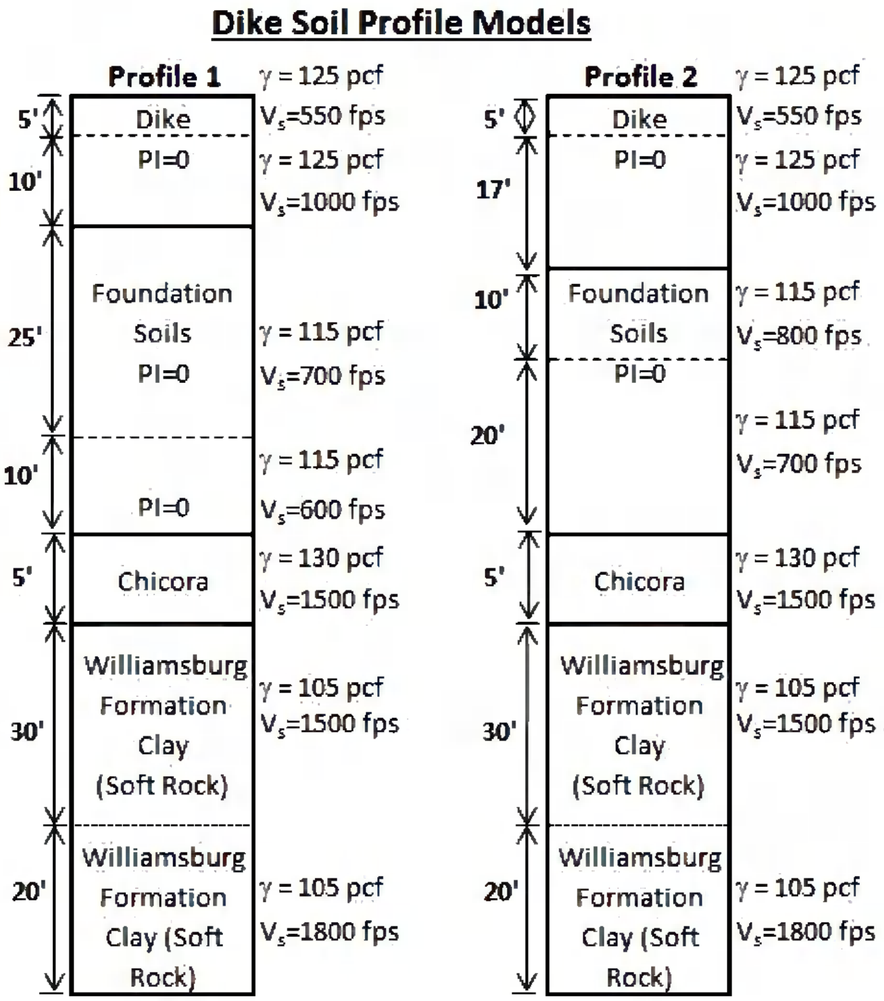
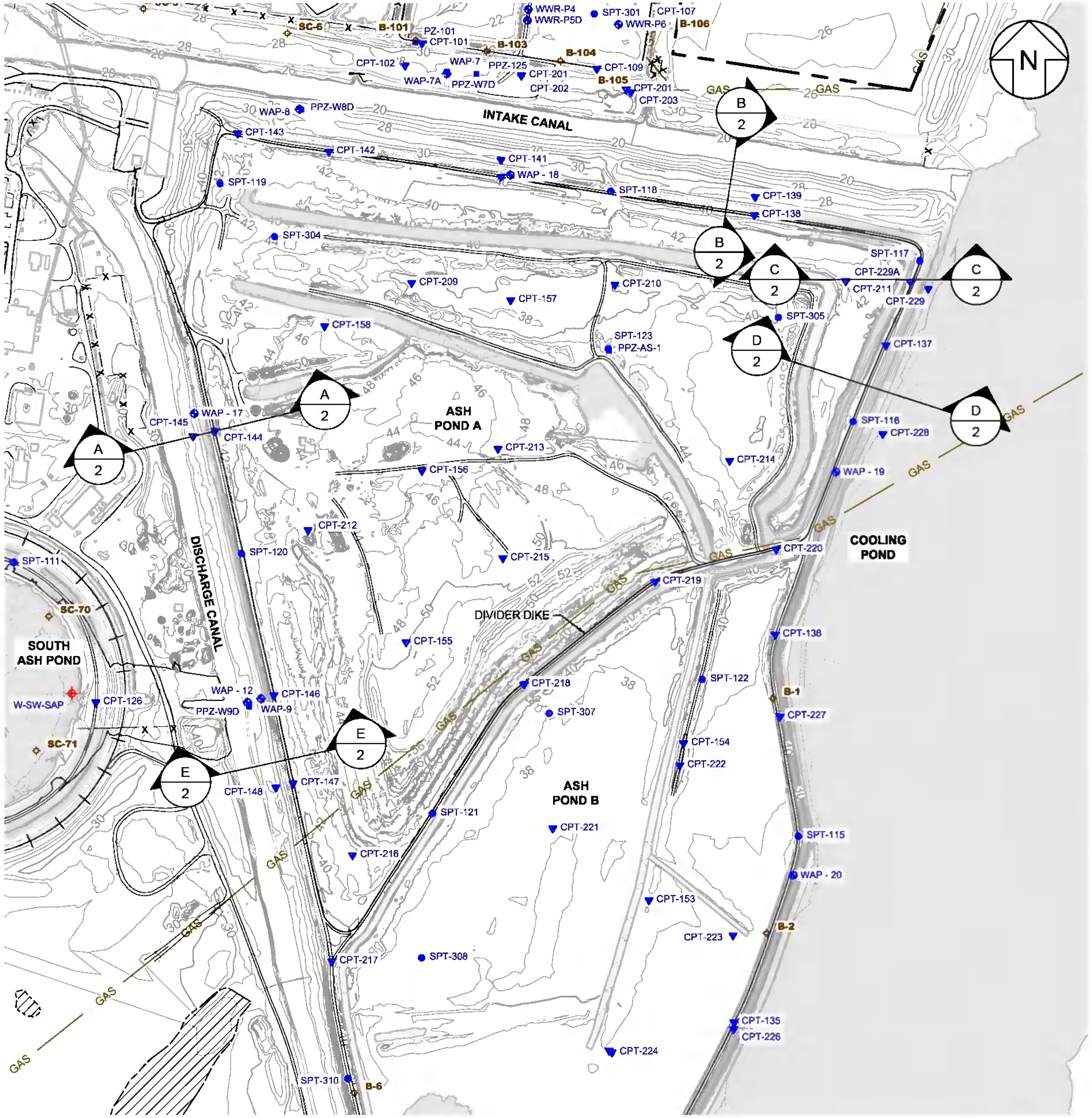


Figure 4. Representative Subsurface Profiles for Site Response Analysis



LEGEND

GAS —— GAS	EXISTING GAS LINE
—— 20 ——	EXISTING MAJOR GRADE CONTOUR
┼┼┼┼┼	EXISTING RAILROAD
▨	EXISTING PONDED WATER
⌖ W-SW-SAP	EXISTING STAFF GAUGE
▼ CPT-101	GEOSYNTEC CONE PENETRATION TEST
● SPT-111	GEOSYNTEC SOIL BORING
⌖ B-1	HISTORICAL BORING
⊕ WAP-7, WWR-P4	MONITORING WELL
■ PPZ-125, PPZ-AS-1, PZ-101	PIEZOMETER

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
3. THE POSITION OF UNDERGROUND UTILITIES SHOWN ON THIS DRAWING IS BASED UPON THE LOCATION OF SURFACE APPURTENANCES AND/OR SURFACE MARKINGS AND SHOULD BE CONSIDERED APPROXIMATE.

0 400' 800'

SCALE IN FEET

ASH POND A
BORING LOCATION MAP

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FIGURE
5

PROJECT NO: GSC5242
OCTOBER 2016

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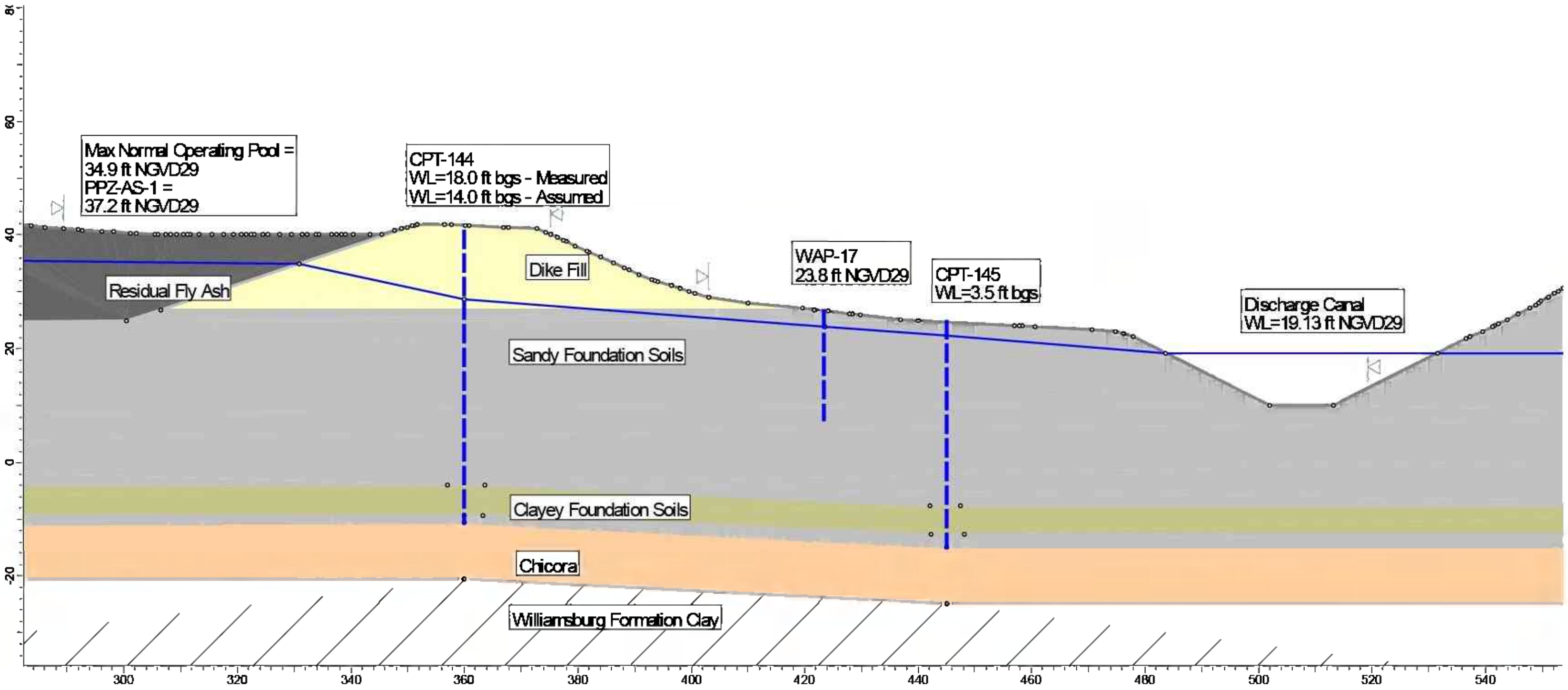


Figure 6. Cross Section A Geometry during Maximum Normal Storage Pool Conditions

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at temporary PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-144 was interpreted as 18 ft bgs but was conservatively assumed as 14.0 ft bgs for these analyses. The water level at WAP-17 (dike toe) was measured as 23.8 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the residual fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.

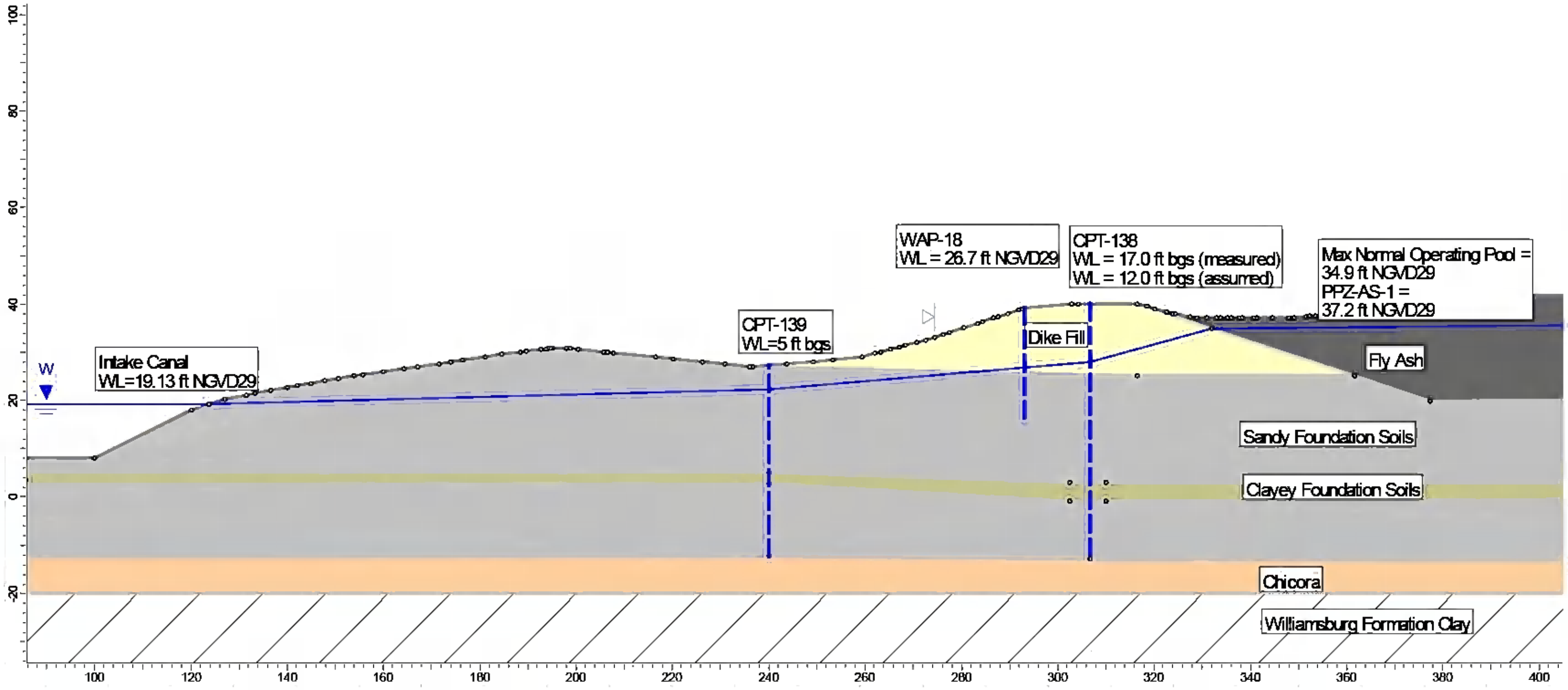


Figure 7. Cross Section B Geometry during Maximum Normal Storage Pool Conditions

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at temporary PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-138 was interpreted as 17.0 ft bgs but was conservatively assumed as 12.0 ft bgs for these analyses. The water level at WAP-18 (dike crest) was measured as 26.7 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the residual fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.

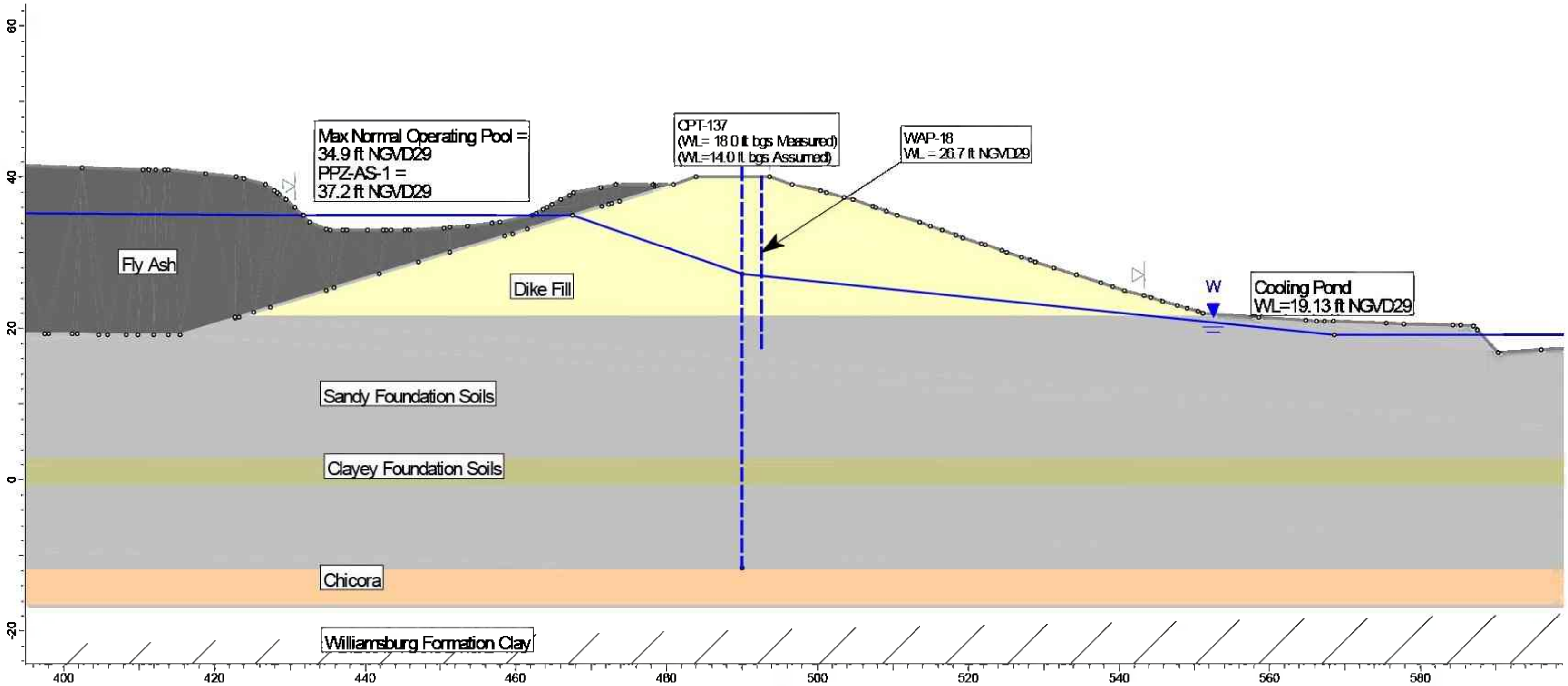


Figure 8. Cross Section C Geometry during Maximum Normal Storage Pool Conditions

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on temporary piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-137 was interpreted as 18.0 ft bgs but was conservatively assumed as 14.0 ft bgs for these analyses. The water level at WAP-18 (dike crest) was measured as 26.7 ft NGVD29 on 20 June 2016 (Attachment 5). WAP-18 selected over WAP-17 due to higher water level and cross section is in between monitoring well locations. The phreatic surface within the residual fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

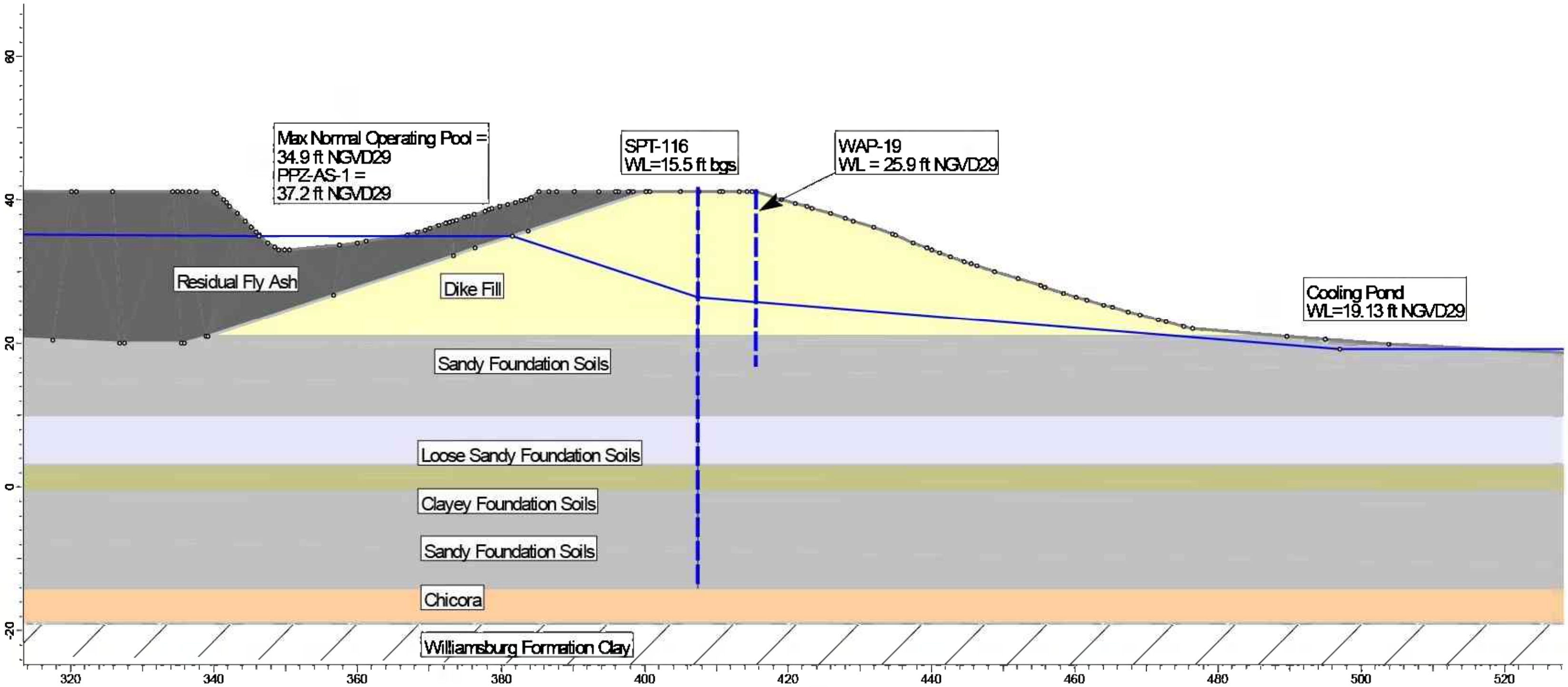


Figure 9. Cross Section D Geometry during Maximum Normal Storage Pool Conditions

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on temporary piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. The water level at WAP-19 (dike crest) was measured as 25.9 ft NGVD29 on 20 June 2016 (Attachment 5).
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

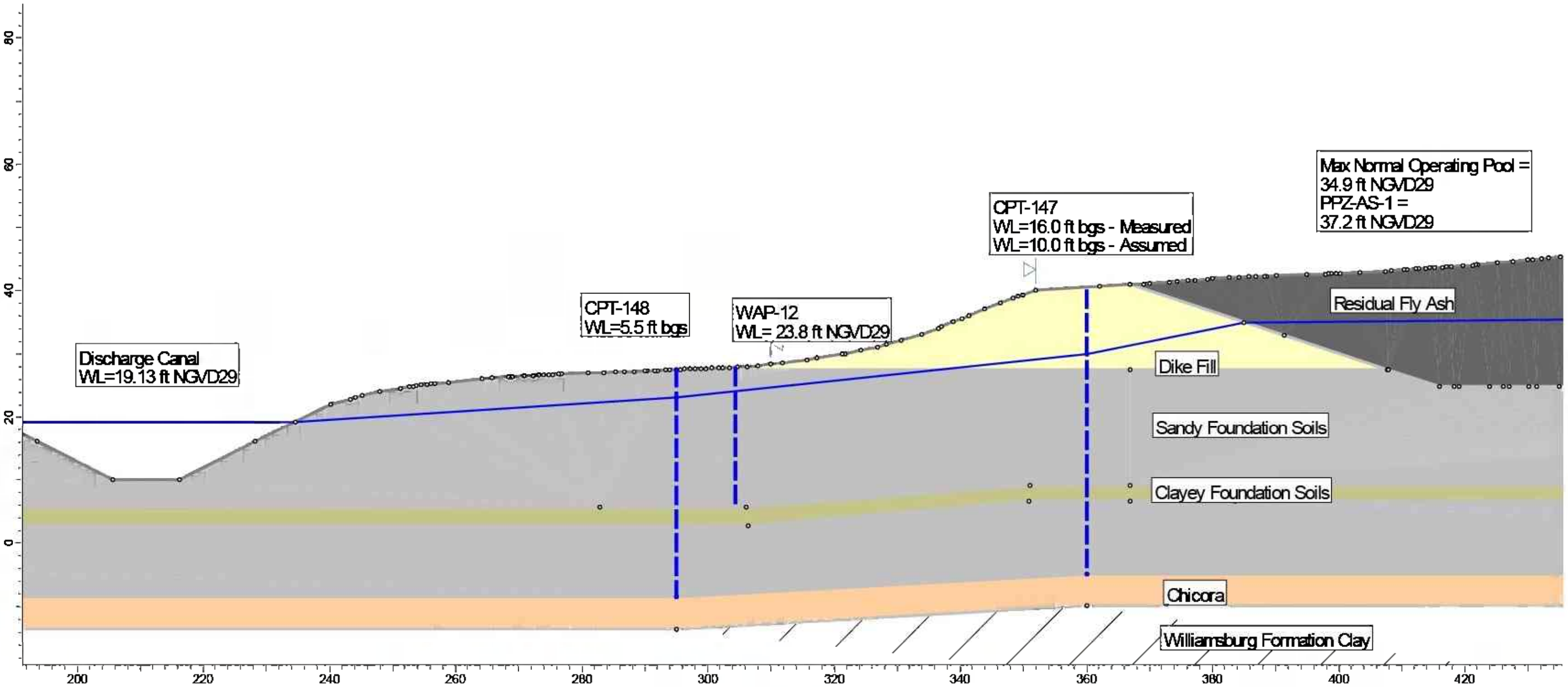
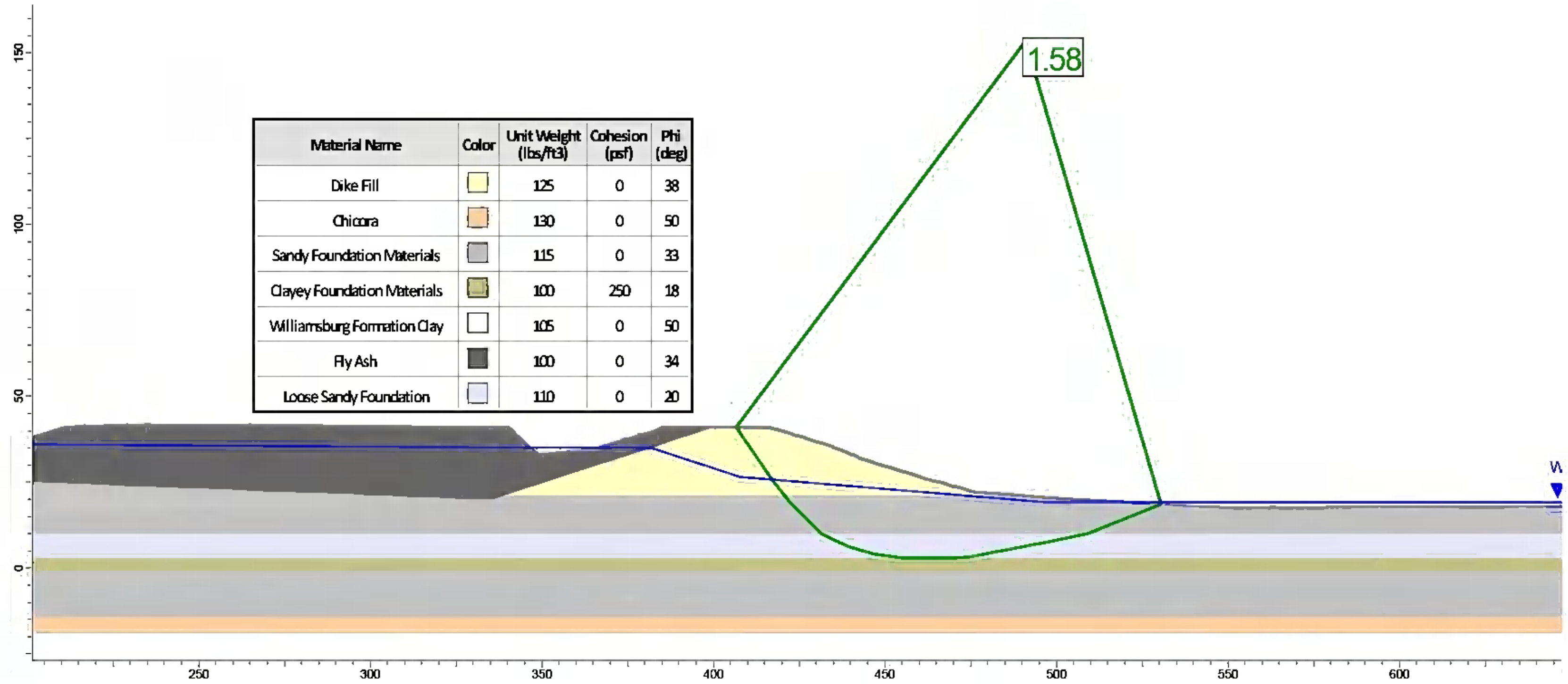


Figure 10. Cross Section E Geometry during Maximum Normal Storage Pool Conditions

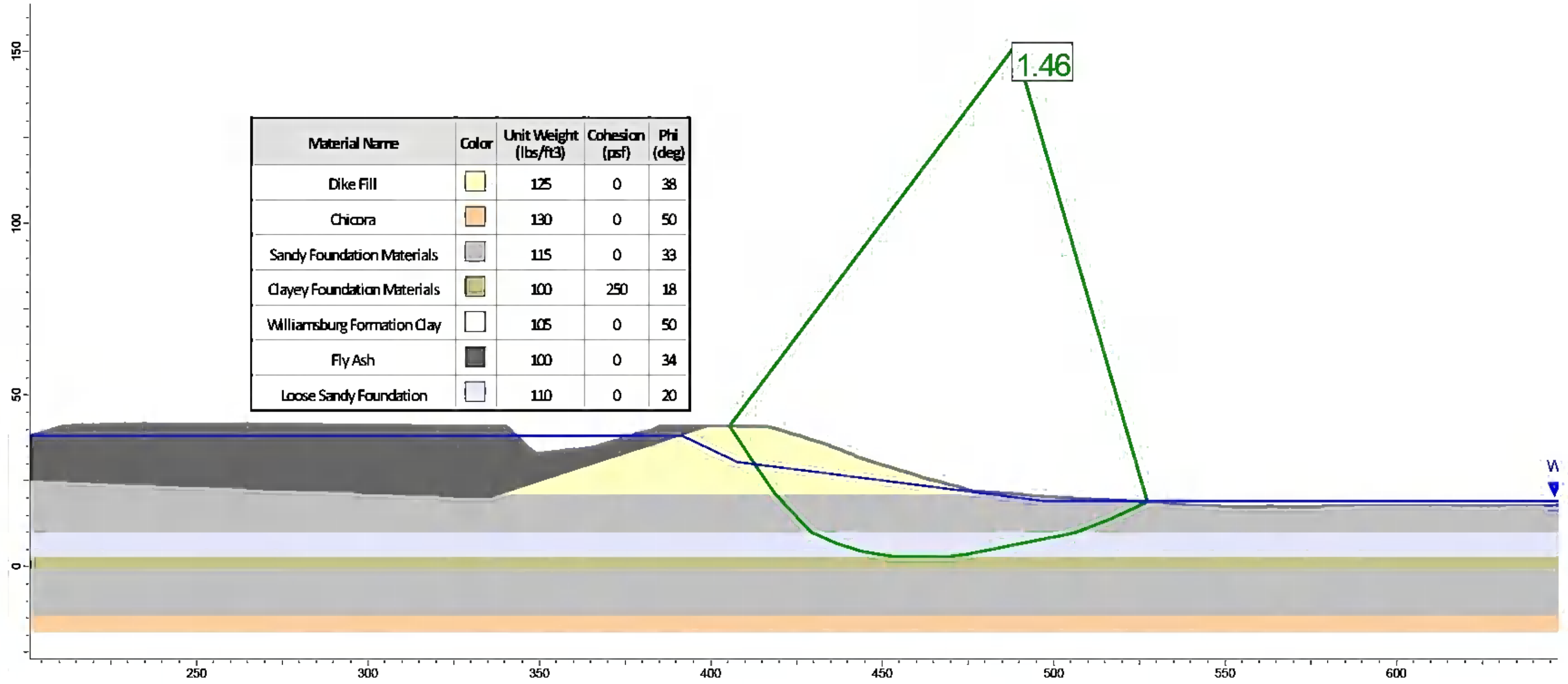
Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at temporary PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-147 was interpreted as 16.0 ft bgs but was conservatively assumed as 10.0 ft bgs for these analyses. The water level at WAP-12 (dike toe) was measured as 23.8 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.



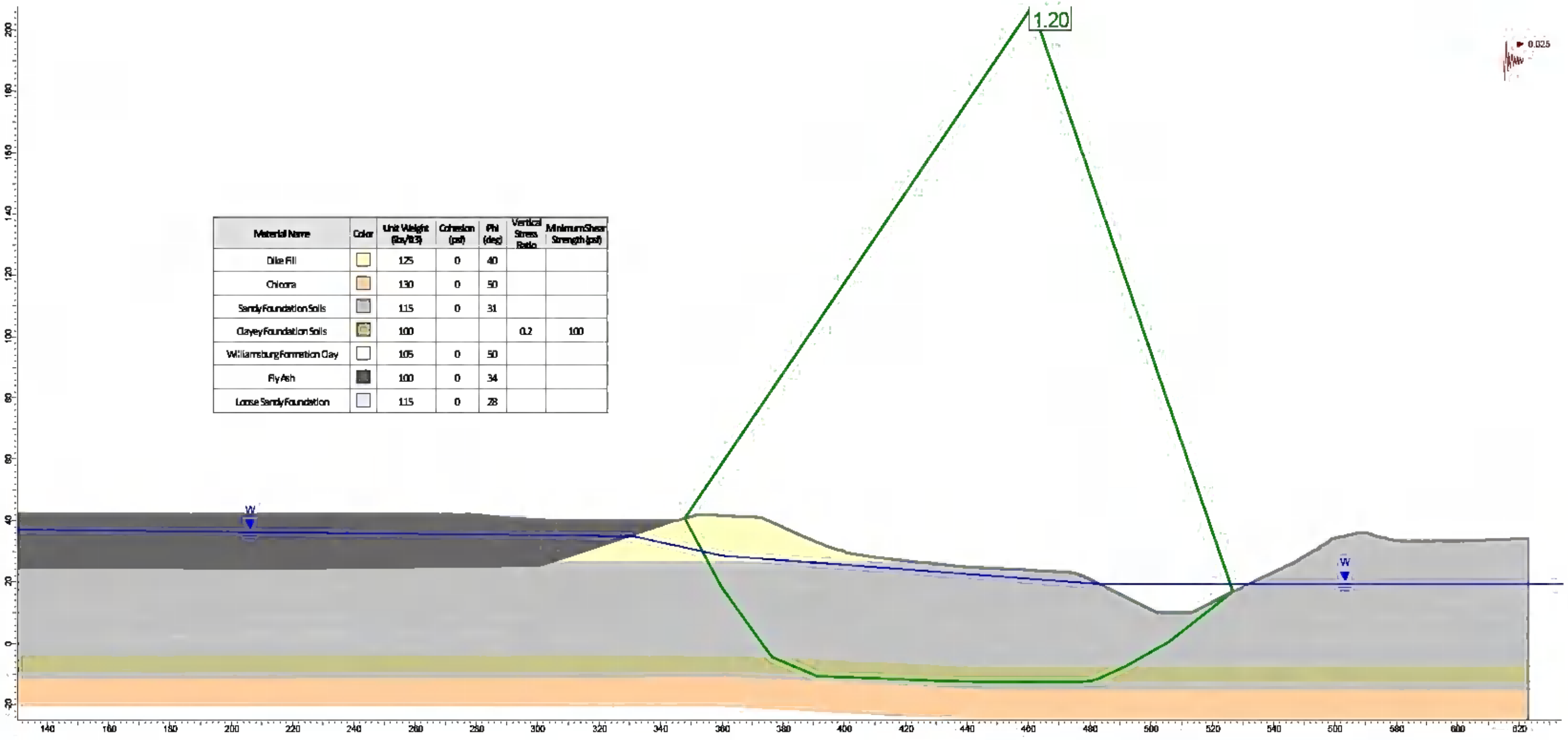
Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	🟨	125	0	38
Chicora	🟧	130	0	50
Sandy Foundation Materials	▨	115	0	33
Clayey Foundation Materials	🟩	100	250	18
Williamsburg Formation Clay	□	105	0	50
Fly Ash	■	100	0	34
Loose Sandy Foundation	▢	110	0	20

Figure 11. Critical FS for Cross Section D: Static FS - Maximum Normal Storage Pool



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	🟨	125	0	38
Chicora	🟧	130	0	50
Sandy Foundation Materials	▨	115	0	33
Clayey Foundation Materials	🟩	100	250	18
Williamsburg Formation Clay	⬜	105	0	50
Fly Ash	⬛	100	0	34
Loose Sandy Foundation	⬜	110	0	20

Figure 12. Critical FS for Cross Section D: Static FS - Maximum Surcharge Pool



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Dike Fill	🟨	125	0	40		
Chicora	🟧	130	0	50		
Sandy Foundation Soils	⬜	115	0	31		
Clayey Foundation Soils	🟩	100			0.2	100
Williamsburg Formation Clay	⬜	105	0	50		
Fly Ash	⬛	100	0	34		
Loose Sandy Foundation	⬜	115	0	28		

Figure 13. Critical FS for Cross Section A: Seismic FS - Maximum Normal Storage Pool

ATTACHMENT 1

Hydrologic and Hydraulic (H&H) Analysis

Geosyntec
consultants

COMPUTATION COVER SHEET

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242**
Task No. **01**

Title of Computations **Hydrologic and Hydraulic Analysis: Ash Pond A**

Computations by:
Signature Sarah M. Herr
2/10/16
Date
Printed Name Sarah Herr
Title Senior Staff Engineer

Assumptions and Procedures Checked by:
(senior reviewer)
Signature Brianna L. Wallace
10/11/16
Date
Printed Name Brianna Wallace
Title Senior Engineer

Computations, Assumptions, and Procedures Checked by:
(peer reviewer)
Signature
10/11/16
Date
Printed Name Hari Parthasarathy
Title Senior Staff Engineer

Computations backchecked by:
(originator)
Signature Sarah M. Herr
10/11/16
Date
Printed Name Sarah Herr
Title Senior Staff Engineer

Approved by:
(pm or designate)
Signature Brianna L. Wallace
10/11/16
Date
Printed Name Brianna Wallace
Title Senior Engineer

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	**S. Herr**	Date:	**10/11/16**	Reviewed by:	**B. Wallace**	Date:	**10/11/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GSC5242**	Task No.:	**01**

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Written by:	**S. Herr**	Date:	**10/11/16**	Reviewed by:	**B. Wallace**	Date:	**10/11/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GSC5242**	Task No.:	**01**

PURPOSE AND BACKGROUND

Winyah Generating Station (WGS or the Site) is a coal-fired, electric generating facility located in Georgetown County, South Carolina. The Site is located between Pennyroyal and Turkey Creeks, tributaries to the Sampit River, and is approximately four miles southwest of Georgetown.

The purpose of this computation package is to evaluate the hydraulic capacity of Ash Pond A to support spillway capacity assessment requirements, static factor of safety analyses, and hazard rankings required by the United States Environmental Protection Agency's (USEPA's) Coal Combustion Residual (CCR) Rule. Ash Pond A is regulated by the CCR Rule as an existing CCR surface impoundment. Under the CCR Rule, a low hazard ranking classification is associated with the 100 year (yr) precipitation event. Since Ash Pond A is a low hazard surface impoundment, the 100 yr storm frequency is analyzed herein.

Ash Pond A, encompassing approximately 90 acres (ac), is located east of the power block. Ash Pond A is bounded by the intake canal to the north, the Cooling Pond to the east, Ash Pond B to the south, and the Discharge Canal to the west. Ash Pond A is separated from Ash Pond B by a divider dike, which traverses from west to east from the Discharge Canal to the Cooling Pond. Ash Pond A is bounded by perimeter dikes ranging from 20.0 feet (ft) to 24.5 ft high on the east to 12.0 ft to 15.0 ft high on the north (Thomas and Hutton, 2012). The minimum crest elevation of the Ash Pond A perimeter dikes is 38.8 ft National Geodetic Vertical Datum of 1929 (NGVD 29) (Thomas and Hutton, 2012). A Site Map including the surface impoundments and hydraulic features associated with Ash Pond A is provided in **Figure 1**.

Ash Pond A currently receives fly ash sluice, bottom ash sluice and boiler slag, and low volume wastewater from the existing coal-fired electric generating units, as well as contact stormwater from the Unit 2 Slurry Pond. Ash Pond A does not have an outfall structure but routes water southward through rim ditches and culverts to Ash Pond B. Ash Ponds A and B are hydraulically connected through a 30 inch (in.) diameter corrugated metal pipe (CMP), a 48 in. diameter smooth steel pipe, and a 42 in. diameter smooth steel pipe (Thomas and Hutton, 2016; Thomas and Hutton, 2012).

METHODOLOGY

Stormwater runoff volumes and associated discharges to Ash Ponds A and B were modeled using *HydroCAD Version 10.0* software (HydroCAD, 2011). *HydroCAD* utilizes frequency-based precipitation events, in conjunction with watershed properties, to calculate peak runoff by several accepted methods. The Soil Conservation System (SCS) Technical Release 20 (TR-20) method was applied in *HydroCAD* to calculate stormwater runoff volumes (SCS, 1982).

The following parameters and assumptions were selected for calculating stormwater runoff volumes for Ash Ponds A and B.

Rainfall

The 72 hour (hr) duration precipitation event was used in this analysis. The rainfall depth corresponding to the 72 hr duration precipitation event for the 100 yr frequency return period for the Site is 12.8 in. (NOAA, 2006). The design storm hyetograph was developed using SCS Type III rainfall distribution and was directly input into the *HydroCAD* model.

Drainage Areas and Curve Numbers

The contributing watershed areas for Ash Ponds A and B are 90.6 ac and 65.7 ac, respectively. These areas were delineated using the dike crests to correspond to the ponds' direct drainage areas. Each pond was assigned a curve number (CN) based on guidance provided in Technical Release 55 (TR-55) (SCS, 1986) representing the type of ground cover in that area. Ash Ponds A and B were assumed to be 90% CCR and 10% water (Weighted CN = 87) (Santee Cooper, 2012a). The contributing watershed areas and CNs are summarized in **Table 1** and were directly input to the *HydroCAD* model.

Time of Concentration and Open Channel Flow Calculations

The time of concentration represents the time required for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. The flow path from the most remote point within Ash Pond A is characterized by sheet flow and channel flow (shown in **Figure 2**).

HydroCAD applied the Overton and Meadows formulation to calculate travel time for sheet flow for distances less than 300 ft (NRCS, 2010):

$$T_t = \frac{0.007(n\mathrm{L})^{0.8}}{P_{2-24}^{0.5} S^{0.4}}$$

where:

T_t = travel time for over land sheet flow (hr);

n = Manning's roughness coefficient for sheet flow (--);

L = flow length (ft);

$P_{2\text{-}24}$ = 2 yr, 24 hr rainfall (in.); and

S = slope of hydraulic grade line (or land slope) (feet per feet [ft/ft]).

A Manning's roughness coefficient of 0.020 was used to represent sheet flow in Ash Pond A. The rainfall depth for the 2 yr, 24 hr frequency storm event is 4.38 in. (NOAA, 2006). The parameters used to model sheet flow within Ash Pond A are shown in **Table 2**.

Open channel flow travel time was calculated as:

$$T_t = \frac{L}{V}$$

where:

T_t = travel time (seconds [s]);

L = flow length (ft); and

V = average velocity (feet per second [ft/s]).

The open channel flow velocity was calculated using Manning's equation. The average velocity was computed assuming bank-full elevation as:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

where: V = average velocity (ft/s);

n = Manning's roughness coefficient (--);

R = hydraulic radius (ft); and

S = slope of hydraulic grade line (or longitudinal channel slope for normal flow conditions) (ft/ft).

A Manning's roughness coefficient of 0.020 was used to represent open channel flow in Ash Pond A. Channel dimensions were estimated using topographic data, and these dimensions are summarized in **Table 3** (Thomas and Hutton, 2012). The hydraulic radius was computed as:

$$R = \frac{A}{P_w}$$

where: R = hydraulic radius (ft);

A = cross sectional flow area (square feet [sq ft]); and

P_w = wetted perimeter (ft).

The cross sectional flow area was calculated by:

$$A = (B + DZ)D$$

where: A = cross sectional flow area (sq ft);

B = bottom width of the channel (ft);

D = depth of the channel (ft); and

Z = side slope of the channel (horizontal run divided by vertical rise) (ft/ft).

The wetted perimeter was calculated by:

$$P_w = B + 2D\sqrt{1+Z^2}$$

where: P_w = wetted perimeter (ft);

B = bottom width of the channel (ft);

D = depth of the channel (ft); and

Z = side slope of the channel (horizontal run divided by vertical rise (ft/ft).

The parameters used to describe open channel flow in Ash Pond A are presented in **Table 2**. The computed times of concentration for Ash Pond A are summarized in **Table 4**.

Flow within Ash Pond B is characterized entirely as open channel flow (shown in **Figure 3**). Open channel flow within Ash Pond B was characterized using the method previously described for Ash Pond A. The flow velocity was calculated using Manning's equation. A Manning's roughness coefficient of 0.020 was used to represent open channel flow in Ash Pond B. Channel dimensions were estimated using topographic data, and these dimensions are summarized in **Table 3** (Thomas and Hutton, 2012). The parameters used to describe flow within Ash Pond B are presented in **Table 2**. The resulting time of concentration is presented in **Table 4**.

Inflows

In the *HydroCAD* model, stormwater inflows associated with Ash Ponds A and B are represented by Sub-Catchments 1S and 2S, respectively. Ponds 3P and 4P represent Ash Ponds A and B, respectively. In addition to direct stormwater inflow, Ash Pond A receives contact stormwater from the Unit 2 Slurry Pond. As shown in **Figure 4**, the Unit 2 Slurry Pond is equipped with a 6JSVE Thompson pump (Thompson Pump, 2016). The maximum pump capacity, 2,600 gallons per minute (gpm) (5.79 cubic feet per second [cfs]), is utilized in the *HydroCAD* model since the design operating point is unavailable. This base flow is modeled as Node 5L in *HydroCAD* and contributes to the inflow to Pond 3P.

Ash Pond A also receives Units 1 and 2 low volume wastewater (550 gpm), Units 1 and 2 hydroveyor water and fly ash sluice (3,364 gpm), Units 1 and 2 bottom ash sluice water (725 gpm), and Units 3 and 4 bottom ash sluice water (1,460 gpm). The base flow considers process water when all four units are operational, resulting in an inflow of 6,099 gpm (13.59 cfs) (Santee Cooper, 2015). This base flow is modeled as Node 6L in *HydroCAD* and contributes to the inflow to Pond 3P. The *HydroCAD* model routing diagram is provided in **Appendix A**.

Storage Capacities

The available stormwater storage volume of Ash Pond A between elevations 34.0 ft and 38.8 ft NGVD 29 was calculated by developing an area-volume curve based on topographic data (Thomas and Hutton, 2012; Thomas and Hutton, 2016). The minimum crest elevation of the Ash Pond A perimeter dikes is 38.8 ft NGVD 29. The surface area of each contour was measured and tabulated at each elevation. The available surface water volume in each 2 ft depth increment was calculated by averaging the surface area of the upper and lower contour and multiplying by the change in elevation between each contour. The cumulative storage volume of Ash Pond A between these elevations is 12.5 acre-feet (ac-ft). The area-volume data are presented in **Table 5**.

Similarly, the available stormwater storage volume of Ash Pond B between elevations 34.0 ft and 39.7 ft NGVD 29 was calculated by developing an area-volume curve based on topographic and bathymetric data (Thomas and Hutton, 2012; Thomas and Hutton, 2016). A bathymetric survey has not been completed for Ash Pond B. The average operating elevation provided by the plant from February 2011 through January 2016 is 34.1 ft (Santee Cooper, WGS, 2016). Elevation 34.1 is used as the starting water surface elevation for Pond B in the model. The minimum crest elevation of the Ash Pond B perimeter dikes is 39.7 ft NGVD 29. The surface area of each contour was measured and tabulated at each elevation. Next, the available surface water volume at each 2 ft depth increment was calculated by averaging the surface area of the upper and lower contour and multiplying by the change in elevation between each contour. The cumulative storage volume of Ash Pond B between these elevations is 220.3 ac-ft. The area-volume data are presented in **Table 5**.

Outlet Structures

The outlets from Ash Pond A to Ash Pond B are a 30 in. diameter CMP culvert with an upstream invert at 37.50 ft NGVD 29 and a downstream invert at 36.52 ft NGVD 29, a 48 in. diameter smooth steel pipe with an upstream invert at 35.49 ft NGVD 29 and a downstream invert at 35.28 ft NGVD 29, and a 42 in. diameter smooth steel pipe with an upstream invert at 36.20 ft NGVD 29 and a downstream invert at 35.70 ft NGVD 29 (Thomas and Hutton, 2016; Thomas and Hutton, 2012). These outlet pipes allow water to drain from Ash Pond A to Ash Pond B.

The operating level in Ash Pond B is maintained by a concrete riser structure with an internal length of 4 ft and an internal width of 4 ft (Santee Cooper, 2012b). The concrete riser structure has 4 ft long stop logs on a single face, and the top stop log elevation is 34.90 ft NGVD 29 (Santee Cooper, 2012b; Thomas and Hutton, 2016). A 24 in. diameter smooth interior, corrugated exterior high density polyethylene pipe culvert with a downstream invert elevation of 17.99 ft NGVD 29 conveys water from the riser structure to the Discharge Canal of the Cooling Pond (Santee Cooper, 2012b; Thomas and Hutton, 2016).

The tailwater effects associated with discharge from Ash Pond B to the Discharge Canal were modeled using a fixed water surface elevation within the Discharge Canal and Cooling Pond. This tailwater surface elevation was estimated by conservatively assuming 2.5 ft depth of water over the Cooling Pond emergency spillway during the 100 yr storm event. The top of the stop log bolted to the top of the concrete spillway of the Cooling Pond is at elevation 21.65 ft NGVD 29 (Thomas and Hutton, 2015). The water surface of the Discharge Canal and Cooling Pond was assumed to be at 24.15 ft NGVD 29 (21.65 ft NGVD 29 plus an additional 2.5 ft of water). The tailwater effects associated with the Discharge Canal and Cooling Pond were represented by Node 7L in the *HydroCAD* model.

RESULTS

As currently operated, Ash Pond A will not contain the 100 yr storm event. Ash Pond A does not have an existing emergency spillway. The construction of a spillway on the divider dike between Ash Ponds A and B is required to convey stormwater from Ash

Pond A to Ash Pond B so the ponds contain the 100 yr storm event. A proposed spillway is included in the *HydroCAD* model and report in **Appendix A**.

Ash Pond A contains the 72 hr, 100 yr storm event given the following assumption:

- Santee Cooper will construct a 100 ft wide spillway with an invert elevation of 37.0 ft NGVD 29 in the divider dike between Ash Ponds A and B. The spillway will be constructed with 10H:1V (Horizontal:Vertical) side slopes and will be located between the 48 in. diameter smooth steel pipe and the 42 in. diameter smooth steel pipe.

The spillway is currently under construction. Operational Plans are in place in case the design storm event occurs before the construction of the spillway is completed.

The resulting peak water surface elevation and storage volume for the 100 yr storm event is shown in **Table 6**. During this storm event, Ash Ponds A and B will effectively operate as a single pond as the culverts and spillway provide a hydraulic connection between the storage areas in both ponds.

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TABLES

Table 1 – Watershed Areas and Curve Numbers

Drainage Basin	Area (ac)	Weighted Curve Number (--)
Ash Pond A	88.954	87
Ash Pond B	65.693	87

Table 2 – Input Parameters Describing Sheet Flow and Open Channel Flow

Flow Path	Sheet Flow				Open Channel Flow				
	Land Slope (ft/ft)	*Manning's Roughness Coefficient (--)*	*Flow Length (ft)*	*2 Yr, 2 Hr Rainfall (in.)*	*Cross Sectional Area (sq ft)*	*Wetted Perimeter (ft)*	*Channel Slope (ft/ft)*	*Manning's Roughness Coefficient (--)*	*Flow Length (ft)*
Ash Pond A									
Sheet	0.0663	0.020	60	4.38	--	--	--	--	--
Channel	--	--	--	--	147	59.0	0.0025	0.020	3,100
Ash Pond B									
Channel	--	--	--	--	78	33.4	0.0025	0.020	3,650

Table 3 – Open Channel Dimensions

Flow Path	Channel Configuration	Side Slope Ratio (H:V) (ft:ft)	Bottom Width of the Channel (ft)	Depth of the Channel (ft)
Ash Pond A				
Channel	Trapezoidal	3:1	40	3
Ash Pond B				
Channel	Trapezoidal	2:1	20	3

Table 4 – Times of Concentration

Flow Path	Time of Concentration (minutes [min])
Ash Pond A	
Sheet	0.7
Channel	7.6
Ash Pond B	
Channel	9.3

Table 5 – Stage Storage Table (Thomas and Hutton, 2012; Thomas and Hutton, 2016)

Ash Pond A				Ash Pond B			
Elevation (NGVD 29) (ft)	*Area (ac)*	*Volume (ac-ft)*	*Cumulative Volume (ac-ft)*	*Elevation (NGVD 29) (ft)*	*Area (ac)*	*Volume (ac-ft)*	*Cumulative Volume (ac-ft)*
38.8	8.529	5.3	12.5	39.68	62.066	101.6	220.3
38	4.726	5.7	7.2	38	58.863	88.8	118.7
36	1.014	1.5	1.5	36	29.915	29.9	29.9
34	0.460	0.0	0.0	34	0.006	0.0	0.0

Table 6 – Peak Elevation and Volume

Storm Event	**Ash Pond A**		
	Elevation (NGVD 29) (ft)	*Volume (ac-ft)*	*Time (hr)*
100 Yr, 72 Hr	38.13	7.875	36.22

FIGURES

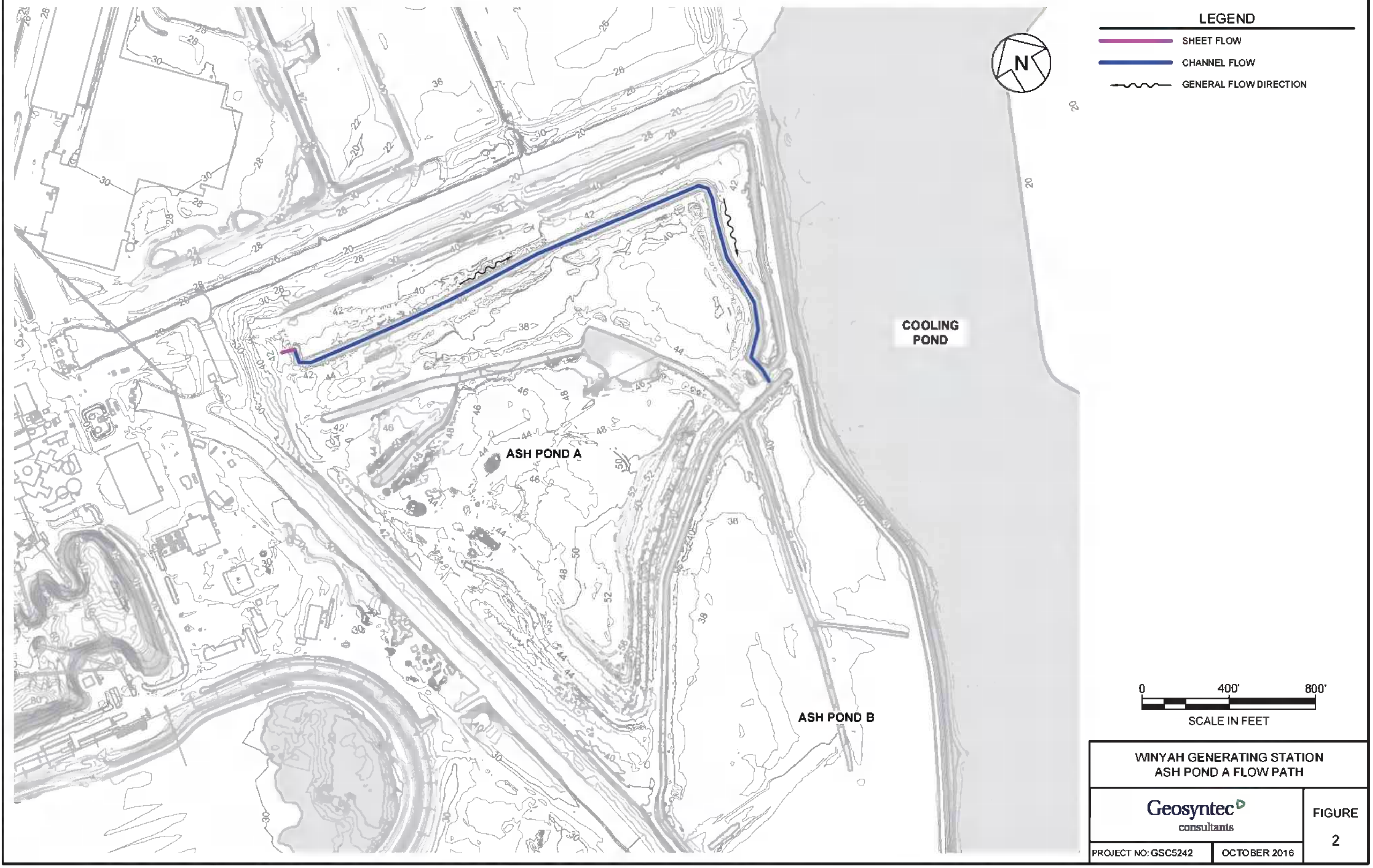
WINYAH GENERATING STATION
SITE MAP

Geosyntec consultants

FIGURE
1

PROJECT NO: GSC5242

OCTOBER 2016



WINYAH GENERATING STATION
ASH POND A FLOW PATH

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FIGURE 2

PROJECT NO: GSC5242

OCTOBER 2016

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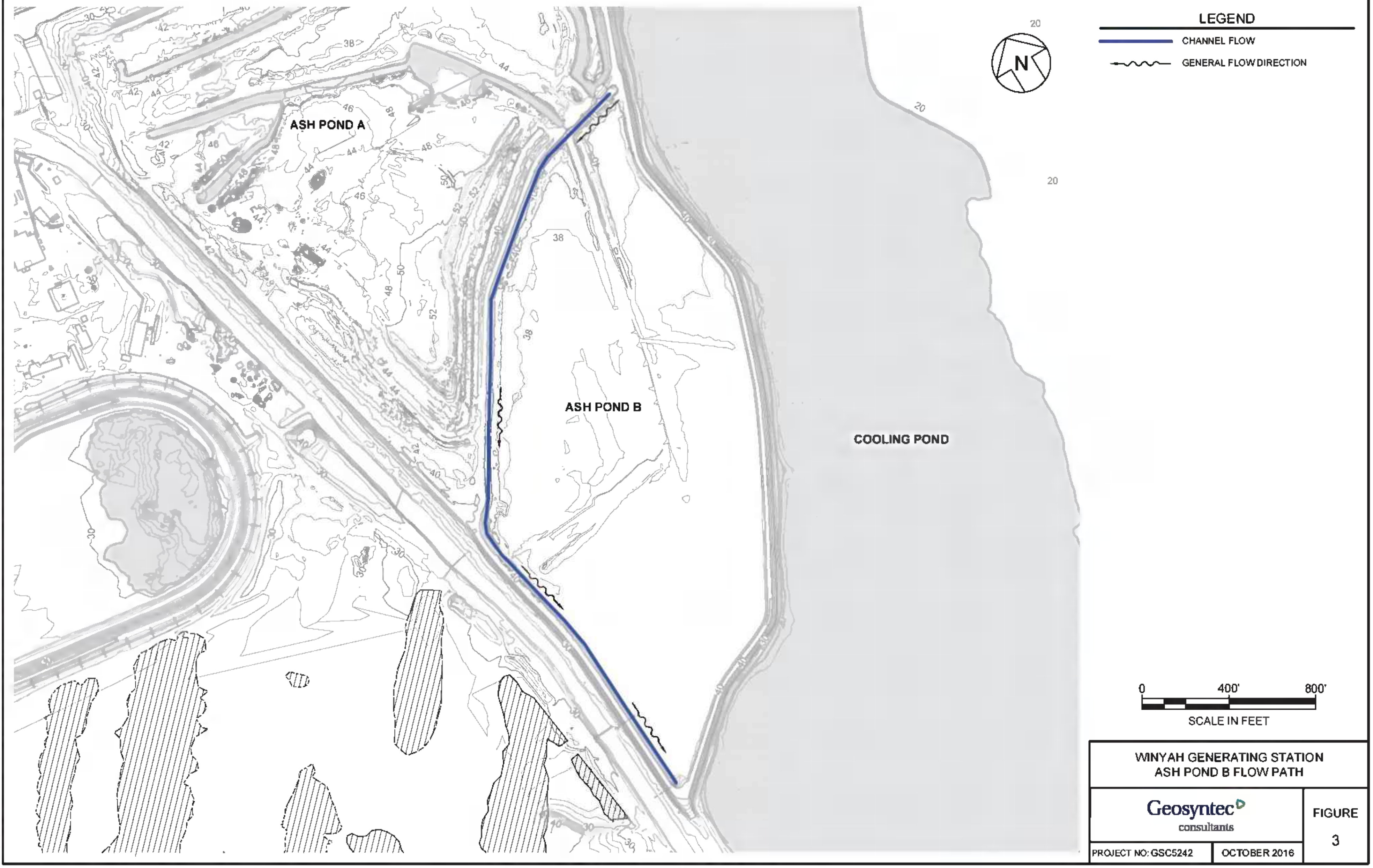

WINYAH GENERATING STATION
ASH POND B FLOW PATH

Geosyntec consultants

FIGURE 3

PROJECT NO: GSC5242

OCTOBER 2016

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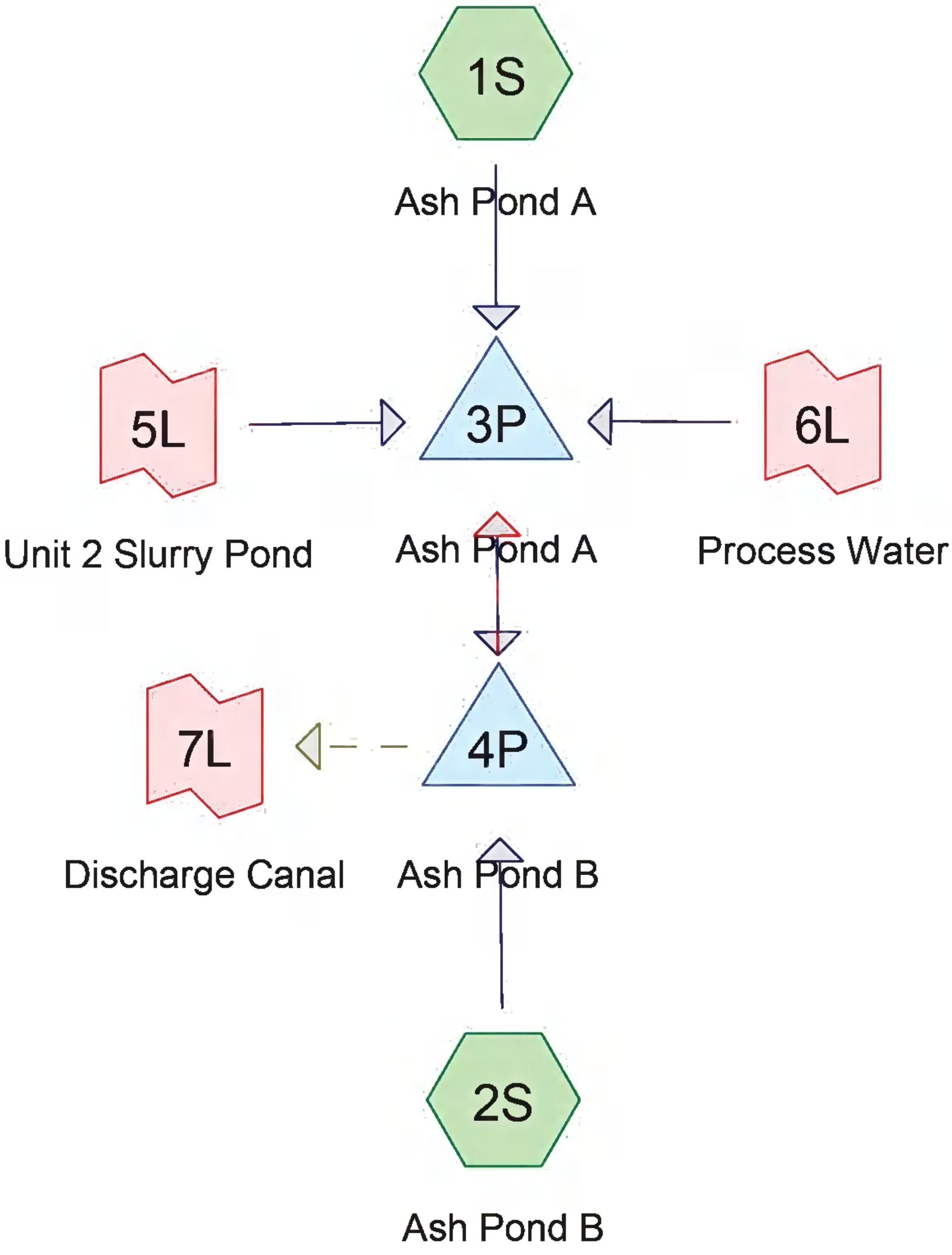
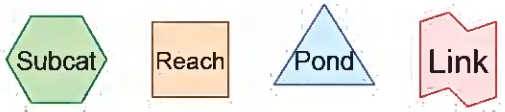
Figure 4 – Photographs of Unit 2 Slurry Pond Pump

APPENDICES

APPENDIX A





Routing Diagram for Ash Pond A B - Spillway Revision
Prepared by Geosyntec Consultants, Printed 10/12/2016
HydroCAD® 10.00-16 s/n 00939 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
154.647	87	90% Ash and 10% Water Surface (1S, 2S)
154.647	**87**	**TOTAL AREA**

```
                      Time span=0.00-600.00 hrs, dt=0.01 hrs, 60001 points
                      Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
             Reach routing by Sim-Route method  -  Pond routing by Sim-Route method

Subcatchment1S: Ash Pond A           Runoff Area=88.954 ac  0.00% Impervious  Runoff Depth=11.17"
                              Flow Length=3,160'  Tc=8.3 min  CN=87  Runoff=403.49 cfs  82.775 af

Subcatchment2S: Ash Pond B           Runoff Area=65.693 ac  0.00% Impervious  Runoff Depth=11.17"
                     Flow Length=3,650'  Slope=0.0025 '/'  Tc=9.3 min  CN=87  Runoff=294.30 cfs  61.130 af

Pond 3P: Ash Pond A                Peak Elev=38.13'  Storage=7.875 af  Inflow=422.87 cfs  1,043.751 af
        Primary=55.02 cfs  839.100 af  Secondary=320.83 cfs  206.216 af  Outflow=375.85 cfs  1,045.316 af

Pond 4P: Ash Pond B                Peak Elev=37.17'  Storage=74.828 af  Inflow=347.09 cfs  900.215 af
Primary=0.00 cfs  0.000 af  Secondary=0.00 cfs  0.000 af  Tertiary=21.76 cfs  865.546 af  Outflow=21.76 cfs  865.546 af

Link 5L: Unit 2 Slurry Pond                           Manual Hydrograph  Inflow=5.79 cfs  287.112 af
                                                                        Primary=5.79 cfs  287.107 af

Link 6L: Process Water                               Manual Hydrograph  Inflow=13.59 cfs  673.896 af
                                                                       Primary=13.59 cfs  673.884 af

Link 7L: Discharge Canal                                                  Inflow=21.76 cfs  865.531 af
                                                                       Primary=21.76 cfs  865.531 af

        Total Runoff Area = 154.647 ac   Runoff Volume = 143.905 af   Average Runoff Depth = 11.17"
                                   100.00% Pervious = 154.647 ac     0.00% Impervious = 0.000 ac
```

Summary for Subcatchment 1S: Ash Pond A

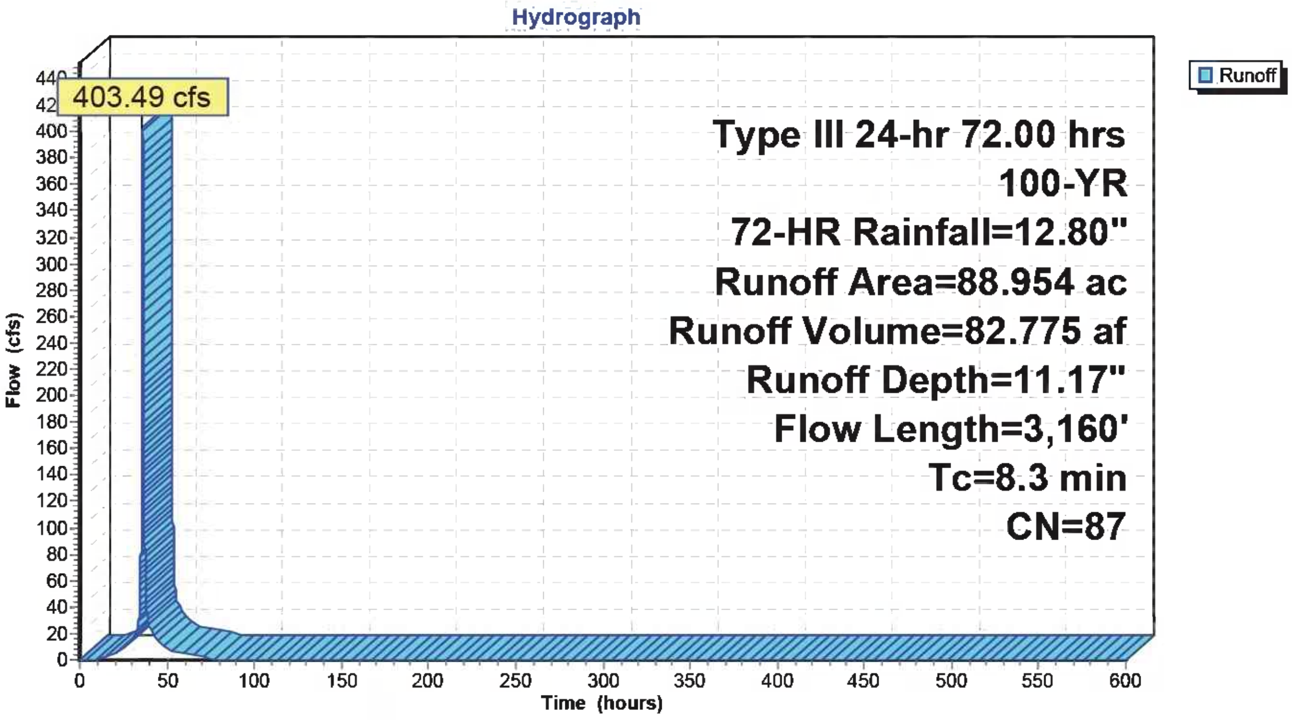
Runoff = 403.49 cfs @ 36.12 hrs, Volume= 82.775 af, Depth=11.17"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs
Type III 24-hr 72.00 hrs 100-YR, 72-HR Rainfall=12.80"

	Area (ac)	CN	Description
*	88.954	87	90% Ash and 10% Water Surface
	88.954		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.7	60	0.0663	1.45		**Sheet Flow, Sheet Flow** n= 0.020 P2= 4.38"
7.6	3,100	0.0025	6.83	1,003.66	**Channel Flow, Channel Flow** Area= 147.0 sf Perim= 59.0' r= 2.49' n= 0.020
8.3	3,160	Total			

Subcatchment 1S: Ash Pond A



Summary for Subcatchment 2S: Ash Pond B

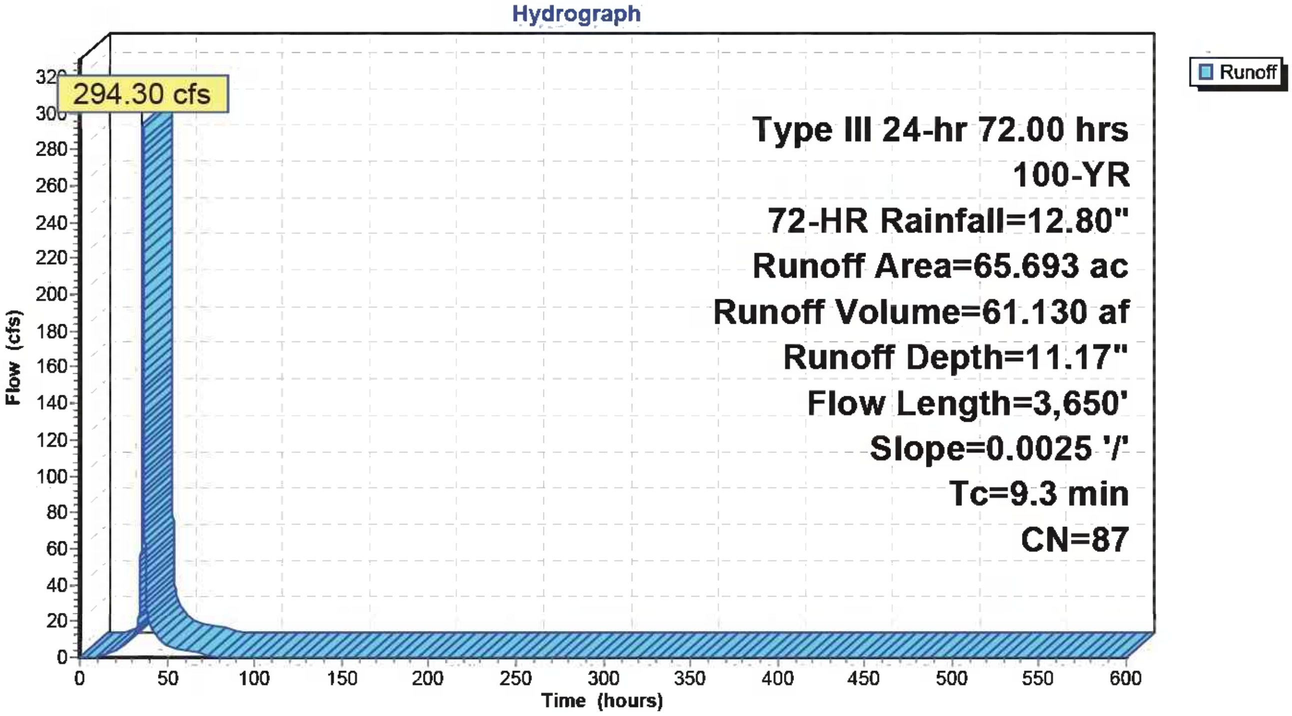
Runoff = 294.30 cfs @ 36.13 hrs, Volume= 61.130 af, Depth=11.17"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs
Type III 24-hr 72.00 hrs 100-YR, 72-HR Rainfall=12.80"

	Area (ac)	CN	Description
*	65.693	87	90% Ash and 10% Water Surface
	65.693		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.3	3,650	0.0025	6.54	510.06	**Channel Flow, Channel Flow** Area= 78.0 sf Perim= 33.4' r= 2.34' n= 0.020

Subcatchment 2S: Ash Pond B



Summary for Pond 3P: Ash Pond A

Inflow = 422.87 cfs @ 36.12 hrs, Volume= 1,043.751 af
Outflow = 375.85 cfs @ 36.22 hrs, Volume= 1,045.316 af, Atten= 11%, Lag= 6.3 min
Primary = 55.02 cfs @ 36.22 hrs, Volume= 839.100 af
Secondary = 320.83 cfs @ 36.22 hrs, Volume= 206.216 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs
Starting Elev= 37.50' Surf.Area= 3.798 ac Storage= 5.083 af
Peak Elev= 38.13' @ 36.22 hrs Surf.Area= 5.350 ac Storage= 7.875 af (2.792 af above start)

Plug-Flow detention time= 123.4 min calculated for 1,040.233 af (100% of inflow)
Center-of-Mass det. time= (not calculated: outflow precedes inflow)

Volume	Invert	Avail.Storage	Storage Description
#1	34.00'	12.516 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

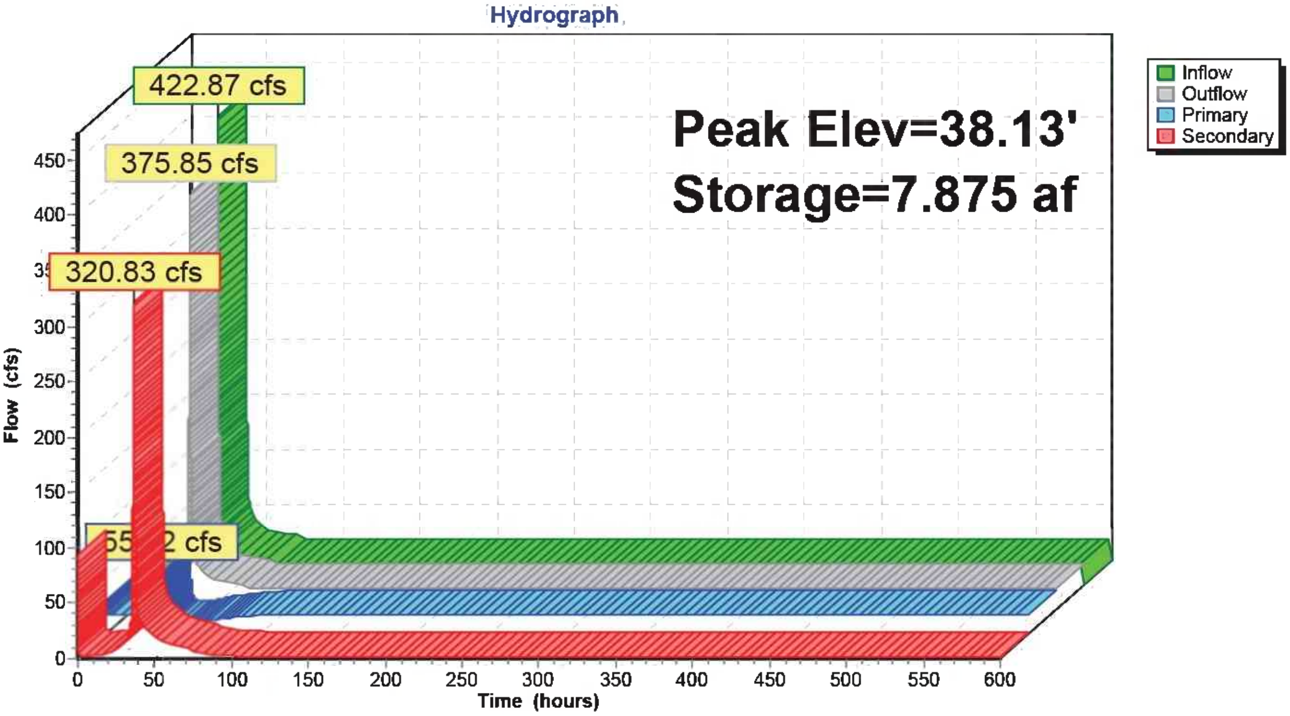
Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
34.00	0.460	0.000	0.000
36.00	1.014	1.474	1.474
38.00	4.726	5.740	7.214
38.80	8.529	5.302	12.516

Device	Routing	Invert	Outlet Devices
#1	Primary	37.50'	**30.0" Round Culvert 1** L= 40.8' CMP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 37.50' / 36.52' S= 0.0240 '/' Cc= 0.900 n= 0.025 Corrugated metal, Flow Area= 4.91 sf
#2	Primary	35.49'	**48.0" Round Culvert 2** L= 30.9' CMP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 35.49' / 35.28' S= 0.0068 '/' Cc= 0.900 n= 0.012 Steel, smooth, Flow Area= 12.57 sf
#3	Primary	36.20'	**42.0" Round Culvert 3** L= 24.6' CMP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 36.20' / 35.70' S= 0.0203 '/' Cc= 0.900 n= 0.012 Steel, smooth, Flow Area= 9.62 sf
#4	Secondary	37.00'	**100.0' long x 12.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64

Primary OutFlow Max=55.01 cfs @ 36.22 hrs HW=38.13' TW=36.80' (Dynamic Tailwater)
1=Culvert 1 (Inlet Controls 2.08 cfs @ 2.14 fps)
2=Culvert 2 (Barrel Controls 32.60 cfs @ 5.25 fps)
3=Culvert 3 (Inlet Controls 20.33 cfs @ 3.74 fps)

Secondary OutFlow Max=320.77 cfs @ 36.22 hrs HW=38.13' (Free Discharge)
4=Broad-Crested Rectangular Weir (Weir Controls 320.77 cfs @ 2.84 fps)

Pond 3P: Ash Pond A



Summary for Pond 4P: Ash Pond B

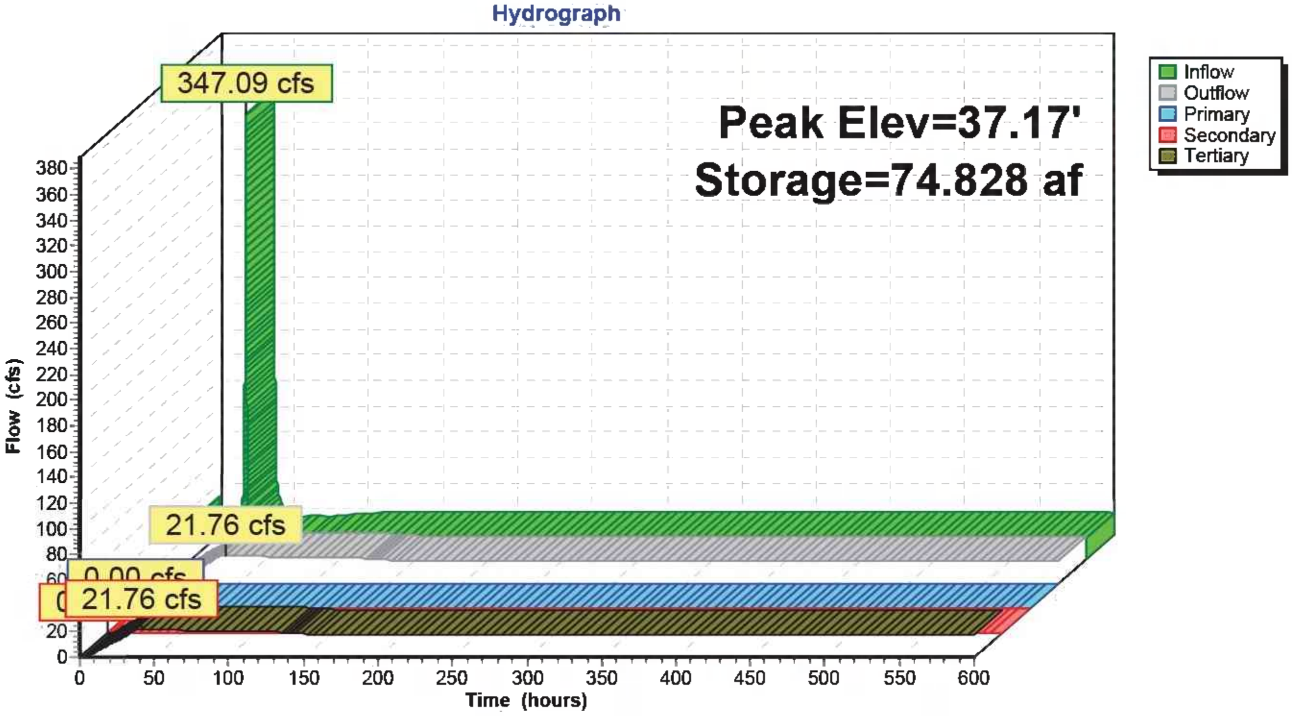
```
Inflow    =    347.09 cfs @  36.14 hrs, Volume=      900.215 af
Outflow   =     21.76 cfs @  41.98 hrs, Volume=      865.546 af, Atten= 94%, Lag= 350.4 min
Primary   =      0.00 cfs @   0.00 hrs, Volume=        0.000 af
Secondary =      0.00 cfs @   0.00 hrs, Volume=        0.000 af
Tertiary  =     21.76 cfs @  41.98 hrs, Volume=      865.546 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs
Starting Elev= 34.14'  Surf.Area= 2.100 ac  Storage= 0.147 af
Peak Elev= 37.17' @ 41.98 hrs  Surf.Area= 46.848 ac  Storage= 74.828 af  (74.681 af above start)

Plug-Flow detention time= 1,529.0 min calculated for 865.384 af (96% of inflow)
Center-of-Mass det. time= 799.9 min ( 18,019.4 - 17,219.6 )

Volume      Invert   Avail.Storage  Storage Description
   #1       34.00'     220.274 af  Custom Stage Data (Prismatic)Listed below (Recalc)

 Elevation     Surf.Area      Inc.Store      Cum.Store
    (feet)       (acres)    (acre-feet)    (acre-feet)
     34.00         0.006          0.000          0.000
     36.00        29.915         29.921         29.921
     38.00        58.860         88.775        118.696
     39.68        62.066        101.578        220.274

Device  Routing         Invert   Outlet Devices
   #1   Tertiary        31.21'   21.6" Round Culvert
                                 L= 113.3'  CPP, projecting, no headwall,  Ke= 0.900
                                 Inlet / Outlet Invert= 31.21' / 17.99'  S= 0.1167 '/'  Cc= 0.900
                                 n= 0.013  Corrugated PE, smooth interior,  Flow Area= 2.54 sf
   #2   Device 1        34.90'   4.0' long Sharp-Crested Rectangular Weir 2 End Contraction(s)
   #3   Primary         37.50'   30.0" Round Culvert 1
                                 L= 40.8'  CMP, projecting, no headwall,  Ke= 0.900
                                 Inlet / Outlet Invert= 36.52' / 37.50'  S= -0.0240 '/'  Cc= 0.900
                                 n= 0.025  Corrugated metal,  Flow Area= 4.91 sf
   #4   Primary         35.49'   48.0" Round Culvert 2
                                 L= 30.9'  CMP, projecting, no headwall,  Ke= 0.900
                                 Inlet / Outlet Invert= 35.28' / 35.49'  S= -0.0068 '/'  Cc= 0.900
                                 n= 0.012  Steel, smooth,  Flow Area= 12.57 sf
   #5   Primary         36.20'   42.0" Round Culvert 3
                                 L= 24.6'  CMP, projecting, no headwall,  Ke= 0.900
                                 Inlet / Outlet Invert= 35.70' / 36.20'  S= -0.0203 '/'  Cc= 0.900
                                 n= 0.012  Steel, smooth,  Flow Area= 9.62 sf
   #6   Secondary       37.00'   100.0' long  x 12.0' breadth Broad-Crested Rectangular Weir
                                 Head (feet)  0.20  0.40  0.60  0.80  1.00  1.20  1.40  1.60
                                 Coef. (English)  2.57  2.62  2.70  2.67  2.66  2.67  2.66  2.64
```

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=34.14' TW=37.50' (Dynamic Tailwater)
3=Culvert 1 (Controls 0.00 cfs)
4=Culvert 2 (Controls 0.00 cfs)
5=Culvert 3 (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=34.14' TW=37.50' (Dynamic Tailwater)
6=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

Tertiary OutFlow Max=21.76 cfs @ 41.98 hrs HW=37.17' TW=24.15' (Dynamic Tailwater)
1=Culvert (Inlet Controls 21.76 cfs @ 8.55 fps)
2=Sharp-Crested Rectangular Weir (Passes 21.76 cfs of 39.66 cfs potential flow)

Pond 4P: Ash Pond B



Summary for Link 5L: Unit 2 Slurry Pond

Inflow = 5.79 cfs @ 0.00 hrs, Volume= 287.112 af
Primary = 5.79 cfs @ 0.01 hrs, Volume= 287.107 af, Atten= 0%, Lag= 0.6 min

Primary outflow = Inflow, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs

61 Point manual hydrograph, To= 0.00 hrs, dt= 10.00 hrs, cfs =

```
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79   5.79
5.79
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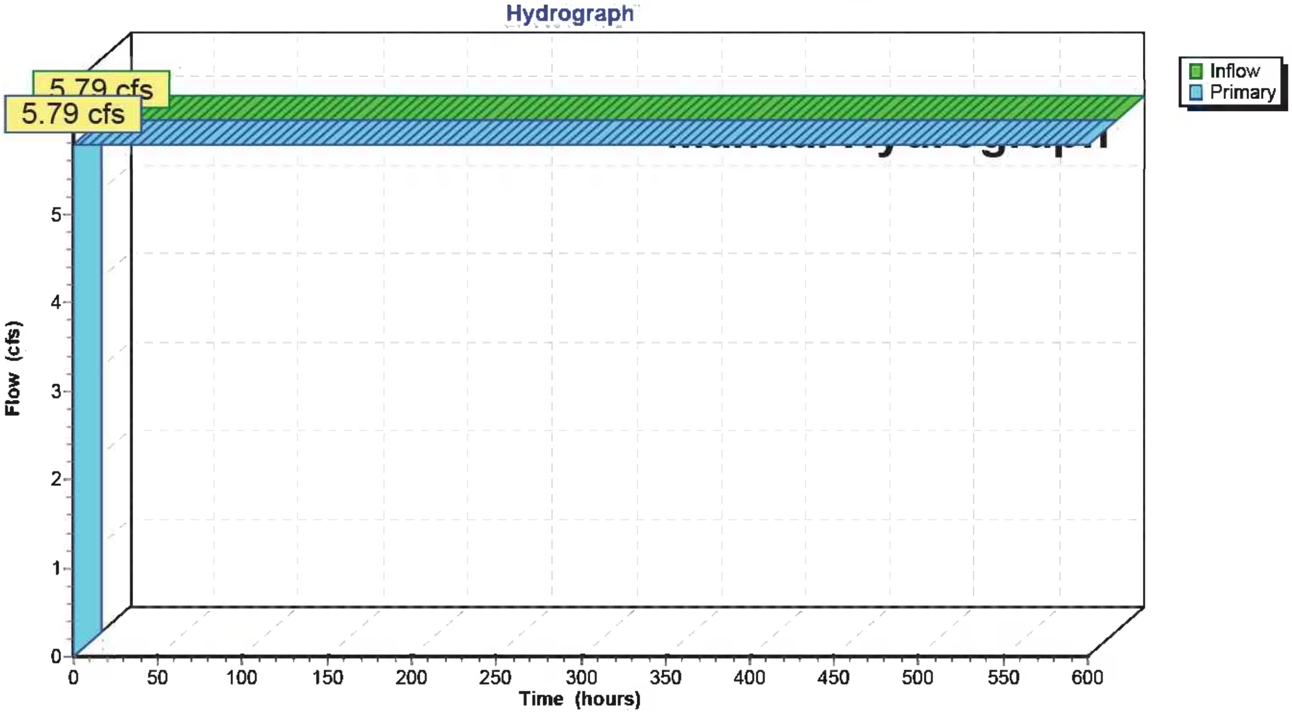
Link 5L: Unit 2 Slurry Pond



Summary for Link 6L: Process Water

```
Inflow   =     13.59 cfs @   0.00 hrs,  Volume=      673.896 af
Primary  =     13.59 cfs @   0.01 hrs,  Volume=      673.884 af,  Atten= 0%,  Lag= 0.6 min

Primary outflow = Inflow, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs

61 Point manual hydrograph,  To= 0.00 hrs,  dt= 10.00 hrs,  cfs =
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59    13.59
     13.59
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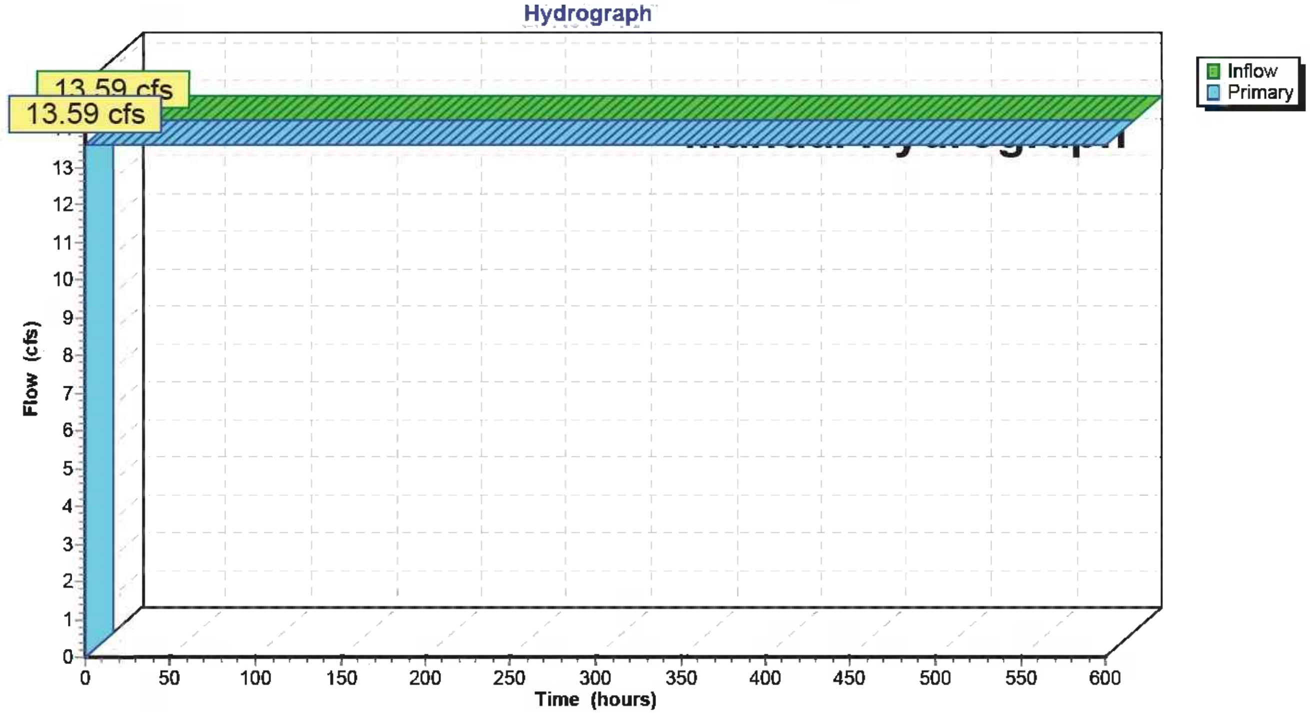
Link 6L: Process Water



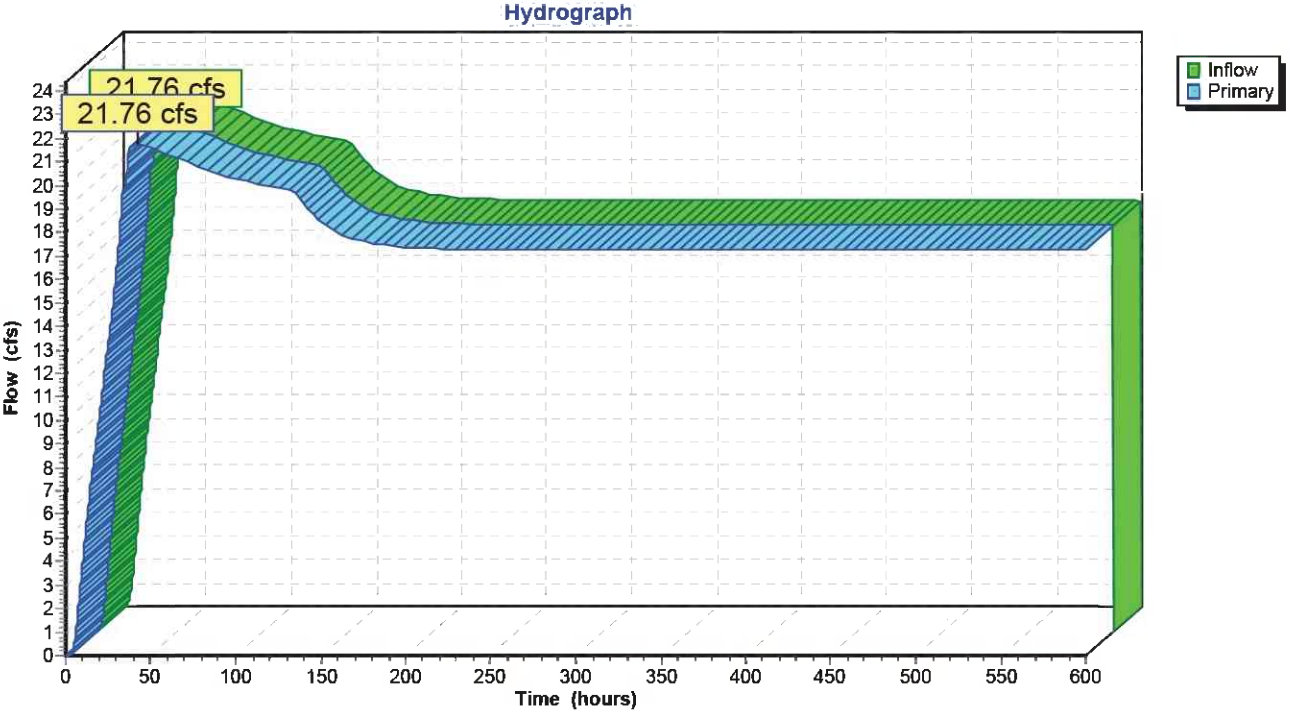
Summary for Link 7L: Discharge Canal

Inflow	=	21.76 cfs @	41.98 hrs,	Volume=	865.531 af
Primary	=	21.76 cfs @	41.99 hrs,	Volume=	865.531 af, Atten= 0%, Lag= 0.6 min

Primary outflow = Inflow, Time Span= 0.00-600.00 hrs, dt= 0.01 hrs

Fixed water surface Elevation= 24.15'

Link 7L: Discharge Canal



ATTACHMENT 2

Boring Logs

Geosyntec consultants
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-116*

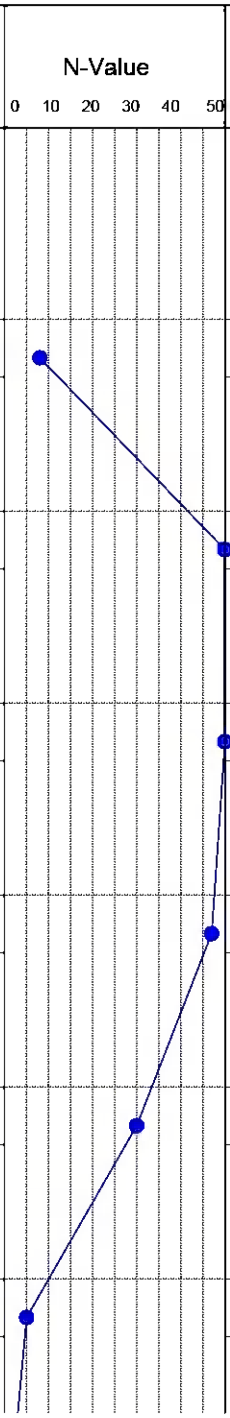
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/26/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *547438.1847*
EASTING: *2505180.2589*
GROUND ELEVATION: *41.44 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
40	0						
35	-5	Loose, gray to brown, fine SAND (SP), slightly silty.		5-7-1		11"	
30	-10	Very dense, light brown, fine SAND (SP), slightly silty.		12-27-39		13"	N-value = 66 blows/ft.
							Water level measured as 15.00 ft bgs on 9/27/2013.
25	-15	Very dense, gray to brown, fine SAND (SP), slightly silty.		16-26-26		13"	N-value = 52 blows/ft; MC = 26.2%; Fines = 6.0%. Driller indicates drilling fluid is being absorbed by the formation.
20	-20	Dense, brown to gray, fine SAND (SP), slightly silty.		15-17-30		13"	
15	-25	Medium dense, gray, fine SAND (SP), slightly silty.		8-15-15		11"	Borehole collapsed prior to abandonment at 25.00 ft bgs. Tremie grouted from 25.00 ft bgs.
10	-30	Loose, gray, silty, fine SAND (SM).		3-3-2		8"	MC = 22.9%; Gravel = 0.2%; Sand = 88.5%; Fines = 11.3%.



All depths referenced to ground surface.

Total Depth: 55.50 ft bgs

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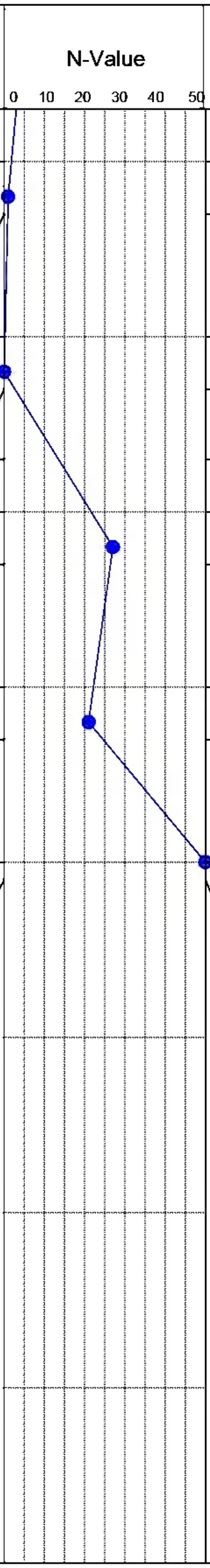
Greenville, SC 29601

BORING LOG

Borehole ID: *SPT-116*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	-35	Very loose, gray, silty, fine SAND (SM).	WOR-WOR-1	8"	MC = 29.9%; Fines = 11.2%.
5					
	-40	Very loose, gray (blue tint), clayey SAND (SC), some shells.	1/3"-WOR-WOR	18"	No recovery on SPT. Pushed Split Spoon sampler to obtain sample.
0			ST-1	24"	Shelby Tube advanced 24" from 41.50 to 43.50 ft bgs. Gravel = 2.7%; Sand = 71.5%; Silt = 6.2%; Clay = 19.6%. LL = 78; PL = 22; PI = 56. S.G. = 2.724
	-45	Medium dense, gray, clayey, fine SAND (SC).	6-10-17	10"	MC = 23.0%; Fines = 13.7%.
-5					
	-50	Medium dense, gray, fine SAND (SP), some shells, slightly silty.	3-7-14	11"	
-10					Hard drilling between 52.00 and 53.75 ft bgs.
	-55	Very dense, gray, fine SAND (SP), some gravel. Boring terminated at 55.50 ft bgs.	50/5"	2"	
-15					
	-60				
-20					
	-65				
-25					
	-70				
-30					
	-75				



All depths referenced to ground surface.

Total Depth: 55.50 ft bgs

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104 South Main Street
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Greenville, SC 29601

BORING LOG

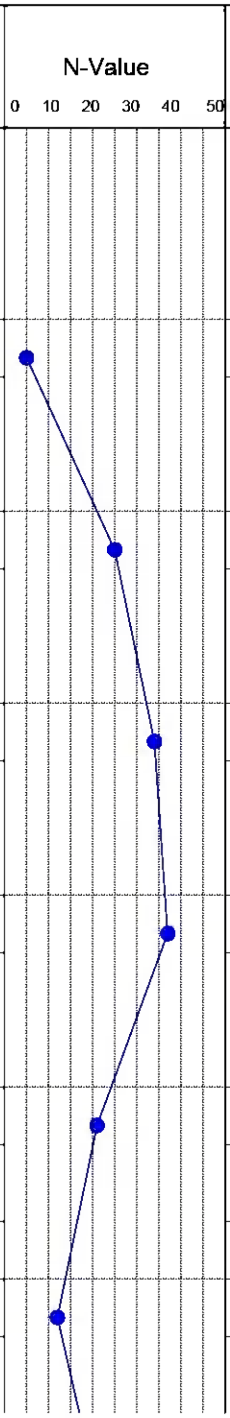
BOREHOLE ID: *SPT-117*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/27/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *547997.5776*
EASTING: *2505412.5604*
GROUND ELEVATION: *39.74 ft NGVD29*



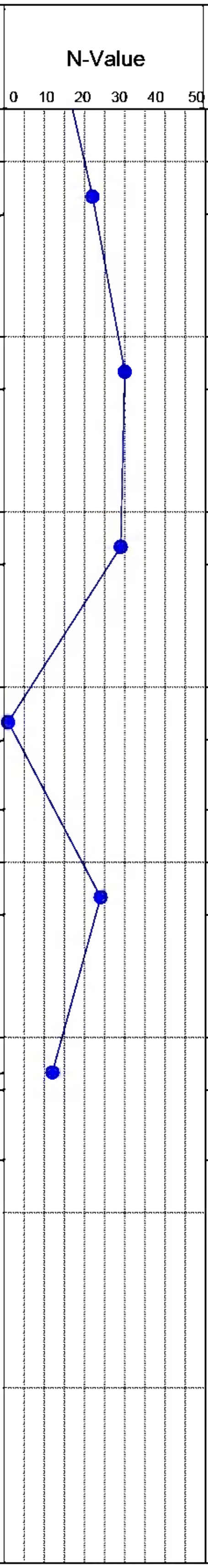
Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0						
35	-5	Loose, dark brown, clayey fine SAND (SC).		2-2-3		11"	MC = 20.3%; Gravel = 0.0%; Sand = 72.3%; Fines = 27.7%.
30	-10	Medium dense, gray to brown SAND (SP), slightly silty.		7-9-16		15"	
25	-15	Dense, light brown to brown layered), clean, fine SAND (SP).		11-15-19		12"	
	▼						Water level measured as 18.20 ft bgs on 9/30/2013.
20	-20	Dense, dark gray, silty, fine SAND (SM).		9-17-20		13"	MC = 21.5%; Fines = 12.5%.
15	-25	Medium dense, brown, silty, fine SAND (SM).		6-9-12		12"	
				ST-1		NR	Shelby Tube Pushed 20" from 26.50 to 27.17 ft bgs. No Recovery.
10	-30	Medium dense, brown, clayey, fine SAND (SC).		4-6-6		10"	MC = 46.6%; Fines = 43.5%.

All depths referenced to ground surface.

Total Depth: 63.50 ft bgs

BORING LOG

Borehole ID: *SPT-117*
Project No: *GSC5242*



Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
5	-35	Top: Medium dense, gray, clayey, fine SAND (SC). Bot: Medium dense, black to light brown, banded, silty, fine SAND (SM).	▨	WOR-5-17	13"	Borehole collapsed prior to abandonment at 35.00 ft bgs. Tremie grouted from 35.00 ft bgs. No H2O rose to surface.
0	-40	Medium dense, gray, fine SAND (SP), slightly silty.	░	4-11-19	12"	
-5	-45	Medium dense, gray, clean, fine SAND (SP).	░	9-12-17	10"	
-10	-50	Very soft, dark gray (blue tint) CLAY (CH), high plasticity.	▨	WOR-WOR-1	10"	MC = 55.1%; Gravel = 0.0%; Sand=42.3%; Silt = 13.3%; Clay = 44.4%. LL = 90; PL = 35; PI = 55.
			▨	ST-2	24"	Shelby Tube Pushed 24" from 51.50 to 53.50 ft bgs. MC = 58.5%; Gravel = 0.2%; Sand = 25.5%; Silt = 15.4%; Clay = 58.9%. LL = 82; PL = 24; PI=58.
-15	-55	Medium dense, gray, clayey, fine SAND (SC), some fine gravel.	▨	13-10-14	14"	Hard drilling at 55.00 ft bgs.
-20	-60	Stiff, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Black Mingo Formation).	▨	14-6-6	18"	Shelby Tube Pushed 24" from 61.50 to 63.50 ft bgs.
		Stiff, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Black Mingo Formation). Boring terminated at 63.50 ft bgs.	▨	ST-3	24"	MC=42.5%; Gravel=0.0%; Sand=19.5%; Silt=38.6%; Clay=41.9%. LL=65; PL=24; PI=41.k=1.40E-8 cm/s
-25	-65					
-30	-70					
-35	-75					

All depths referenced to ground surface.

Total Depth: 63.50 ft bgs

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104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-118*

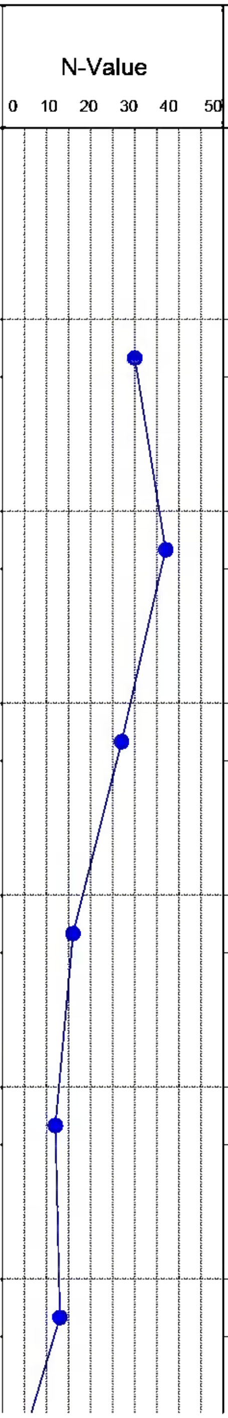
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/27/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *548238.4782*
EASTING: *2504339.6862*
GROUND ELEVATION: *39.67 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				
35	-5	Medium dense, light brown, fine SAND (SP), slightly silty.	7-13-17	14"	
30	-10	Dense, light brown to gray, clayey, fine SAND (SC), dry.	8-16-21	17"	MC = 14.3%; Gravel = 0.0%; Sand = 87.0%; Fines = 13.0%. Water level measured as 11.60 ft bgs on 9/30/2013.
25	-15	Medium dense, dark brown, clayey, fine SAND (SC).	8-13-14	11"	
20	-20	Medium dense, dark gray to black, fine SAND (SP), slightly silty.	5-7-9	14"	
15	-25	Medium dense, dark brown, fine SAND (SP), slightly silty.	3-5-7	9"	MC = 26.7%; Fines = 3.6%.
10	-30	Medium dense, gray to dark brown, fine SAND (SP), slightly silty.	3-4-9	11"	MC = 26.5%; Fines = 8.2%. Borehole collapsed prior to abandonment at 30.00 ft bgs.



All depths referenced to ground surface.

Total Depth: 55.25 ft bgs

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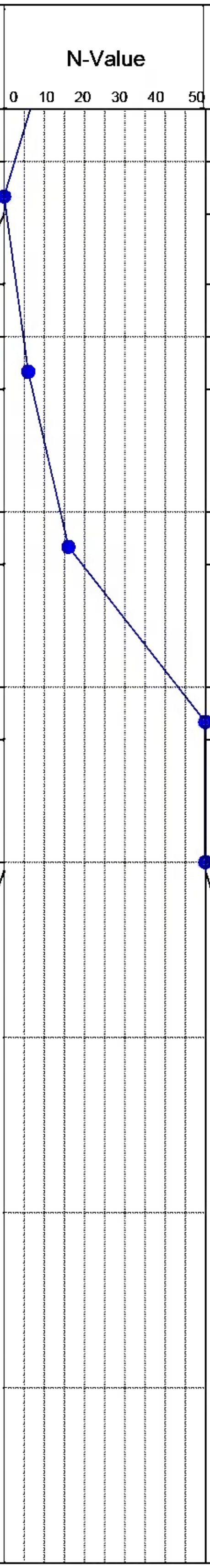
BORING LOG

Borehole ID: *SPT-118*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
5	-35	Very soft, gray to blue (with black streak) CLAY (CH), medium to high plasticity.	WOR-WOR-WOR	18"	Tremie grouted from 30.00 ft bgs. No H2O rose to surface during grouting.
			ST-1	24"	Shelby Tube advanced 24" from 36.50 to 38.50 ft bgs. Gravel = 0.0%; Sand = 1.2%; Silt = 22.4%; Clay = 76.4%. LL = 140; PL = 49; PI = 91.
0	-40	Top 7": Firm, gray, CLAY (CH), medium to high plasticity. Bot 6": Firm, gray, sandy CLAY (CL) with many shells.	1-3-3	13"	
-5	-45	Medium dense, gray (blue tint), fine SAND (SW) with some shells.	5-6-10	12"	MC = 20.9%; Gravel = 5.4%; Sand = 88.7%; Fines = 5.9%.
-10	-50	Very hard, gray, clayey SAND (SC), with some fine gravel.	4-36-25	9"	N-value = 61 blows/ft. Hard drilling between 52.00 and 52.50 ft bgs.
-15	-55	Boring terminated at 55.25 ft bgs.	50/3"	NR	Hard drilling between 54.10 and 55.00 ft bgs.
-20	-60				
-25	-65				
-30	-70				
-35	-75				



All depths referenced to ground surface.

Total Depth: 55.25 ft bgs

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104 South Main Street
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Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-119*

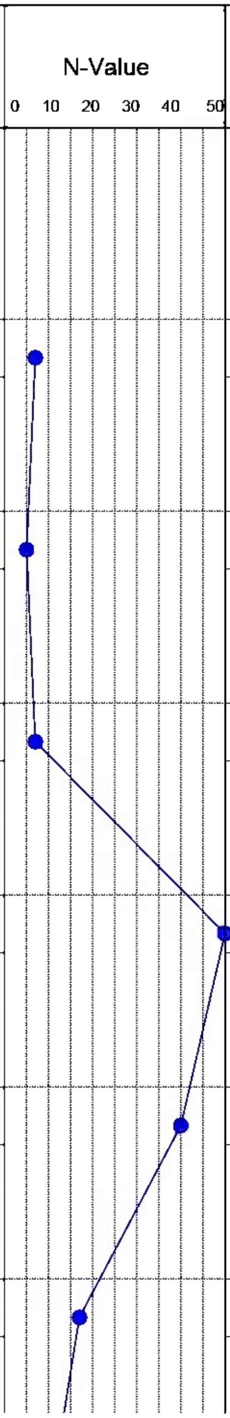
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *10/1/2013*
GEOSYNTEC REPRESENTATIVE: *S. Sanchez*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *548265.3975*
EASTING: *2502982.7280*
GROUND ELEVATION: *42.72 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				
40					
	-5	Loose, gray to dark gray, silty SAND (SM), moist.	2-3-4	14"	
35					
	-10	Top 8": Loose, dark gray, silty SAND (SM), wet. Bot 2": Soft, dark gray SILT (ML), wet, low plasticity.	2-2-3	17"	MC = 36.0%; Gravel = 0.4%; Sand = 9.0%; Fines = 90.6%.
30					
	-15	Loose, gray to dark gray, fine to coarse SAND (SW), moist, slightly silty, slightly gravelly.	2-3-4	11"	MC = 19.9%; Fines = 9.8%.
25					
	▼				Water level measured as 18.40 ft bgs on 10/9/2013.
	-20	Top 4": Very dense, gray to dark gray, silty SAND (SM), wet. Bot 8": Very dense, brown, fine to medium SAND (SP), wet.	12-30-40	14"	N-value = 70 blows/ft.
20					
	-25	Top 9": Dense, gray, fine to medium SAND (SP), wet, some shells, slightly gravelly.	12-16-24	9"	Driller indicates drilling fluid is being absorbed by the formation.
15					
	-30	Medium dense, gray to light gray, fine to coarse SAND (SW), wet, some shells, slightly silty.	4-8-9	11"	
10					



All depths referenced to ground surface.

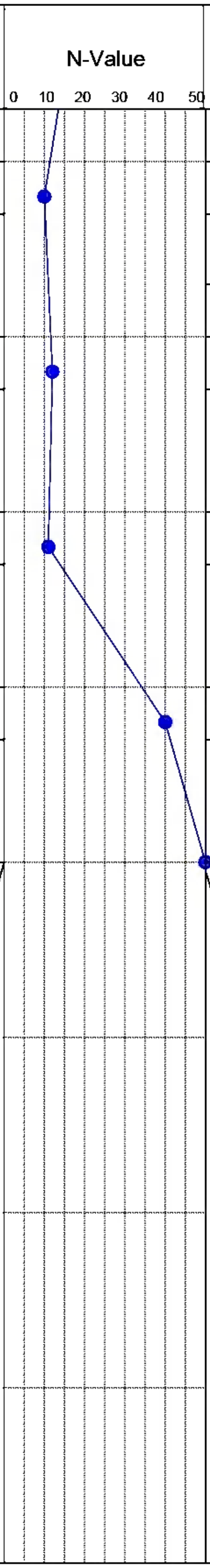
Total Depth: 55.00 ft bgs

BORING LOG

Borehole ID: *SPT-119*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	-35	Medium dense, gray to light gray, fine to coarse SAND (SW), wet, some shells, slightly silty.		3-4-6	18"	MC = 22.3%; Gravel = 1.0%; Sand = 89.8%; Fines = 9.2%.
5					24"	Borehole collapsed prior to abandonment at 35.00 ft bgs. Tremie grouted from 35.00 ft bgs. No H2O rose to surface during grouting.
	-40	Medium dense, gray to light gray, fine to medium SAND (SW), wet, some shells, slightly silty.		5-6-6	13"	
0						
	-45	Medium dense, gray, fine to medium SAND (SP), wet, some shells, slightly silty.		3-4-7	12"	MC = 25.8%; Fines = 9.4%.
-5						
	-50	Dense, gray, gravelly fine SAND (SP), moist, slightly silty.		2-22-18	9"	MC = 16.8%; Gravel = 32.7%; Sand = 45.5%; Fines = 21.8%.
-10						
	-55	Boring terminated at 55.00 ft bgs.		50/0"	NR	
-15						
	-60					
-20						
	-65					
-25						
	-70					
-30						
	-75					



All depths referenced to ground surface.

Total Depth: 55.00 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-120*

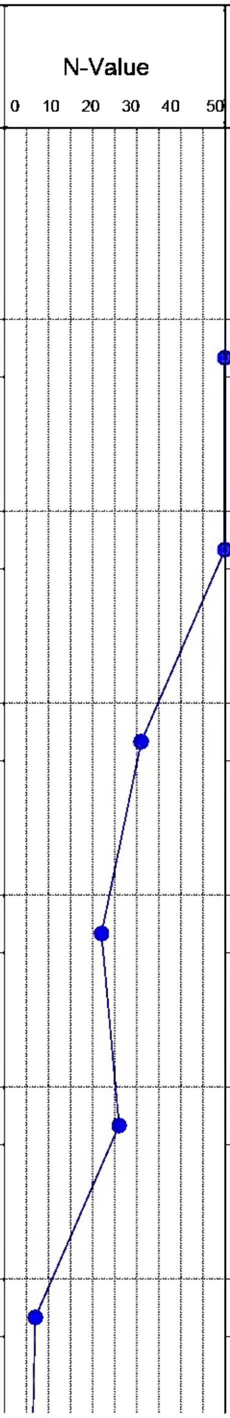
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/25/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *546980.5202*
EASTING: *2503057.0075*
GROUND ELEVATION: *41.06 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
40	0						
35	-5	Very dense, light brown SAND (SP), slightly silty.		15-26-35		15"	N-value = 61 blows/ft.
30	-10	Very dense, light brown to dark brown layered, fine SAND (SP).		20-29-37		12"	N-value = 66 blows/ft.
	▼						Water level measured as 12.20 ft bgs on 9/26/2013.
25	-15	Dense, black to gray, silty, fine SAND (SM).		10-14-17		11"	MC = 18.1%; Fines = 13.5%.
20	-20	Medium dense, gray, silty, fine SAND (SM), some gravel.		9-11-11		12"	
15	-25	Medium dense, dark brown, fine SAND (SP), slightly silty.		6-11-15		12"	
10	-30	Loose, brown fine SAND (SP), slightly silty.		2-3-4		9"	MC = 27.4%; Fines = 4.1%



All depths referenced to ground surface.

Total Depth: 61.50 ft bgs

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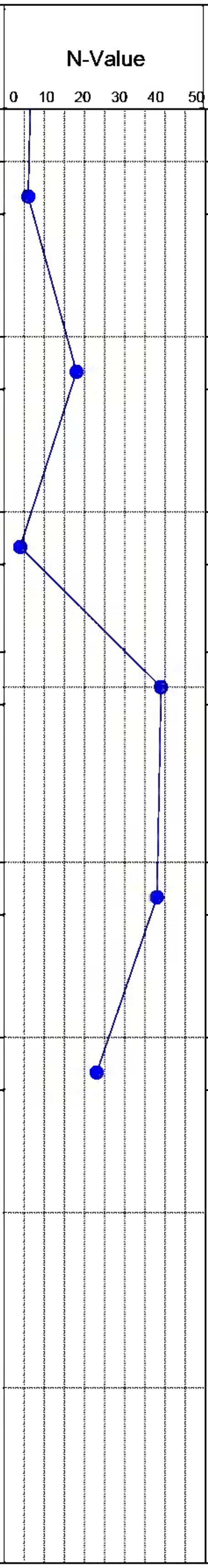
BORING LOG

Borehole ID: *SPT-120*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
5	-35	Loose, light brown to gray, fine SAND (SP), slightly silty.		1-3-3	9"	MC = 25.5%; Fines = 4.3% Driller indicates drilling fluid is being absorbed by the formation.
0	-40	Medium dense, gray, medium to coarse SAND (SP), slightly silty.		2-6-12	10"	
-5	-45	Very loose, gray, clayey, fine to medium SAND (SC) with some shells.		3-1-3	10"	MC = 23.3%; Gravel = 1.7%; Sand = 86.0%; Fines = 12.3%.
-10	-50	Hard, gray, clayey, fine GRAVEL (GC).		7-25-14	10"	Hard drilling at 49.00 ft bgs. Borehole collapsed prior to abandonment at 50.00 ft bgs. Tremie grouted from 50.00 ft bgs. Hard drilling between 52.00 and 54.00 ft bgs.
-15	-55	Dense, clayey, fine SAND (SC) with some fine gravel. Cemented fragment 1" from top of sample.		7-27-11	12"	Hard drilling between 58.80 and 59.90 ft bgs.
-20	-60	Medium dense, gray, clayey, fine sand (SC), some shells, some fine gravel. Boring terminated at 61.50 ft bgs.		16-12-11	13"	
-25	-65					
-30	-70					
	-75					



All depths referenced to ground surface.

Total Depth: 61.50 ft bgs

Geosyntec consultants
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-121*

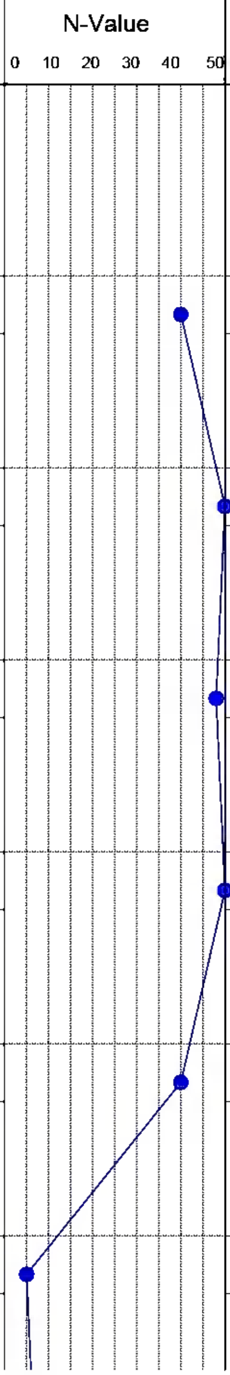
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/30/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *546076.8677*
EASTING: *2503720.3193*
GROUND ELEVATION: *40.82 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
40	0						
35	-5	Dense, light brown with brown layering, clean, fine SAND (SP).		9-16-24		12"	
30	-10	Very dense, light brown with brown layering, fine SAND (SP) with silt.		15-27-33		12"	N-value = 60 blows/ft; MC = 14.8%; Fines = 10.7%. Water level measured as 11.50 ft bgs on 10/1/2013.
25	-15	Dense, light brown with some black, fine SAND (SP), slightly silty.		15-21-27		12"	
20	-20	Very dense, dark brown, fine SAND (SP), slightly silty.		10-27-38		13"	N-value = 65 blows/ft. Borehole collapsed prior to abandonment at 20.00 ft bgs. Tremie grouted from 20.00 ft bgs.
15	-25	Dense, dark brown, fine SAND (SP), slightly silty.		16-23-17		11"	
10	-30	Loose, brown, clayey, fine SAND (SP) with silt.		1-2-3		10"	MC = 27.8%; Gravel = 0.0%; Sand = 92.6%; Fines = 7.4%.



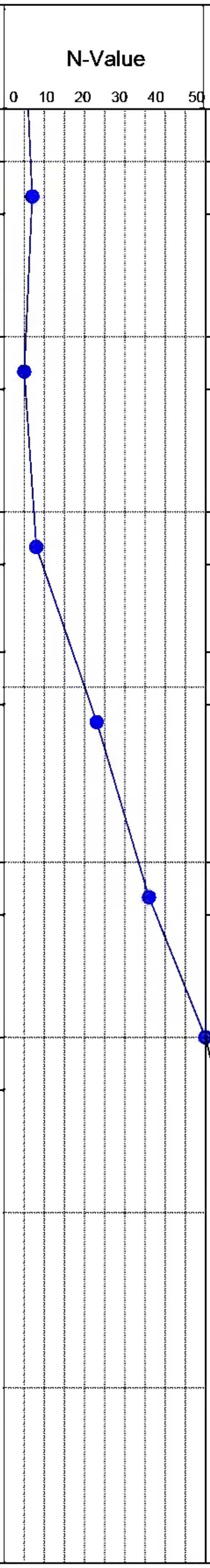
All depths referenced to ground surface.

Total Depth: 60.00 ft bgs

BORING LOG

Borehole ID: *SPT-121*

Project No: *GSC5242*



Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
5	-35	Loose, gray to dark gray, fine SAND (SC).		1-3-4	12"	MC = 36.0%; Fines = 14.3%.
0	-40	Loose, gray, clayey, fine SAND (SC) with some shellls.		8-3-2	7"	MC = 17.3%; Gravel = 11.0%; Sand = 73.7%; Fines = 15.3%.
-5	-45	Loose, gray, clayey fine GRAVEL and shells (GC), some fine sand.		3-4-4	18"	MC = 31.1%; Fines = 14.5%.
-10	-50	Medium dense, gray, clayey, fine GRAVEL and shells (GC), some fine sand.		13-15-8	15"	Hard drilling between 52.50 and 54.75 ft bgs.
-15	-55	Dense, clayey, fine SAND (SC), some fine gravel.		11-10-26	14"	
-20	-60	Boring terminated at 60.00 ft bgs.		50/0"	NR	
-25	-65					
-30	-70					
	-75					

All depths referenced to ground surface.

Total Depth: 60.00 ft bgs

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104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-123*

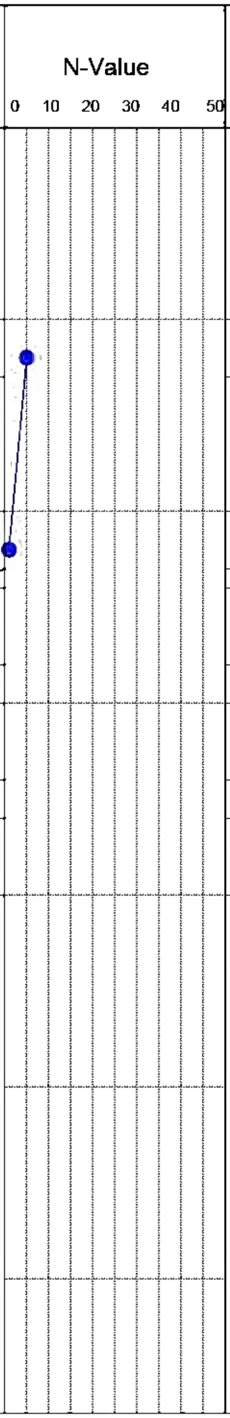
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/30/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 550 X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *547690.6373*
EASTING: *2504330.9712*
GROUND ELEVATION: *44.96 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0		▒				
40	-5	Firm, black SILT (ML) (Fly Ash), slightly sandy.	▒	2-3-2	●	11"	
35	-10	Very soft, black SILT (ML) (Fly Ash), slightly sandy.	▒	2-1-WOR	●	14"	Water level measured as 11.30 ft bgs on 10/9/2013.
			▒	ST-1		0"	Shelby Tube advanced 24" from 12.00 to 14.00 ft bgs. No Recovery.
30	-15		▒	ST-2		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 15.00 to 17.00 ft bgs. Gravel = 0.0%; Sand = 3.8%; Silt = 70.4%; Clay = 25.8%. LL =NP; PL = NP; PI = NP. S.G. = 2.308. pH = 5.7. CO3= 0%.
		Loose, dark gray to brown, clayey SAND (SC), wet.	▨	ST-3		22"	Shelby Tube advanced by means of a Piston Sampler 24" from 18.00 to 20.00 ft bgs. Gravel = 0.0%; Sand = 80.2%; Silt = 10.8%; Clay = 9.0%. LL = NP; PL = NP; PI = NP.
25	-20	Boring terminated at 20.00 ft bgs.					Borehole did not collapse prior to abandonment. Tremie grouted from 20.00 ft bgs. Driller encounters sand at 20.00 ft bgs.
20	-25						
15	-30						



All depths referenced to ground surface.

Total Depth: 20.00 ft bgs

Geosyntec consultants
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

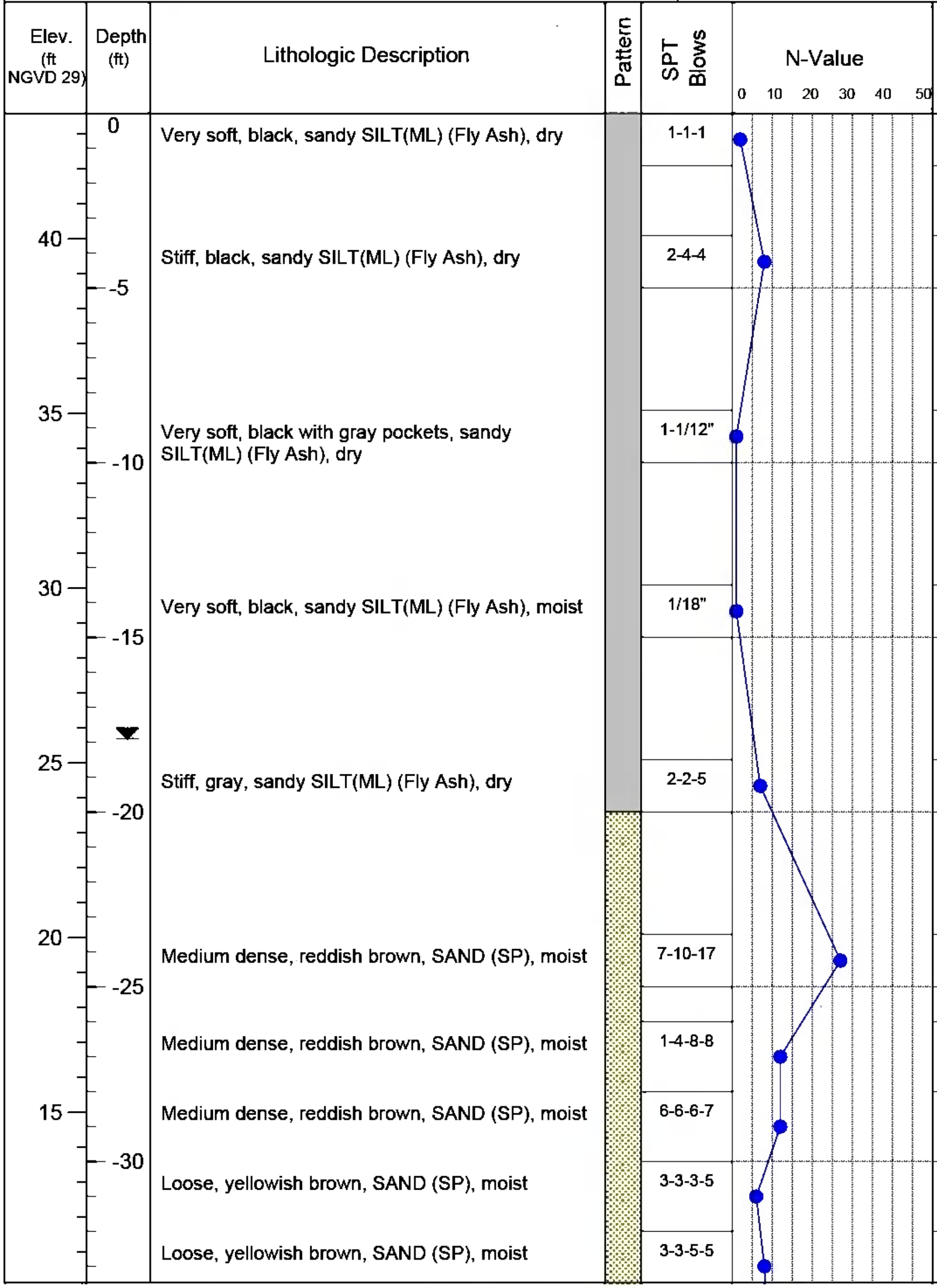
BOREHOLE ID: *SPT-304*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/22/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Carolina Drilling, Co.*
DRILLER NAME: *J. Anderson*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *548081.123*
EASTING: *2503172.195*
GROUND ELEVATION: *43.59 ft NGVD29*



Lithologic Description	SPT Blows	Recovery	Comments
Very soft, black, sandy SILT(ML) (Fly Ash), dry	1-1-1	15"	
Stiff, black, sandy SILT(ML) (Fly Ash), dry	2-4-4	11"	
Very soft, black with gray pockets, sandy SILT(ML) (Fly Ash), dry	1-1/12"	14"	
Very soft, black, sandy SILT(ML) (Fly Ash), moist	1/18"	13"	MC = 48.9%; LL =NP; PL = NP; PI = NP; Fines = 92.7%
Stiff, gray, sandy SILT(ML) (Fly Ash), dry	2-2-5	10"	Water level was measured 17.9 ft bgs at 7.10 am on 03/31/2016
Medium dense, reddish brown, SAND (SP), moist	7-10-17	16"	
Medium dense, reddish brown, SAND (SP), moist	1-4-8-8	12"	
Medium dense, reddish brown, SAND (SP), moist	6-6-6-7	12"	
Loose, yellowish brown, SAND (SP), moist	3-3-3-5	16"	MC = 27%; Fines = 2.5%
Loose, yellowish brown, SAND (SP), moist	3-3-5-5	14"	

All depths referenced to ground surface.

Total Depth: 59.0 ft bgs

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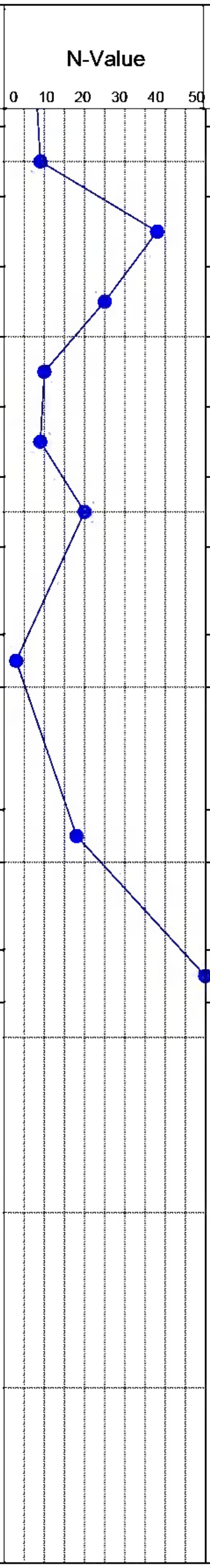
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

Borehole ID: *SPT-304*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
10	-35	Loose, dark brown to gray, clayey SAND (SC), moist	░ ▨	3-4-5-8	13"	
		Dense, light gray, clayey SAND (SC), moist	▨	8-16-22-25	12"	MC = 23%; Fines = 6.3%
5		Medium dense, gray, SAND (SP), moist	░	8-13-12-9	18"	
	-40	Loose, gray, clayey SAND (SC), moist	▨	4-5-5-5	11"	
0		Loose, gray, clayey SAND (SC), moist, with shell and rock fragments	▨	3-3-6-9	18"	
	-45	Medium dense, gray, clayey SAND (SC), moist, with shell fragments	▨	8-9-11-9	12"	
-5			▨			
	-50	Very loose, gray, clayey SAND(SC), moist, with shell fragments	▨	2-1-2	18"	MC = 31%; LL = NP; PL = NP; PI = NP; Fines =15.0%
-10			▨			
	-55	Medium dense, gray, clayey SAND (SC), moist, with shell fragments	▨	9-8-10	12"	
			▩			
-15		Very dense, gray, clayey GRAVEL (GC) with crushed rock, moist Boring terminated at 59 ft bgs.	▩	50/1"	4"	N-value = 50 blows/1" (Chicora)
	-60					
-20						
	-65					
-25						
	-70					
-30						
	-75					



All depths referenced to ground surface.

Total Depth: 59.0 ft bgs

Geosyntec consultants

104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-305*

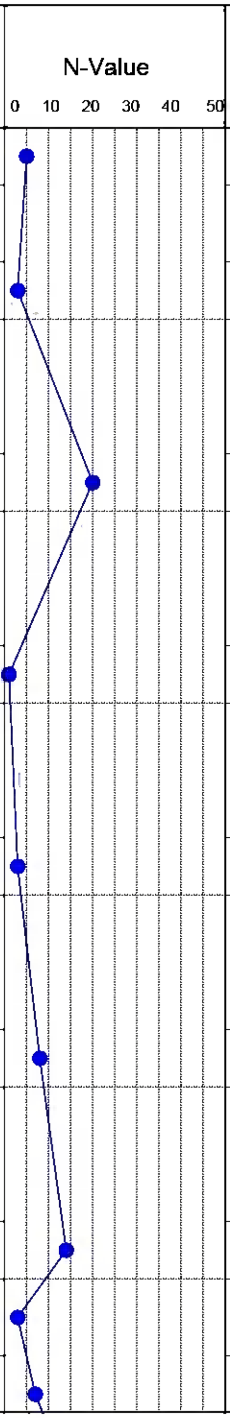
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/29/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Carolina Drilling, Co.*
DRILLER NAME: *J. Anderson*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *547800.0973*
EASTING: *2504921.296*
GROUND ELEVATION: *44.72 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	0	Firm, black, sandy SILT(ML) (Fly Ash), dry	▨	1-1-4	10"	
40	-5	Soft, black, sandy SILT(ML) (Fly Ash), moist	▨	2-1-2	10"	
35	-10	Very stiff, black, sandy SILT(ML) (Fly Ash), moist	▨	5-9-11	14"	
	▼		▨			Water level was measured 11.63 ft bgs at 8.14am on 03/31/2016
30	-15	Very soft, black, sandy SILT(ML) (Fly Ash), moist	▨	1-1/12"	17"	MC = 33%; LL =NP; PL = NP; PI = NP; Gravel = 0.0%; Sand = 6.9%; Fines = 93.1%
25	-20	Soft, black, sandy SILT(ML) (Fly Ash), dry	▨	2-1-2	10"	
20	-25	Stiff, gray to dark brown, sandy SILT (ML), moist	▥	WOH-2-6	10"	MC = 26%; LL = NP; PL = NP; PI = NP; Gravel = 1.0%; Sand = 43.5%; Fines = 55.5%;
15	-30	Medium dense, dark brown to reddish brown, SAND (SP), moist	░	6-7-7	11"	
		Very loose, reddish brown, SAND (SP), moist	░	1-1-2-5	10"	MC = 29%; Fines = 4.2%
		Loose, reddish brown, SAND (SP), moist	░	2-2-5-5	11"	



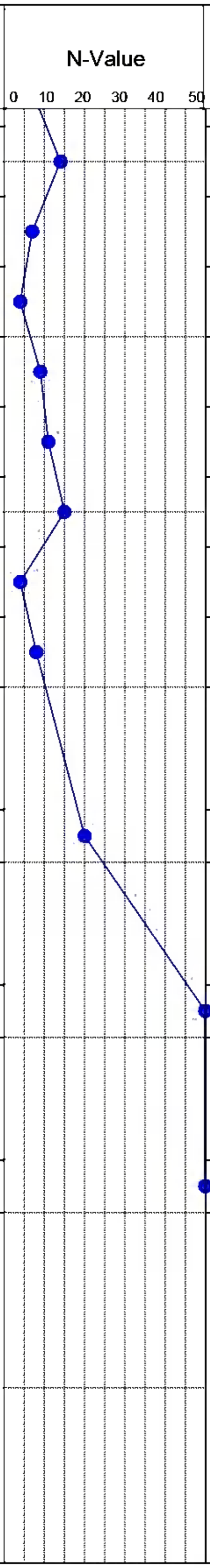
All depths referenced to ground surface.

Total Depth: 65 ft bgs

BORING LOG

Borehole ID: *SPT-305*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
10	-35	Medium dense, reddish brown, SAND (SP), moist		5-7-7-7	11"	
		Loose, reddish brown, SAND (SP), moist		4-3-4-4	16"	
5	-40	Very loose, light gray, silty SAND (SM), moist		2-2-2-2	18"	MC = 28%; Fines = 6.6%
		Loose, gray to light gray, clayey SAND (SC), moist		1-1-8-10	13"	
		Medium dense, light gray, SAND (SP), moist		6-6-5-5	9"	
0	-45	Medium dense, gray, SAND (SP), moist		8-8-7-7	17"	
		Very loose, dark gray, SAND (SP), moist		1-1-3-2	14"	MC = 22%; LL =NP; PL = NP; PI = NP; Gravel = 0.1%; Sand = 94.2%; Fines = 5.6%
-5	-50	Loose, dark gray, SAND (SP), moist		3-3-5-5	9"	
-10	-55	Medium dense, gray, SAND (SP), moist		7-9-11	9"	
-15	-60	Very dense, dark gray, clayey GRAVEL (GC), moist, with rock fragments		45-40-31	15"	N = 81 blows
-20	-65	Very dense, dark gray, clayey GRAVEL (GC) with crushed rock, moist		50/3"	3"	N-value = 50 blows/3" (Chicora)
		Boring terminated at 65 feet.				
-25	-70					
-30	-75					



All depths referenced to ground surface.

Total Depth: 65 ft bgs

Geosyntec consultants
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-306*

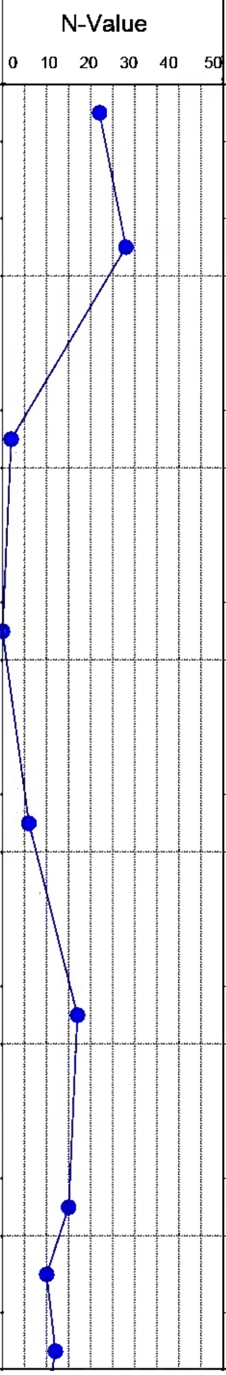
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/22/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Carolina Drilling, Co.*
DRILLER NAME: *J. Anderson*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *546810.8873*
EASTING: *2503498.253*
GROUND ELEVATION: *44.18 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	0	Very stiff black, sandy SILT(ML) (Fly Ash), moist	▒	6-10-12	18"	
40	-5	Very stiff black, sandy SILT(ML) (Fly Ash), moist	▒	9-13-15	15"	
35	-10	Very soft, black, sandy SILT(ML) (Fly Ash), moist	▒	1-1-1	12"	
30	-15	Very soft, black, sandy SILT(ML) (Fly Ash), moist	▒	WOH/18"	18"	
25	-20	Loose, reddish brown, SAND (SP), moist	░	3-3-3	8"	
20	-25	Medium dense, reddish brown, SAND (SP), moist	░	3-7-10	15"	MC = 30%; Fines=8.6%
15	-30	Medium dense, reddish brown, SAND (SP), moist	░	4-7-8	12"	
		Loose, brown, SAND (SP), moist	░	WOH-2-4-6	10"	
		Loose, brown, SAND (SP), moist	░	WOH-2-5-7	9"	MC = 28%; LL =NP; PL = NP; PI = NP; Fines=4.7%

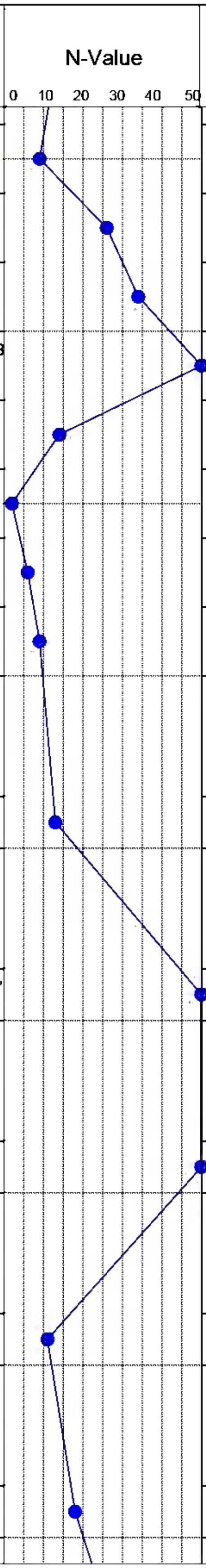


All depths referenced to ground surface.

Total Depth: 170.0 ft bgs

BORING LOG

Borehole ID: *SPT-306*
Project No: *GSC5242*



Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
10	-35	Medium dense, gray, SAND (SP), moist		1-8-3-6	10"	
		Medium dense, gray, SAND (SP), moist		2-3-11-15	11"	
5		Medium dense, gray, SAND (SP), moist		5-11-13-21	11"	
	-40	Dense, gray, SAND (SP), moist		12-16-26-28	18"	
		Medium dense, gray, SAND (SP), moist		8-10-8-6	13"	
0	-45	Very loose, gray, clayey SAND (SC), moist		3-1-1/12"	18"	
		Very loose, gray, clayey SAND (SC), moist, with traces of rock fragments		2-1-5/12"	17"	
-5		Loose, gray, clayey SAND (SC), moist, with traces of rock fragments and shells		3-3-4-5	21"	
	-50					
-10	-55	Medium dense, gray, clayey SAND (SC), moist, with gravel and traces of shell fragments		6-6-7	8"	MC = 17%; LL =NP; PL = NP; PI = NP; Fines=12.2%
-15	-60	Very dense, gray, clayey SAND (SC), moist, with rock fragments		24-29-50/4"	18"	N-value = 50 blows/4" (Chicora)
-20	-65	Very dense, gray, clayey SAND (SC), moist, with rock fragments		28-50-23	17"	
-25	-70	Medium dense, gray, clayey SAND (SC), moist, with shell fragments		14-6-5	16"	
-30	-75	Very stiff, greenish gray, fat CLAY (CH) with sand, moist		5-7-11	18"	MC = 49%; LL = 59; PL = 26; PI = 33; Gravel = 2.7%; Sand = 31.7%; Fines = 65.7%

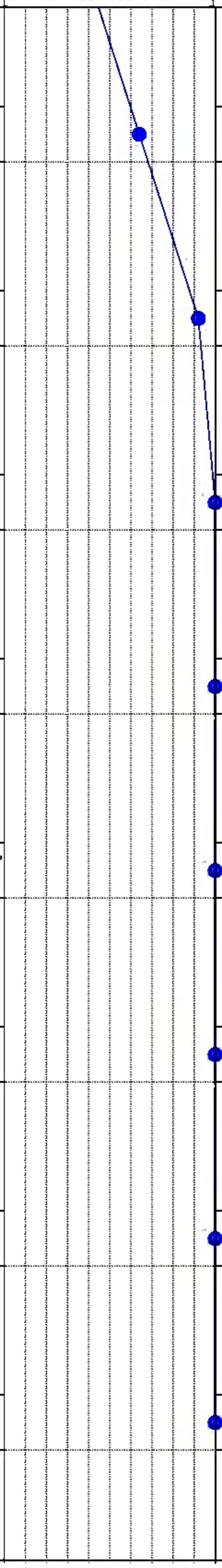
All depths referenced to ground surface.

Total Depth: 170.0 ft bgs

BORING LOG

Borehole ID: *SPT-306*

Project No: *GSC5242*



Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
-35	-80	Hard, greenish gray, fat CLAY (CH) with sand, moist	▨	17-17-15		18"	
-40	-85	Hard, greenish gray, fat CLAY (CH) with sand, moist	▨	11-16-30		22"	
-45	-90	Very hard, greenish gray, fat CLAY (CH) with sand, moist	▨	13-37-22		24"	
-50	-95	Very dense, dark gray, clayey SAND (SC), moist, with rock fragments	▨	36-50/1"		12"	MC = 48%; LL = 56; PL = 34; PI =22; Gravel = 3.2%; Sand = 88.8%; Fines = 8%
-55	-100	Very hard, dark gray, fat CLAY (CH), moist, with sand and rock fragments	▨	13-36-50/3"		18"	
-60	-105	Very hard, dark gray, fat CLAY (CH), moist, with sand and rock fragments	▨	16-27-37		24"	
-65	-110	Hard, dark gray, fat CLAY (CH) with sand, moist, with rock fragments	▨	7-13-37		22"	MC = 65%; LL = 77; PL = 38; PI =39; Gravel = 0.0%; Sand = 27.5%; Fines = 72.5%
-70	-115	Very hard, greenish gray, fat CLAY (CH) with sand, moist, with rock fragments	▨	50/4"		4"	

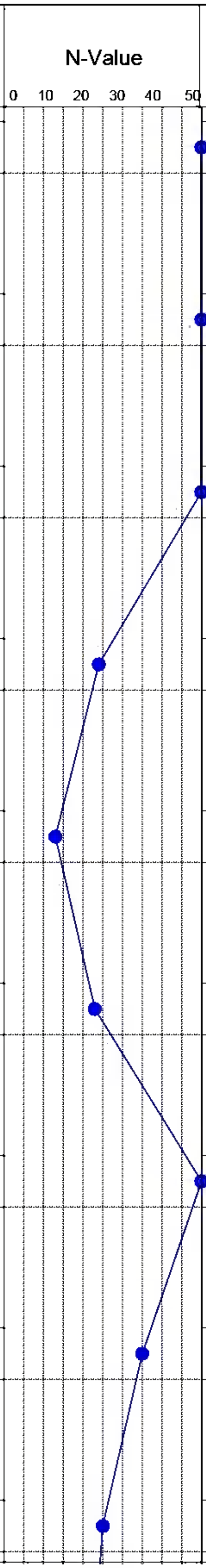
All depths referenced to ground surface.

Total Depth: 170.0 ft bgs

BORING LOG

Borehole ID: *SPT-306*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
-75	-120	Very hard, greenish gray, fat CLAY (CH) with sand, moist, with cemented rock fragments	▨	50/1"	1"	
-80	-125	Very dense, greenish gray, clayey SAND (SC), moist, with rock fragments	▨	15-50/5"	14"	MC = 27%; LL = NP; PL = NP; PI = NP; Fines = 20.8%
-85	-130	Very dense, white, clayey SAND (SC) with crushed rock and shell fragments, moist	▨	50/3"	3"	
-90	-135	Very stiff, greenish gray, sandy CLAY (CL), moist	▨	6-8-16	24"	
-95	-140	Stiff, white, sandy CLAY (CL) with sand, moist	▨	3-6-7	20"	MC = 37%; LL = 36; PL = 24; PI =12; Gravel = 0.0%; Sand = 47.6%; Fines = 52.4%
-100	-145	Stiff, greenish gray, sandy CLAY (CL), moist	▨	10-11-12	20"	
-105	-150	Very hard, white, sandy SILT (ML), moist	▥	5-7-50/1"	20"	MC = 30%; LL = 24; PL = 21; PI = 3; Gravel = 5.1%; Sand = 43.2%; Fines = 51.6%
-110	-155	Dense, greenish gray, clayey SAND (SC), moist	▨	8-11-24	24"	
-115	-160	Medium dense, gray, clayey SAND (SC), moist	▨	6-9-16	22"	MC = 34%; LL = NP; PL = NP; PI = NP; Gravel = 0.0%; Sand = 71.1%; Fines = 28.9%



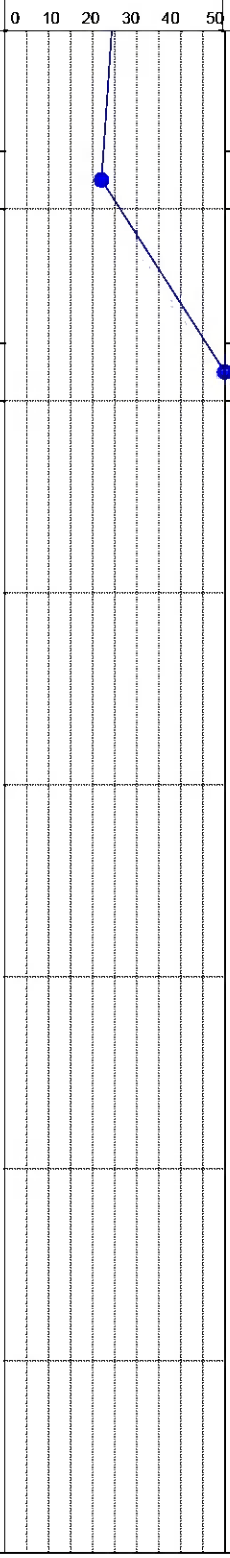
All depths referenced to ground surface.

Total Depth: 170.0 ft bgs

BORING LOG

Borehole ID: *SPT-306*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
-120	-165	Medium dense, gray, clayey SAND (SC), moist	▨	7-7-15		24"	MC = 36%; LL = NP; PL = NP; PI = NP; Gravel = 0.0%; Sand = 67.8%; Fines = 32%
-125	-170	No recovery	▨	50/0"		0"	
		Boring terminated at 170 ft					Water level was not measured as borehole was grouted right after the drilling.
-130	-175						
-135	-180						
-140	-185						
-145	-190						
-150	-195						
-155	-200						



All depths referenced to ground surface.
Total Depth: 170.0 ft bgs

ATTACHMENT 3

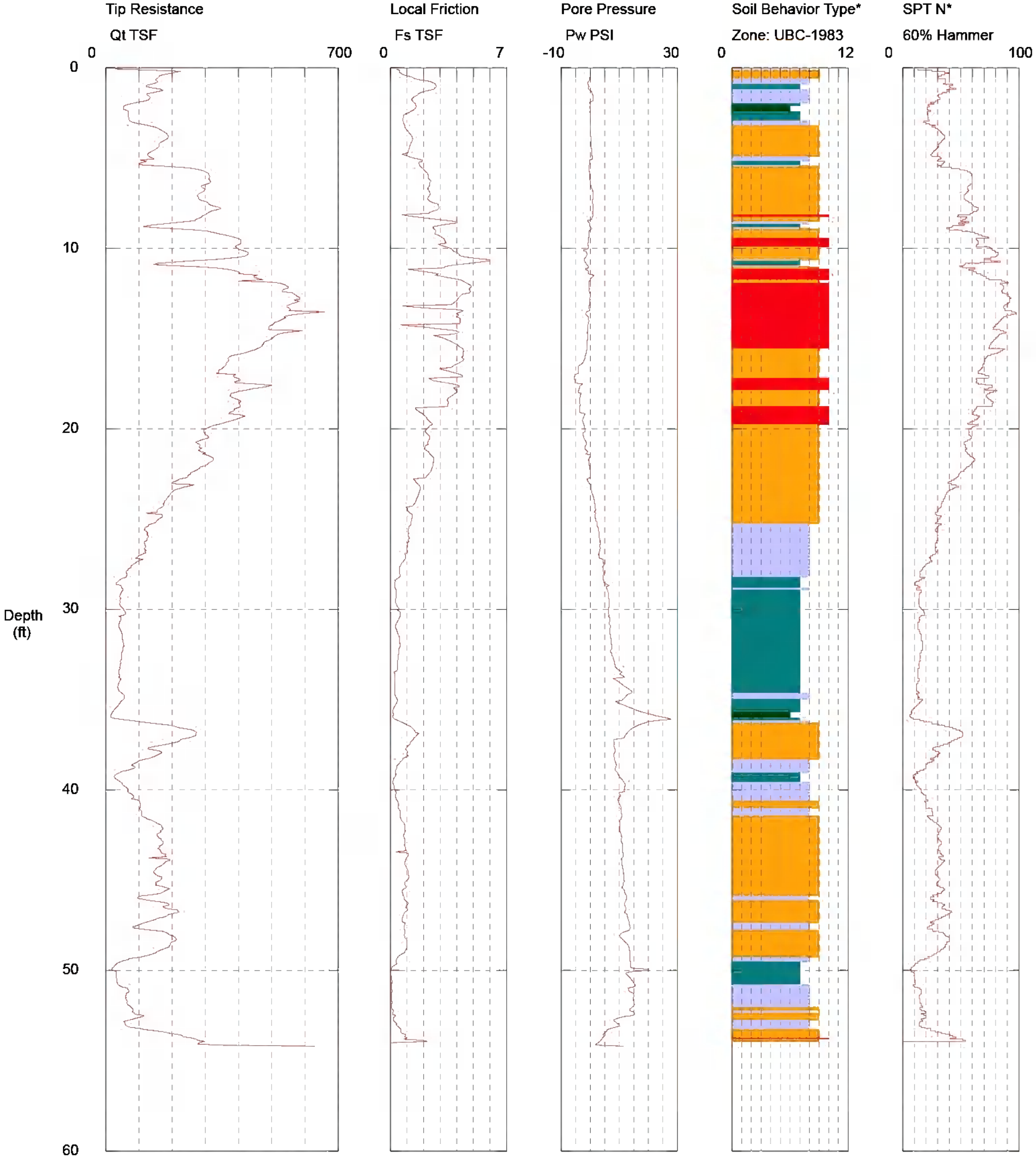
CPT Sounding Data

ATTACHMENT 3-A

CPT Sounding Logs
(Provided by Mid-Atlantic Drilling and
Terracon)

Operator: Cory Robison
Sounding: CPT-137
Cone Used: DDG1195

CPT Date/Time: 10/2/2013 8:30:56 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 54.20 feet

Depth Increment = 0.066 feet

⬜ 1 sensitive fine grained
⬛ 2 organic material
🟥 3 clay
🟫 4 silty clay to clay
🟫 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟪 8 sand to silty sand
🟧 9 sand
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

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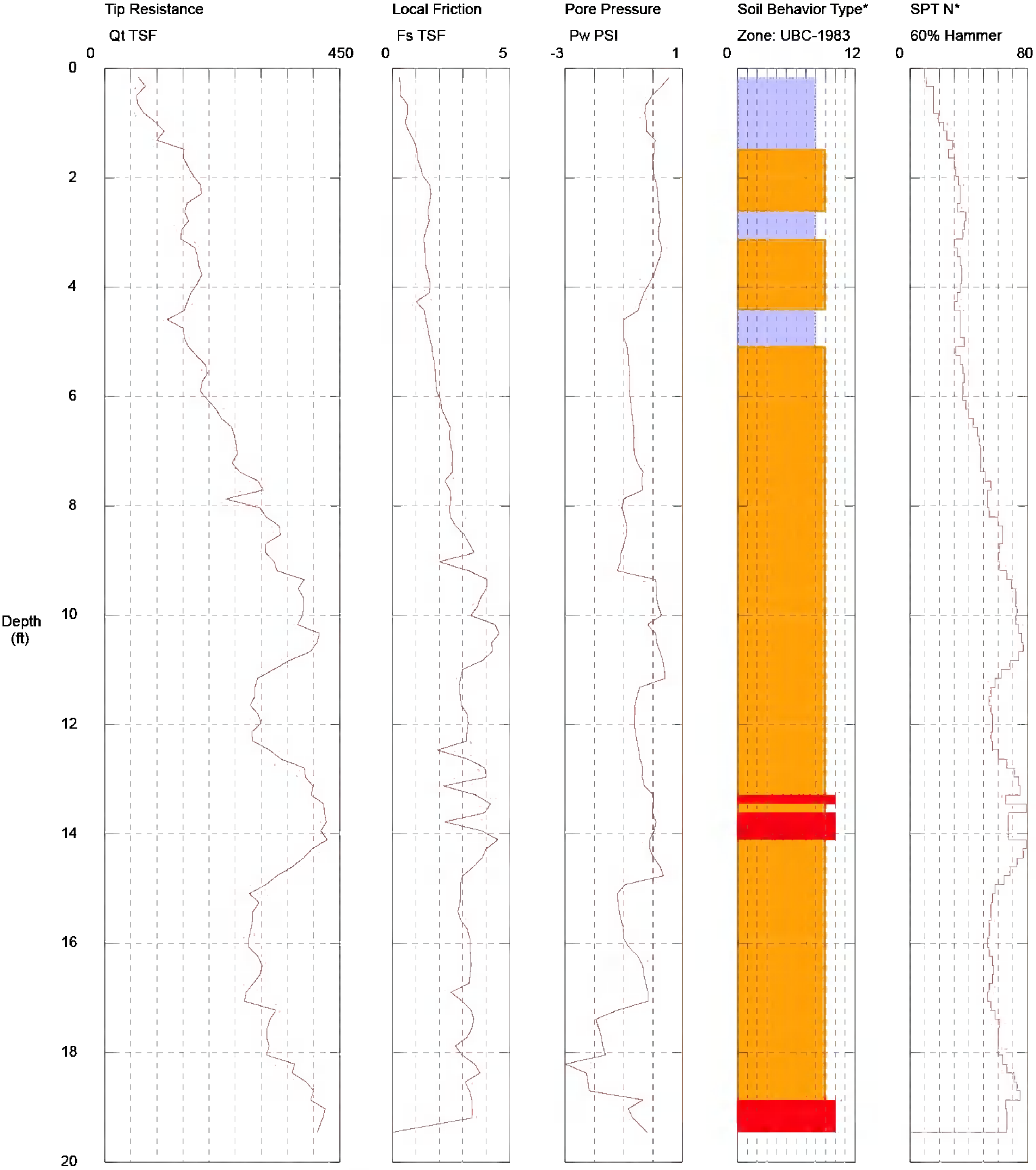
Operator: Cory Robison
Sounding: CPT-138
Cone Used: DDG1195

CPT Date/Time: 9/26/2013 10:36:07 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 19.46 feet

Depth Increment = 0.164 feet

- ▨ 1 sensitive fine grained
- ▨ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ■ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ■ 7 silty sand to sandy silt
- ■ 8 sand to silty sand
- ■ 9 sand
- 🟥 10 gravelly sand to sand
- ■ 11 very stiff fine grained (*)
- ■ 12 sand to clayey sand (*)

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*Soil behavior type and SPT based on data from UBC-1983

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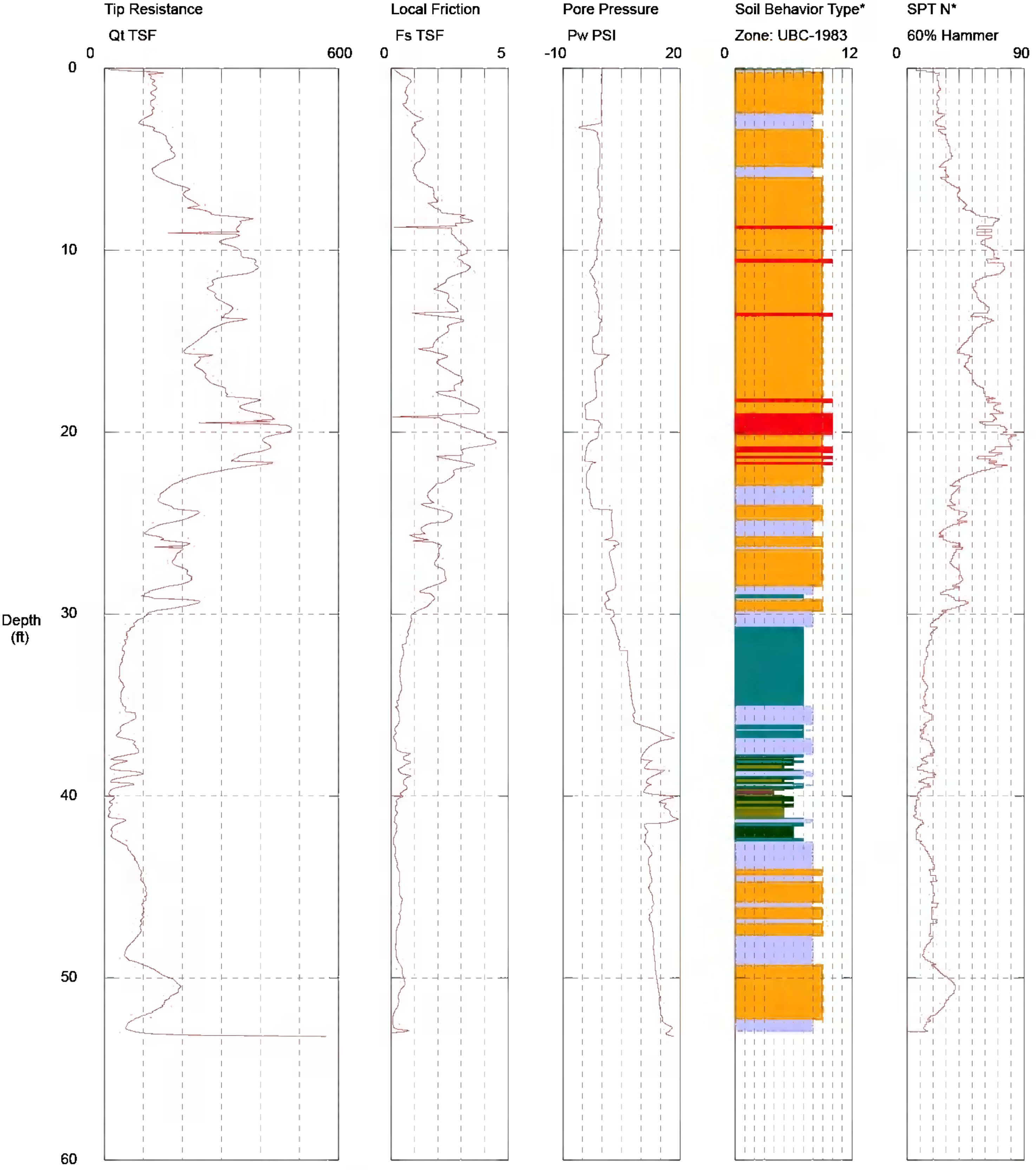
Operator: Cory Robison
Sounding: CPT-138A
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 5:53:18 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 53.22 feet

Depth Increment = 0.066 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay

■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt

■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Footer 1

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*Soil behavior type and SPT based on data from UBC-1983

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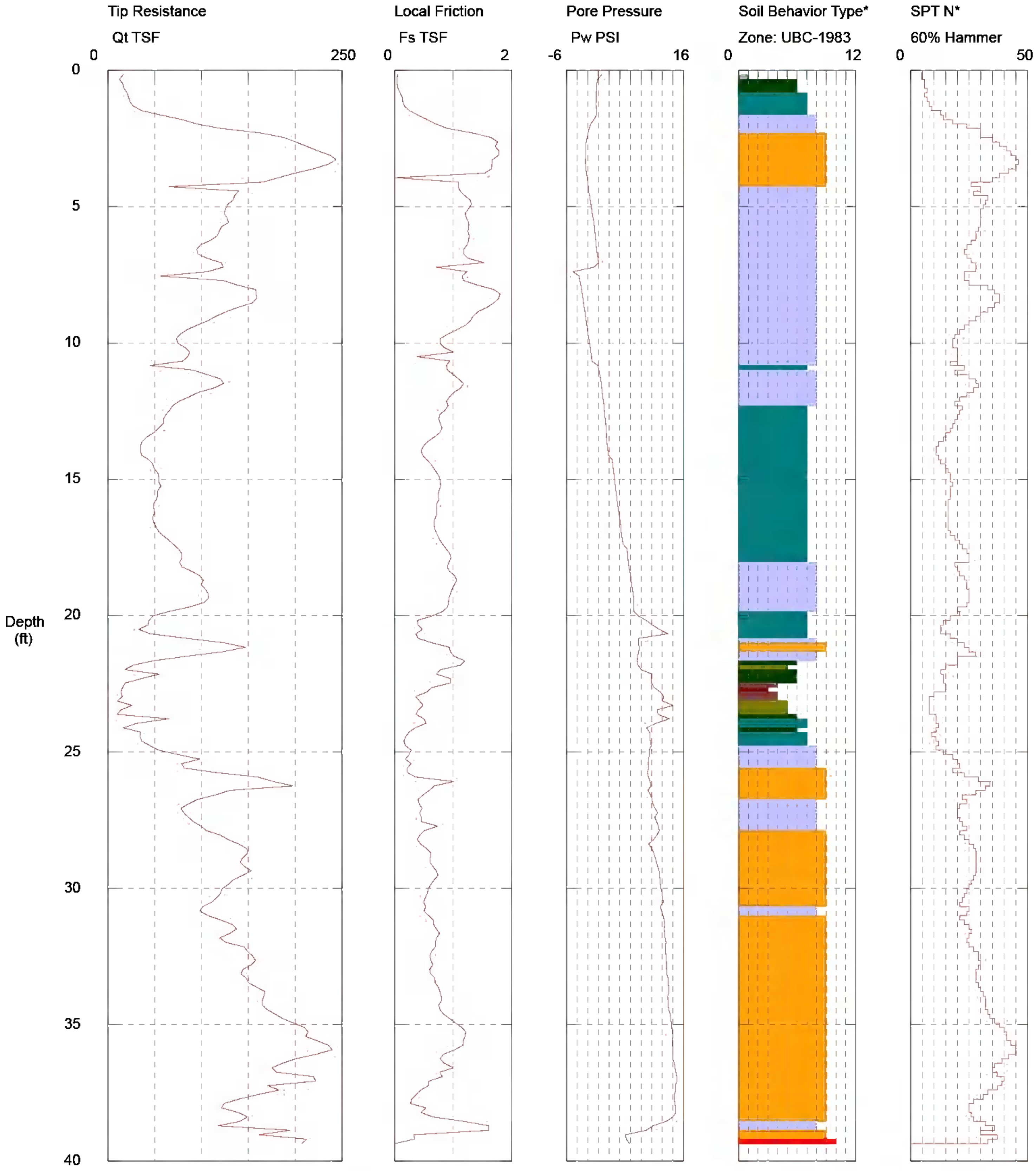
Operator: Cory Robison
Sounding: CPT-139A
Cone Used: DDG1195

CPT Date/Time: 9/26/2013 9:43:25 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 39.37 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

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*Soil behavior type and SPT based on data from UBC-1983

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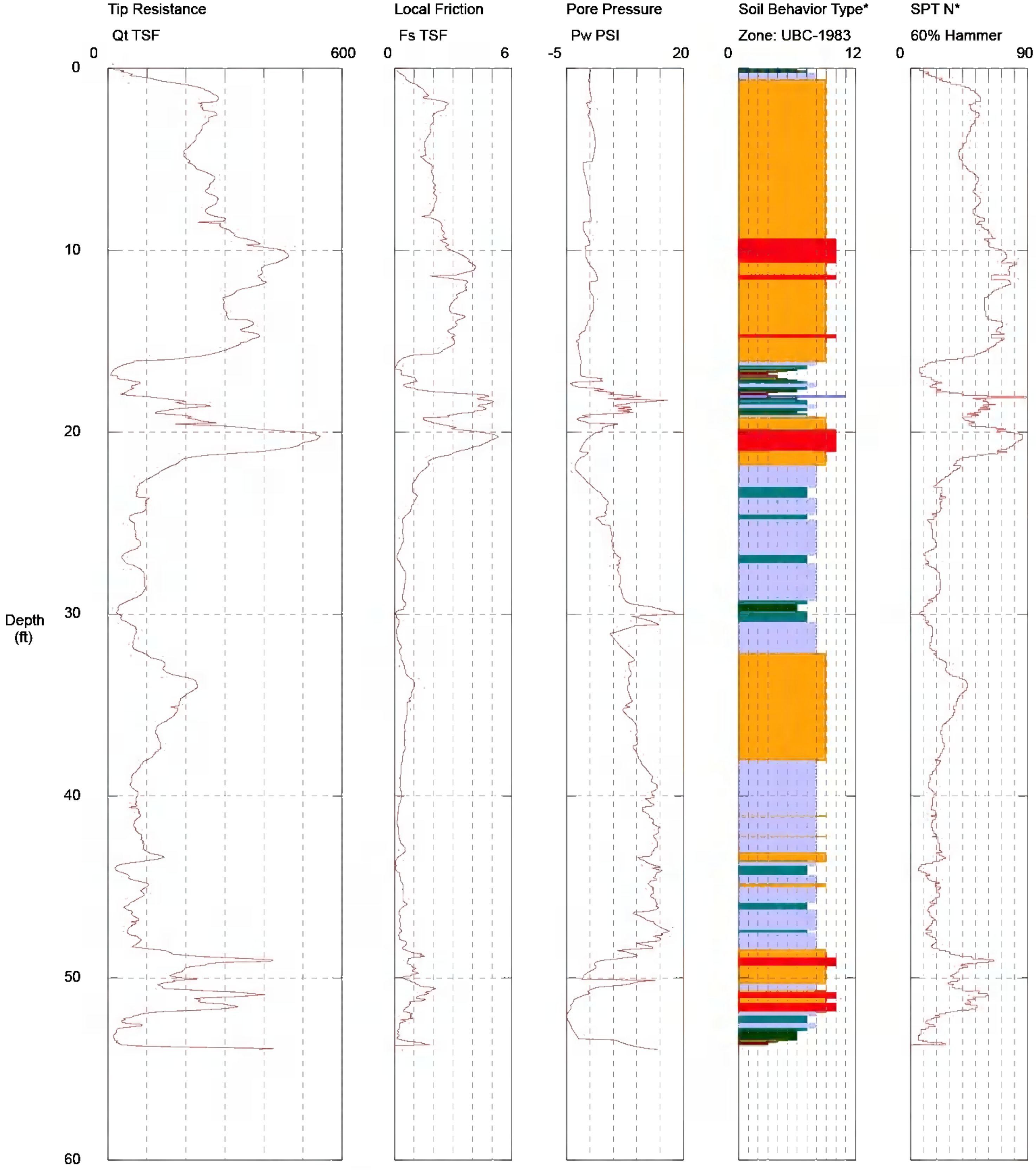
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-140
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 4:34:02 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 53.94 feet

Depth Increment = 0.066 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec
consultants
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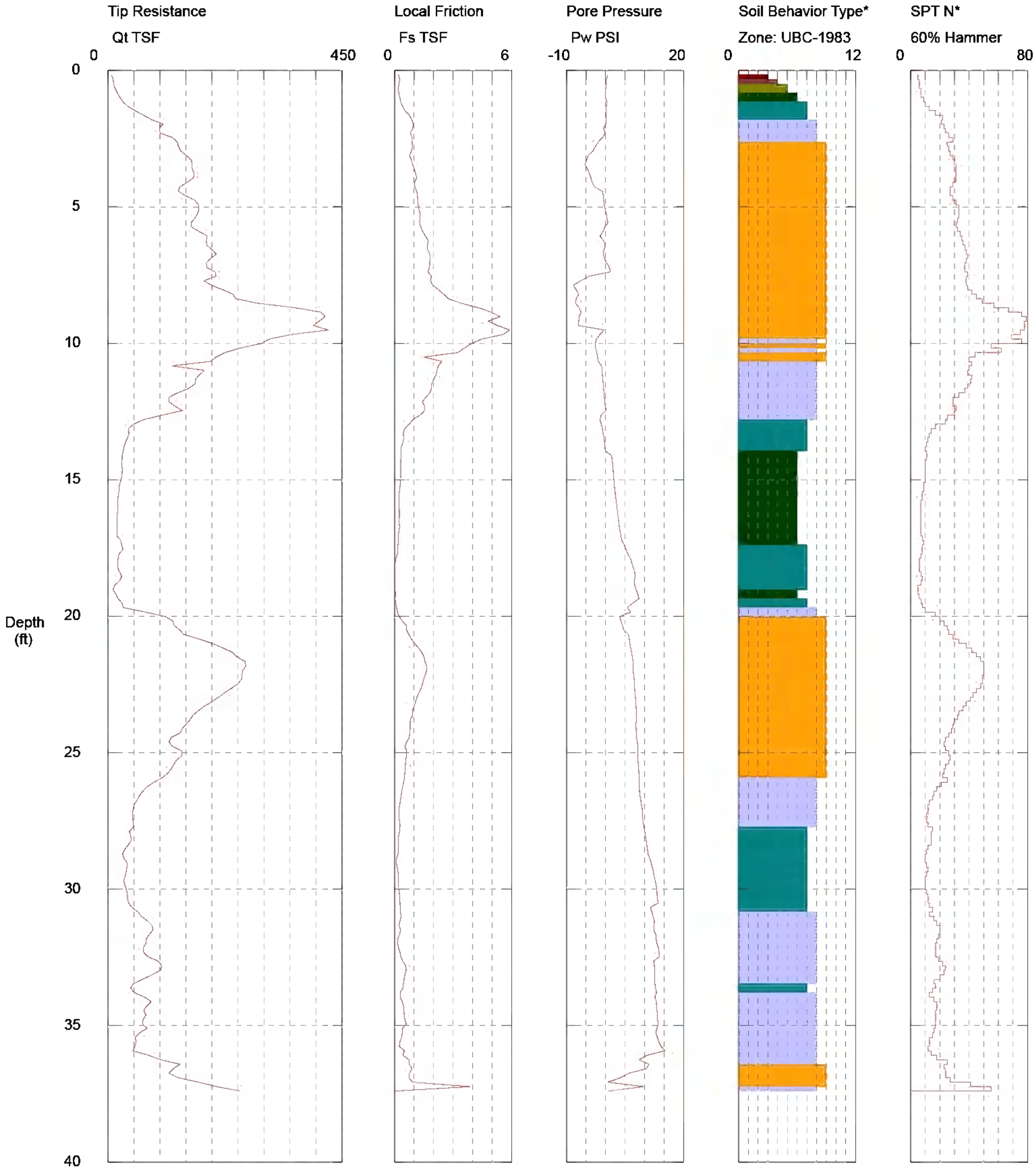
Operator: Cory Robison
Sounding: CPT-141
Cone Used: DDG1195

CPT Date/Time: 9/26/2013 8:26:32 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 37.40 feet

Depth Increment = 0.164 feet

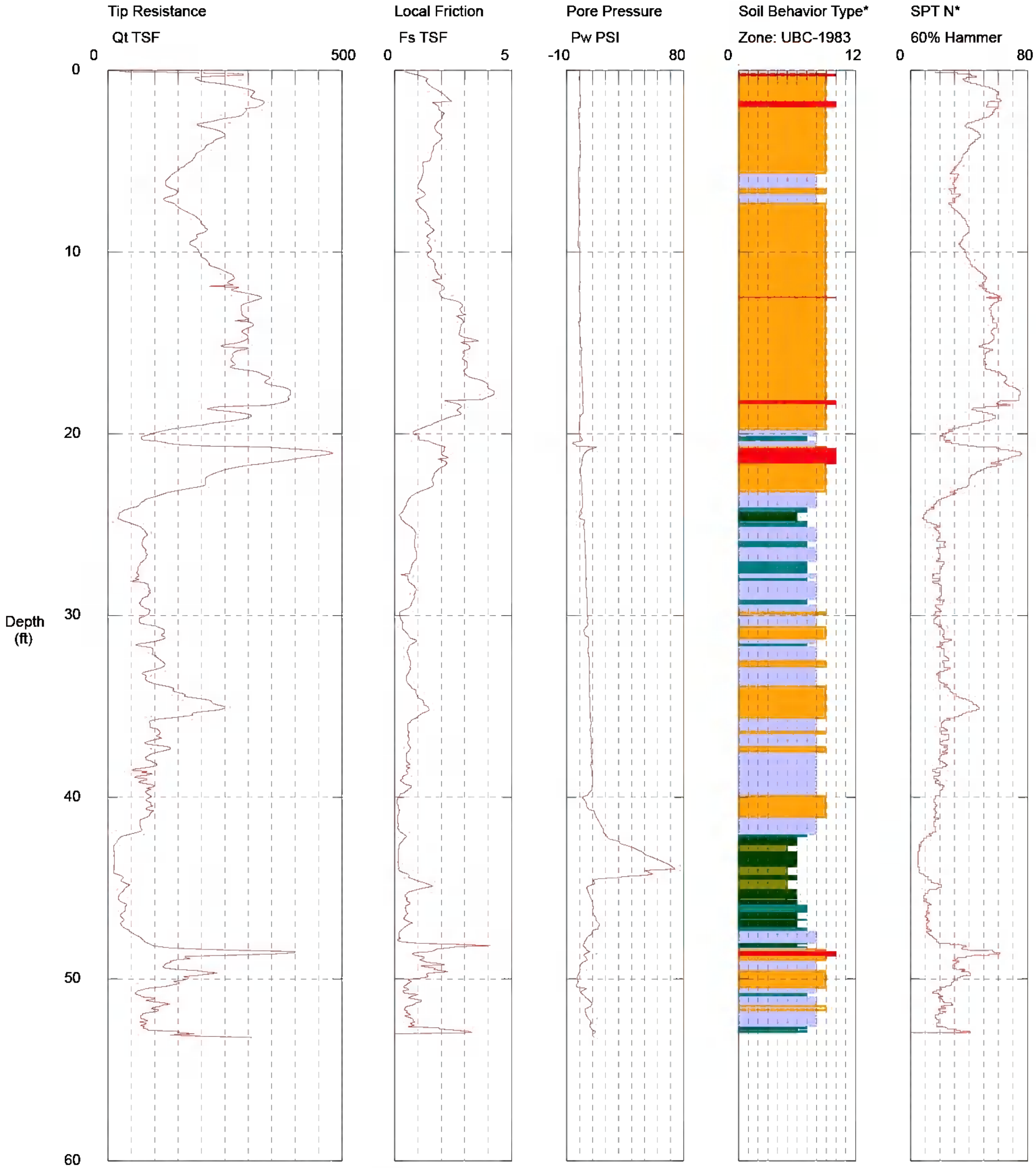
- ■ 1 sensitive fine grained
- ■ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ■ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ■ 7 silty sand to sandy silt
- ■ 8 sand to silty sand
- ■ 9 sand
- ■ 10 gravelly sand to sand
- ■ 11 very stiff fine grained (*)
- ■ 12 sand to clayey sand (*)

Geosyntec
*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-142
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 3:07:44 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 53.22 feet

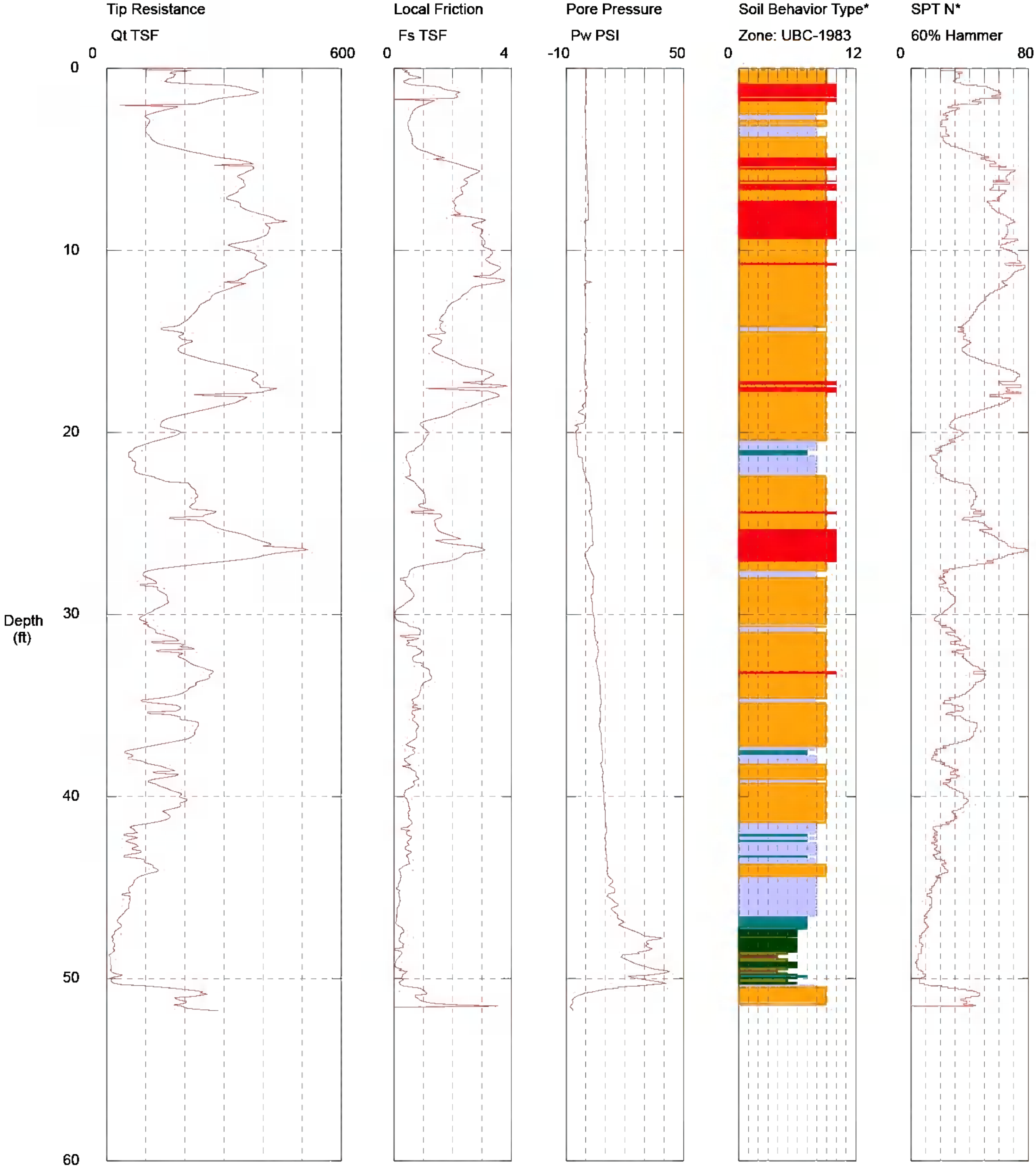
Depth Increment = 0.066 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Operator: Cory Robison
Sounding: CPT-143
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 1:34:00 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 51.77 feet

Depth Increment = 0.066 feet

⬜ 1 sensitive fine grained
⬜ 2 organic material
🟥 3 clay

🟫 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt

🟦 7 silty sand to sandy silt
🟦 8 sand to silty sand
🟧 9 sand

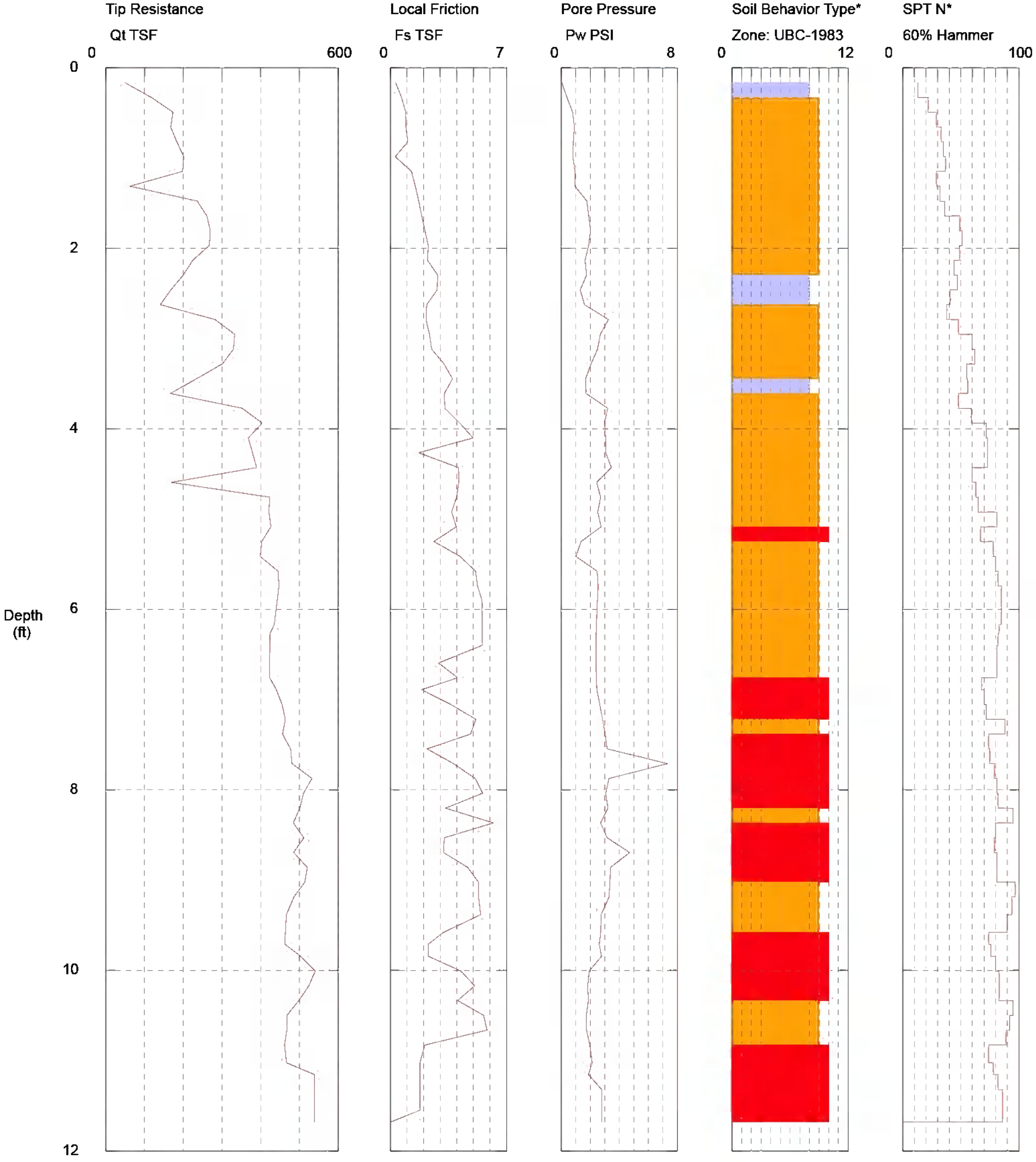
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-144
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 4:33:38 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 11.68 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Geosyntec

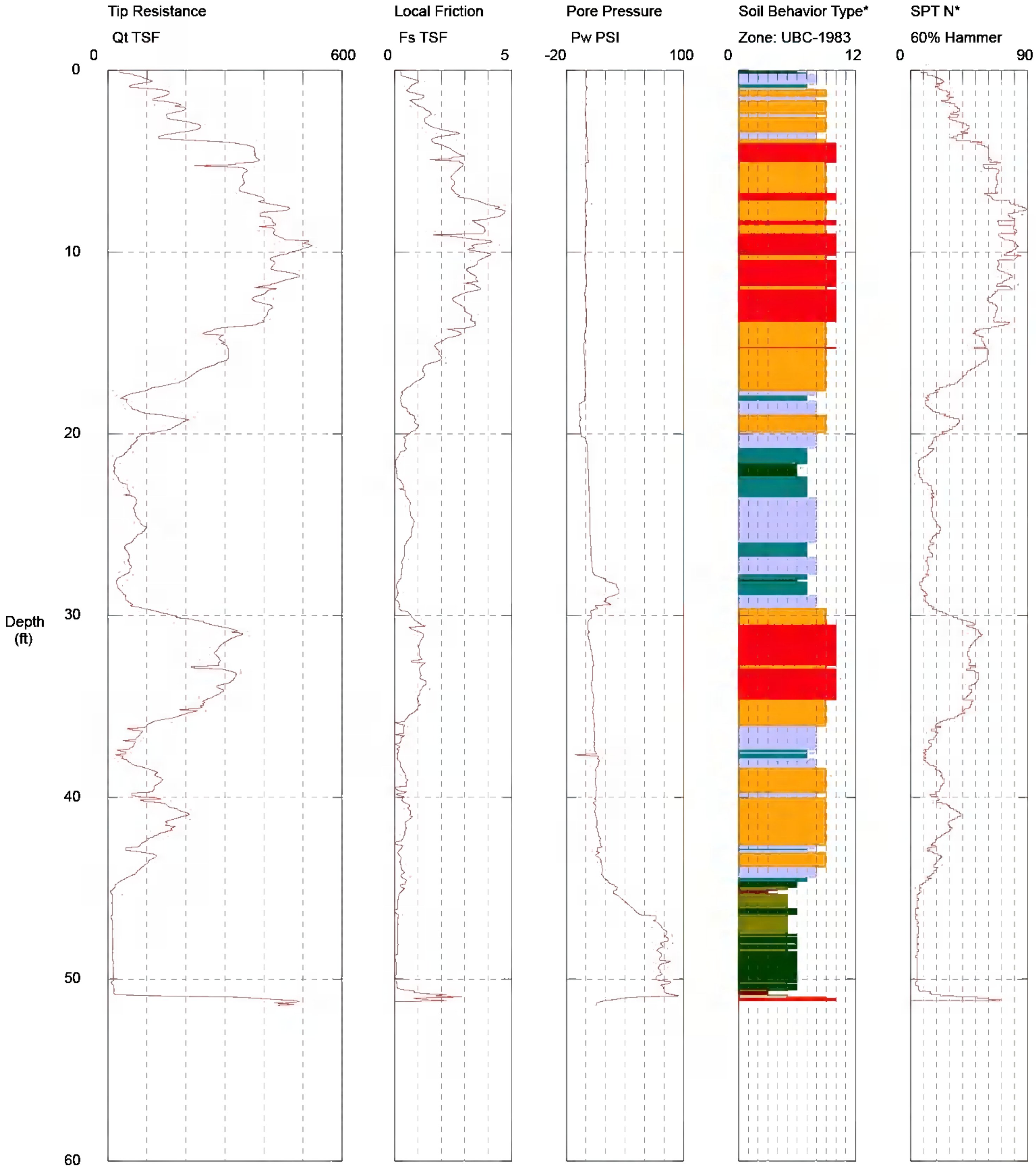
*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants

engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-144A
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 9:51:08 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 51.44 feet

Depth Increment = 0.066 feet

▨ 1 sensitive fine grained
▨ 2 organic material
▨ 3 clay
▨ 4 silty clay to clay
▨ 5 clayey silt to silty clay
▨ 6 sandy silt to clayey silt
▨ 7 silty sand to sandy silt
▨ 8 sand to silty sand
▨ 9 sand
▨ 10 gravelly sand to sand
▨ 11 very stiff fine grained (*)
▨ 12 sand to clayey sand (*)

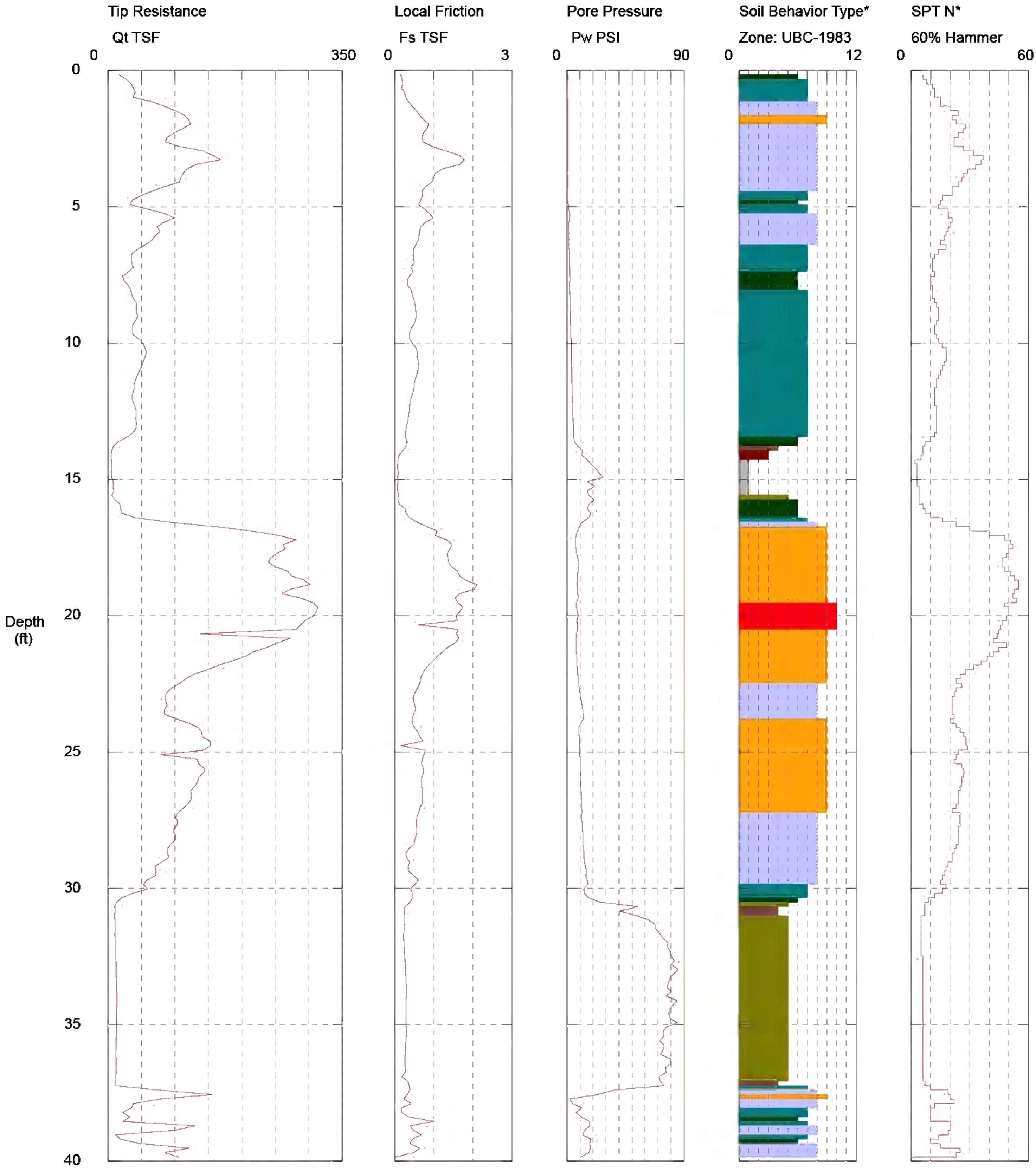
Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-145
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 5:51:31 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 39.86 feet

Depth Increment = 0.164 feet

▨ 1	sensitive fine grained
▨ 2	organic material
▨ 3	clay
▨ 4	silty clay to clay
▨ 5	clayey silt to silty clay
▨ 6	sandy silt to clayey silt
▨ 7	silty sand to sandy silt
▨ 8	sand to silty sand
▨ 9	sand
▨ 10	gravelly sand to sand
▨ 11	very stiff fine grained (*)
▨ 12	sand to clayey sand (*)

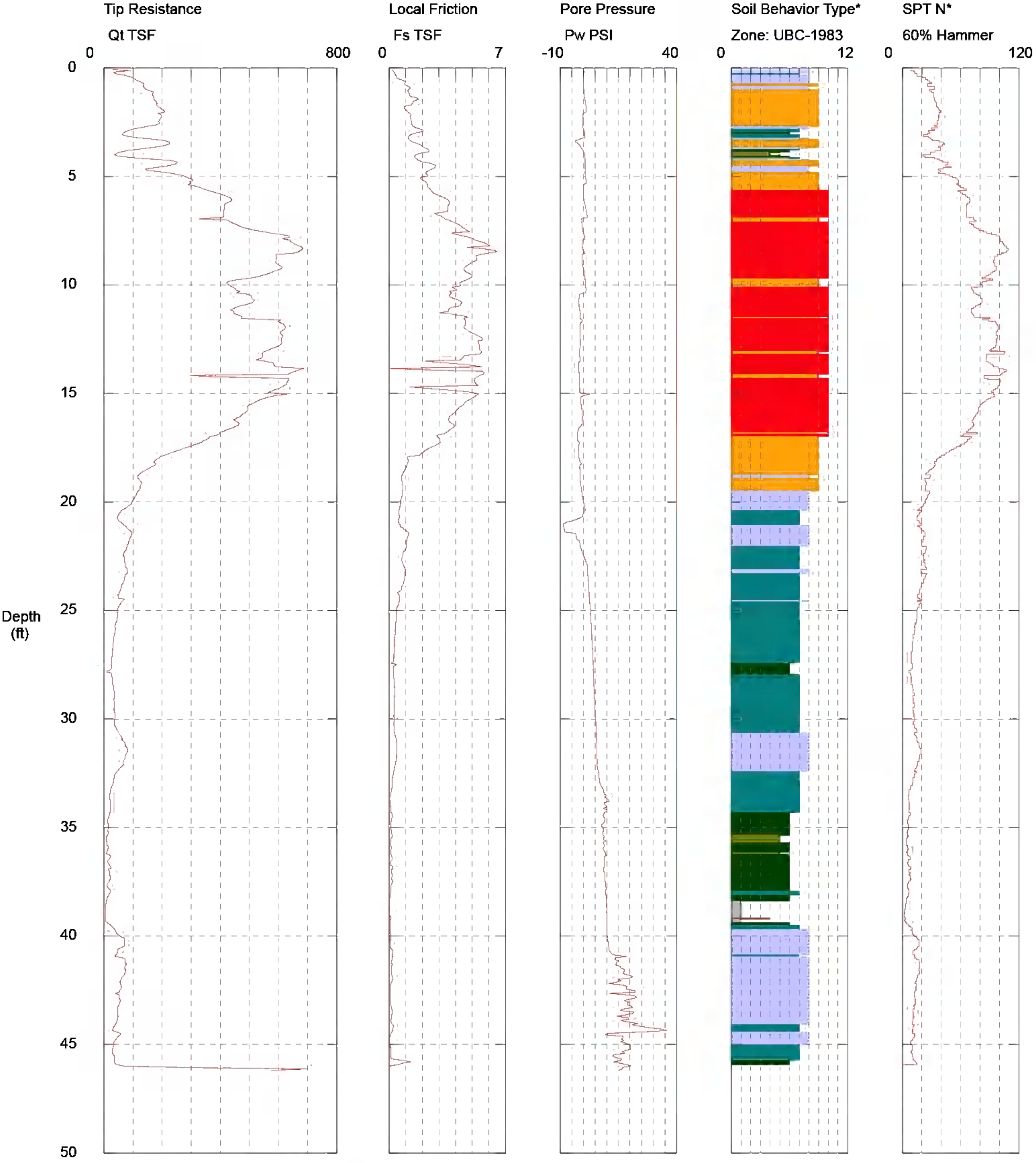
Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-146
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 8:03:00 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 46.19 feet

Depth Increment = 0.066 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay

■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt

■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
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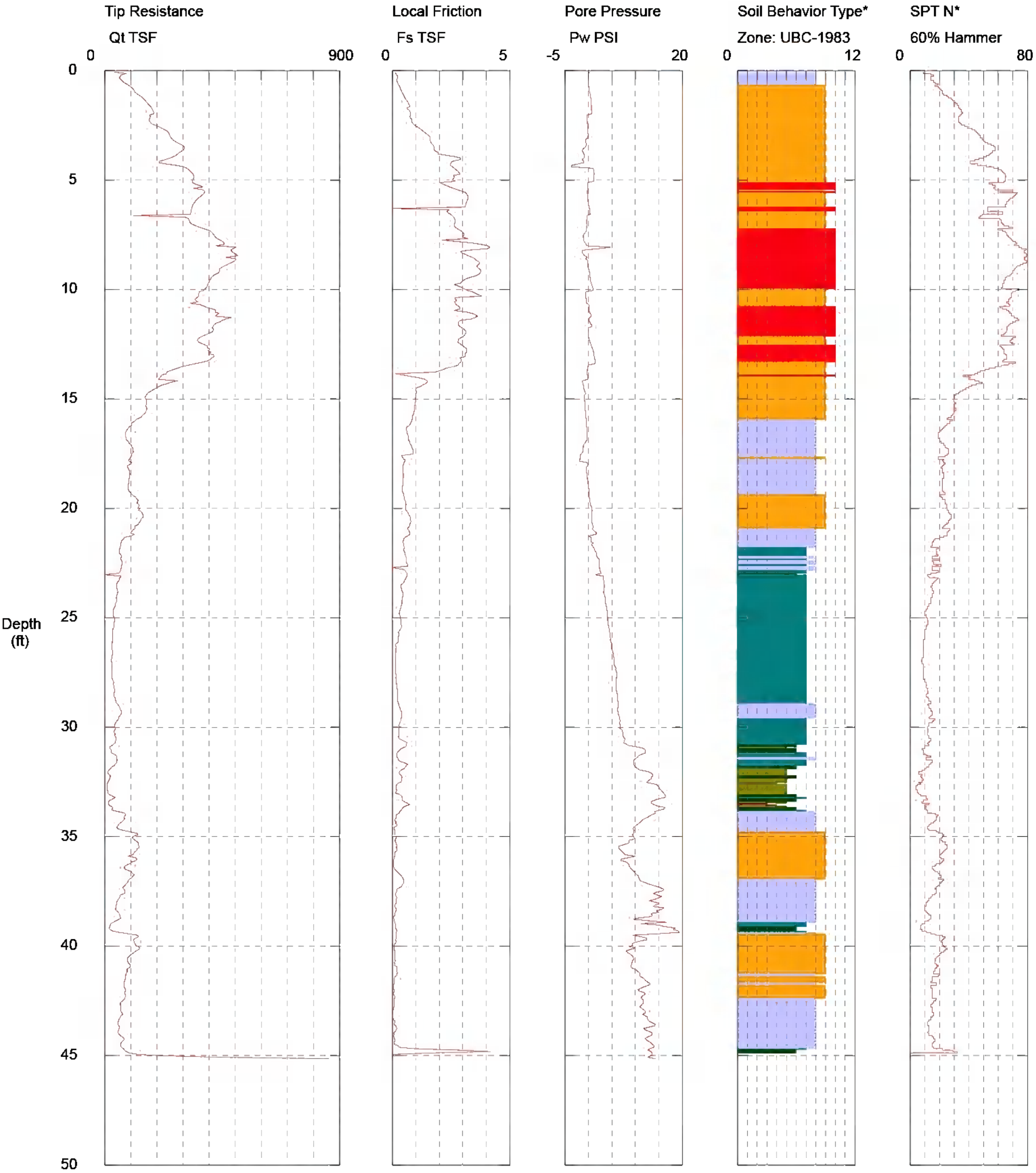
Operator: Cory Robison
Sounding: CPT-147
Cone Used: DDG1195

CPT Date/Time: 10/2/2013 5:46:15 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 45.14 feet

Depth Increment = 0.066 feet

⬜ 1 sensitive fine grained	🟫 4 silty clay to clay	🟦 7 silty sand to sandy silt	🟥 10 gravelly sand to sand
⬛ 2 organic material	🟨 5 clayey silt to silty clay	⬜ 8 sand to silty sand	🟪 11 very stiff fine grained (*)
🟥 3 clay	🟩 6 sandy silt to clayey silt	🟧 9 sand	🟦 12 sand to clayey sand (*)

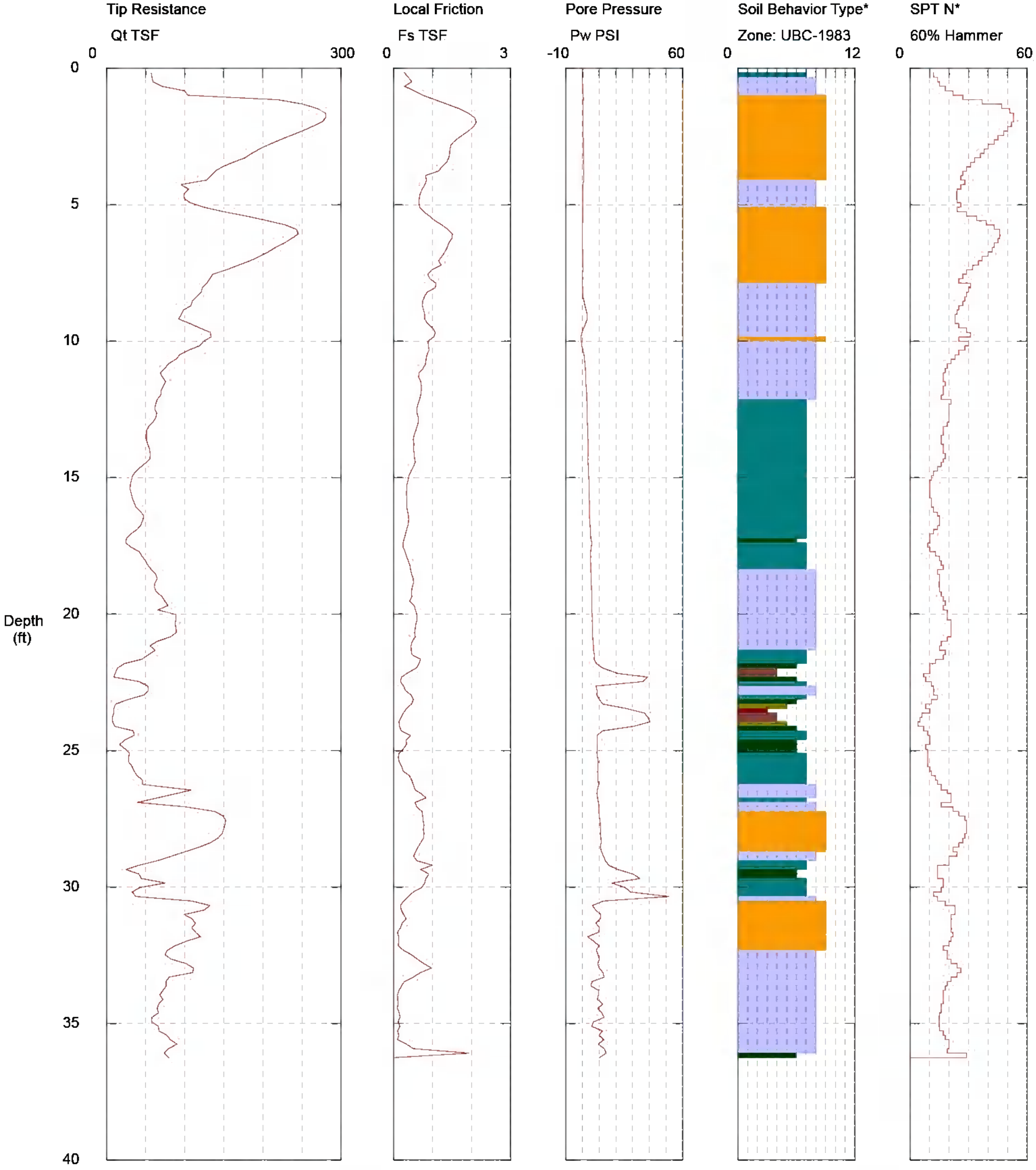
Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-148
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 8:35:58 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 36.25 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec
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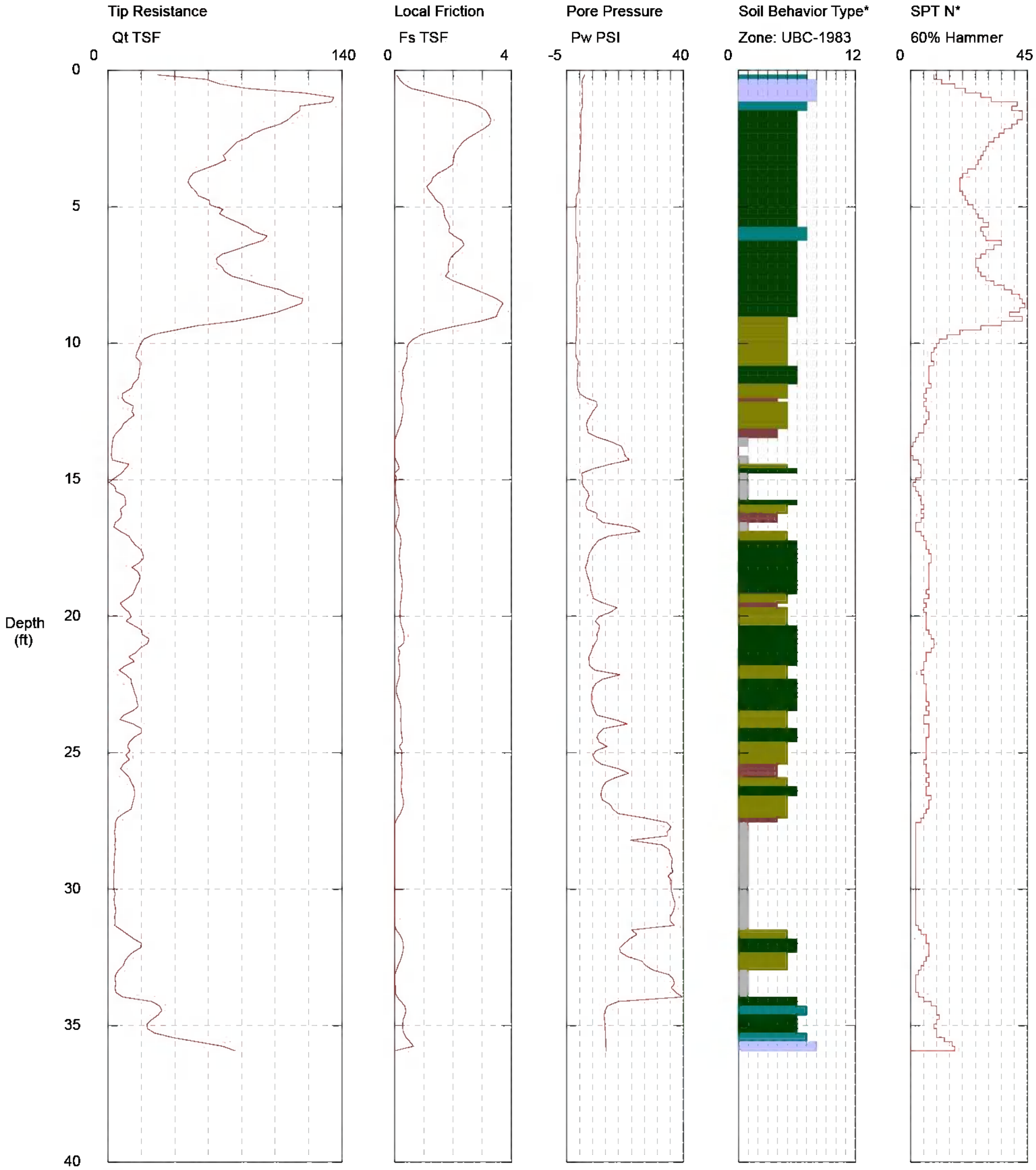
Operator: Cory Robison
Sounding: CPT-155
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 5:48:34 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 35.93 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants
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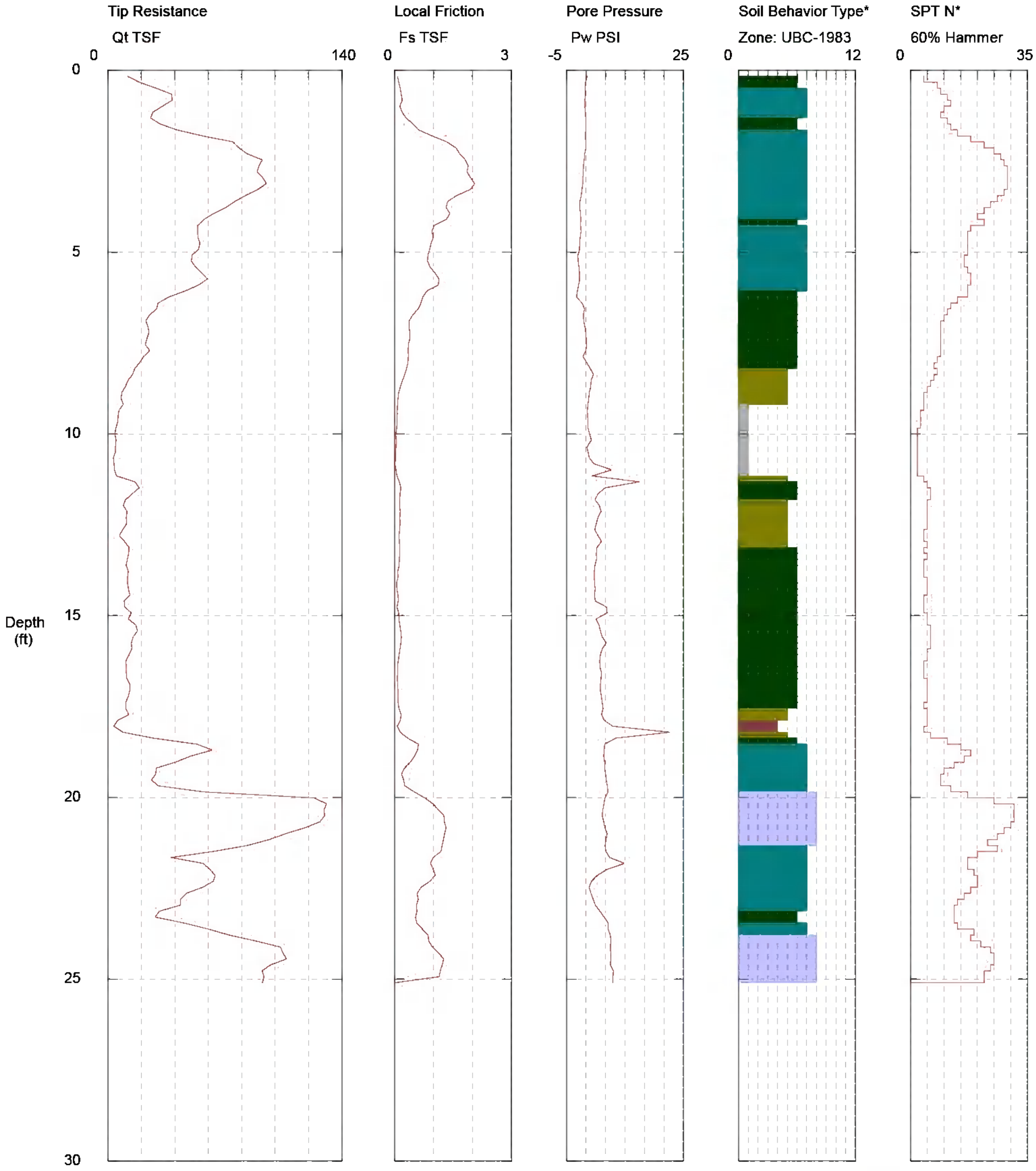
Operator: Cory Robison
Sounding: CPT-156
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 5:15:31 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 25.10 feet

Depth Increment = 0.164 feet

- ■ 1 sensitive fine grained
- ■ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ■ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ■ 7 silty sand to sandy silt
- ■ 8 sand to silty sand
- ■ 9 sand
- ■ 10 gravelly sand to sand
- ■ 11 very stiff fine grained (*)
- ■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

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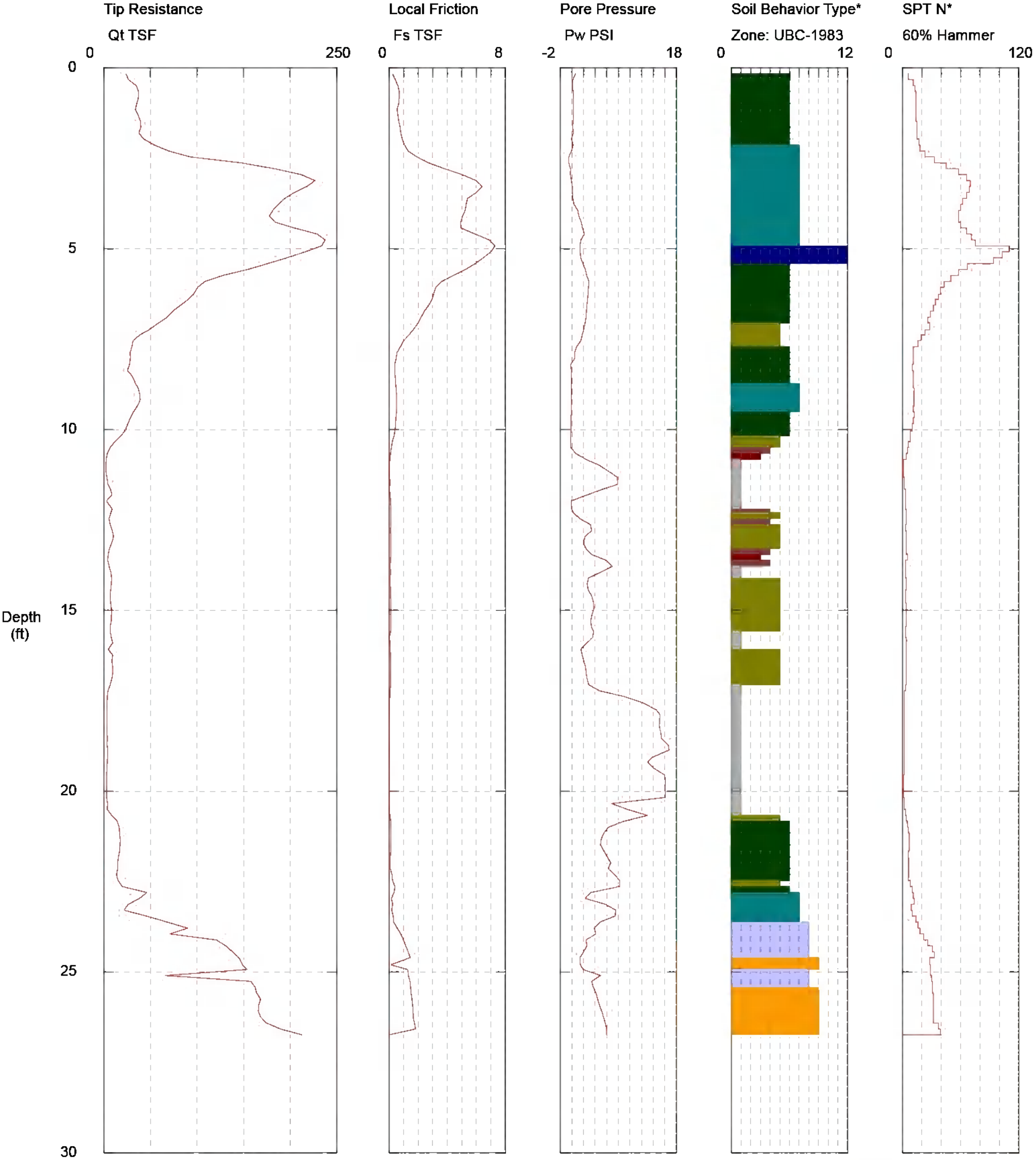
Operator: Cory Robison
Sounding: CPT-157a
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 3:21:59 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 26.74 feet

Depth Increment = 0.164 feet

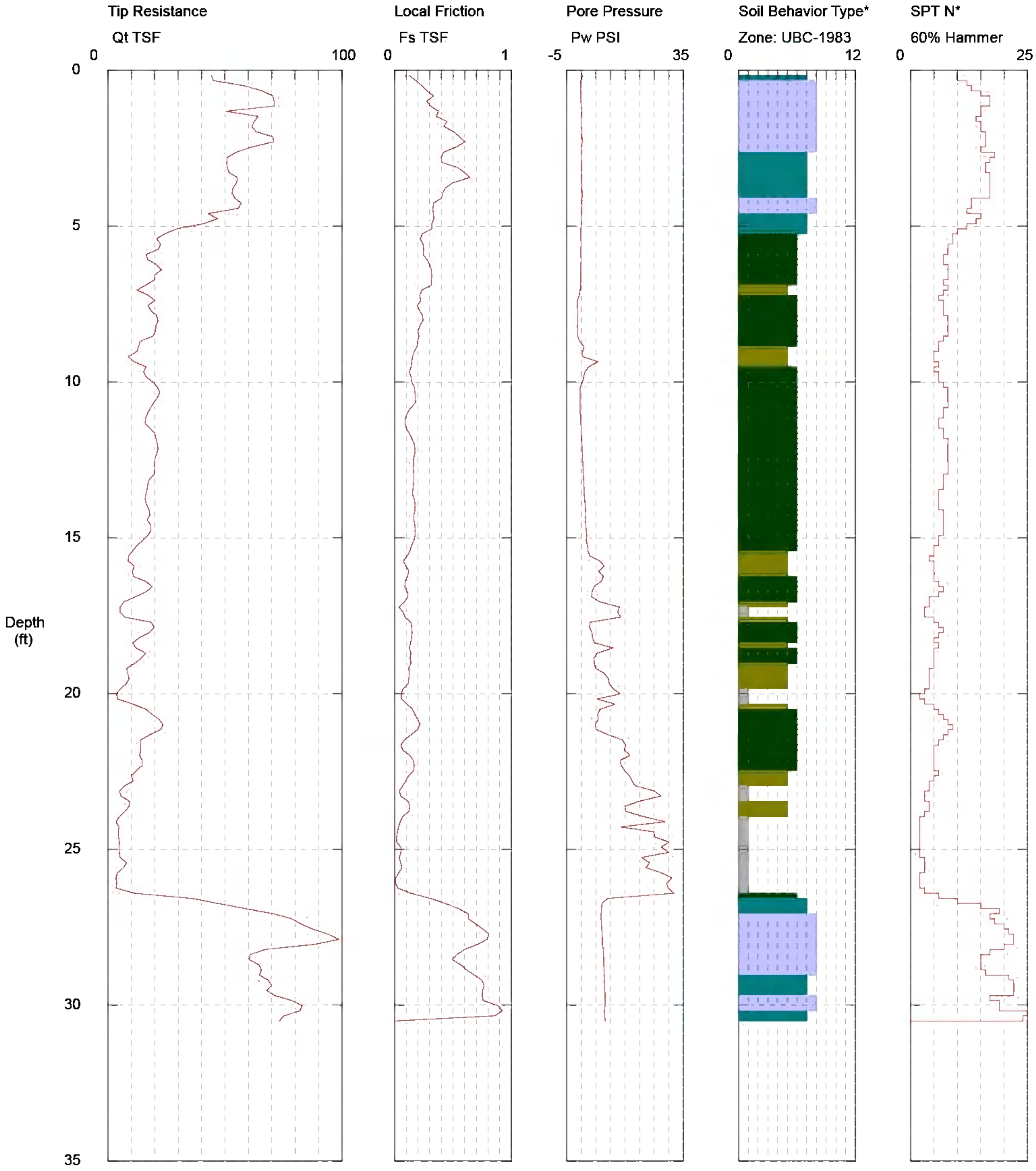
■ 1 sensitive fine grained	■ 4 silty clay to clay	■ 7 silty sand to sandy silt	■ 10 gravelly sand to sand
■ 2 organic material	■ 5 clayey silt to silty clay	■ 8 sand to silty sand	■ 11 very stiff fine grained (*)
■ 3 clay	■ 6 sandy silt to clayey silt	■ 9 sand	■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

Operator: Cory Robison
Sounding: CPT-158
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 4:26:24 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 30.51 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

Geosyntec

*Soil behavior type and SPT based on data from UBC-1983

CPT LOG NO. CPT-209

Page 1 of 1

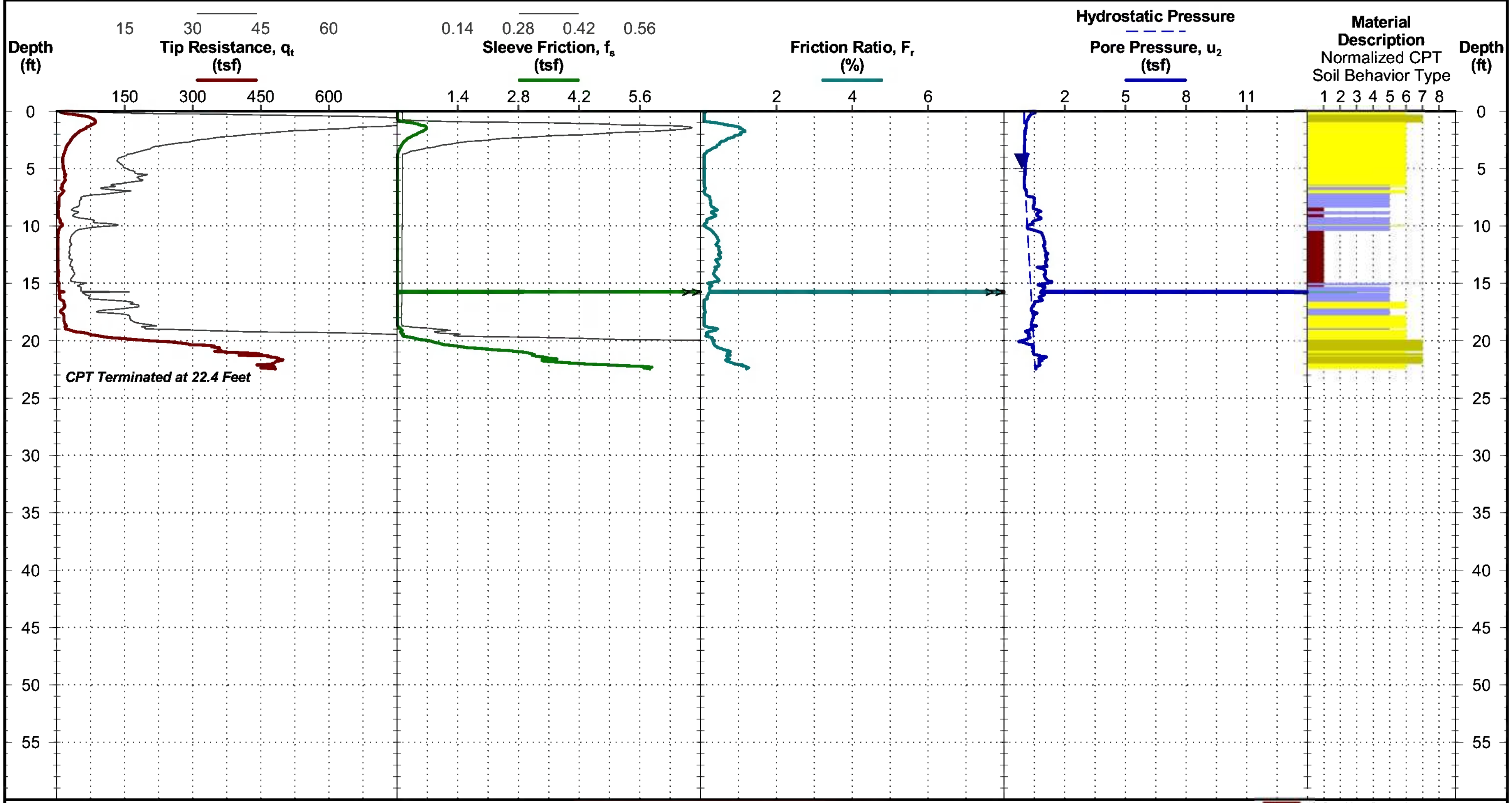
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.3508°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
CPT Completed: 3/22/2016
Rig: Pagani TG73-200
Operator: BR
Project No.: EN165065
Exhibit: A-4

CPT LOG NO. CPT-210

Page 1 of 1

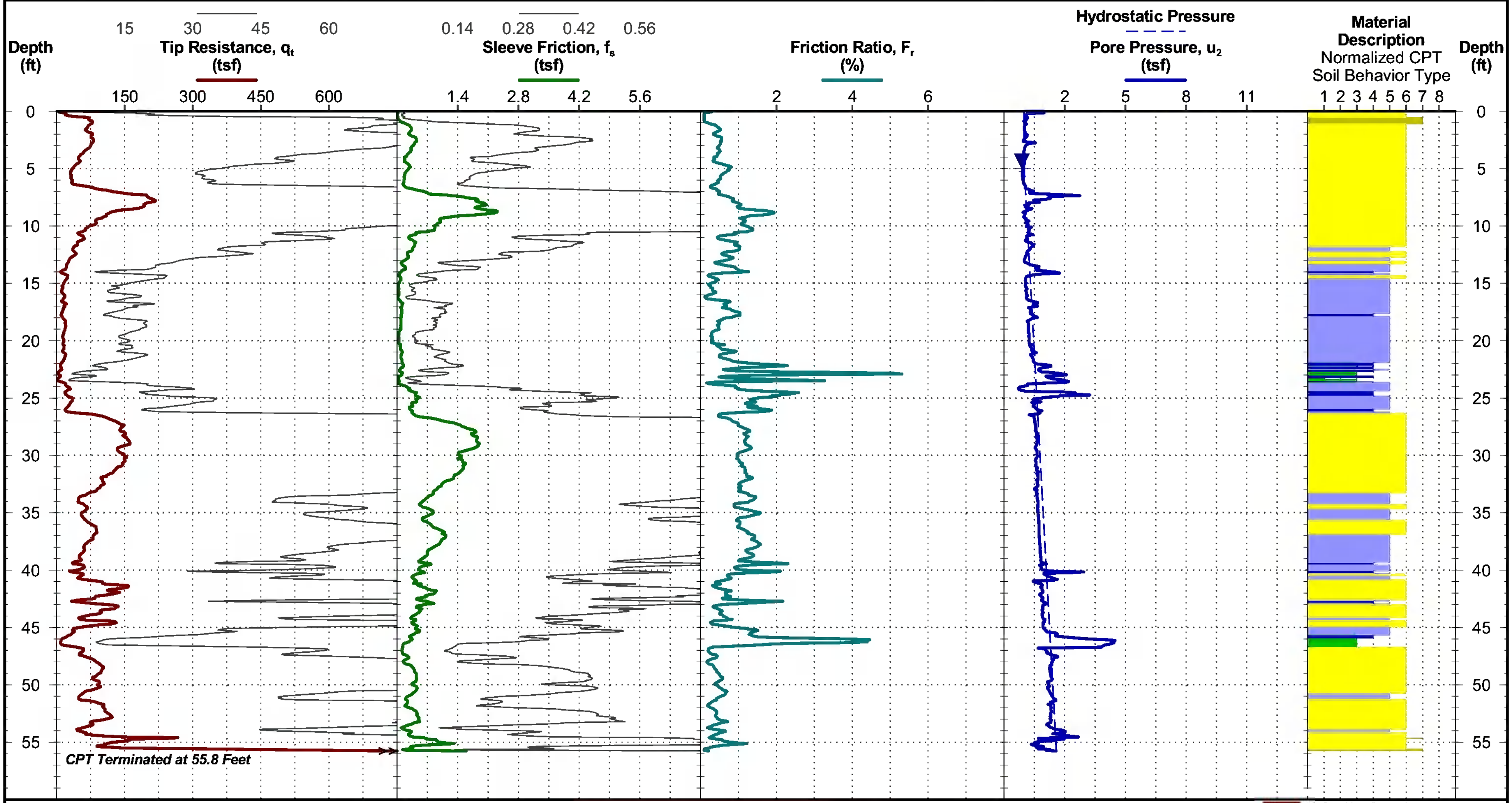
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.3485°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟦 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016	CPT Completed: 3/22/2016
Rig: Pagani TG73-200	Operator: BR
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. CPT-211

Page 1 of 2

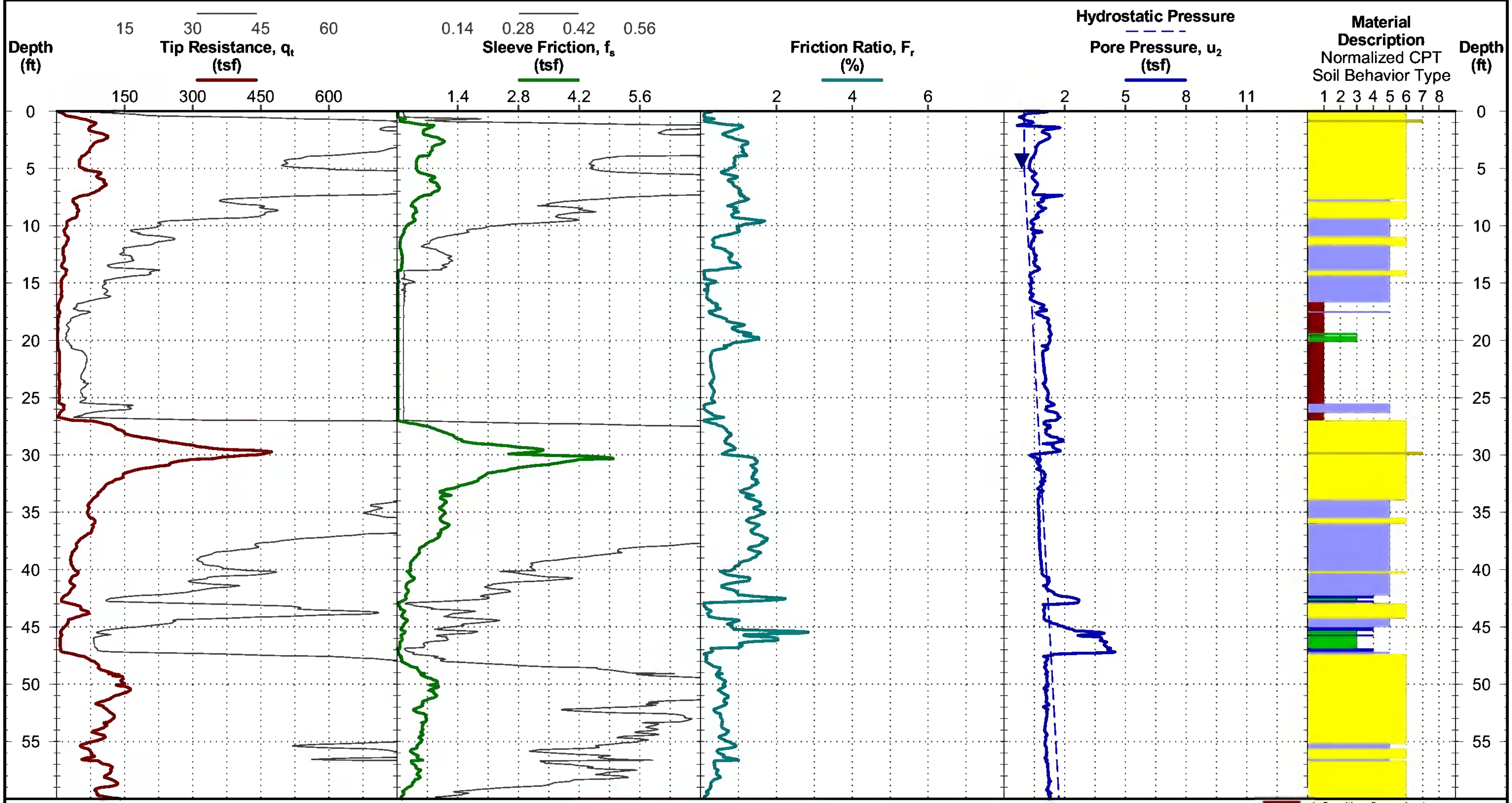
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.3459°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

🟥	1 Sensitive, fine grained
🟧	2 Organic soils - clay
🟩	3 Clay - silty clay to clay
🟦	4 Silt mixtures - clayey silt to silty clay
🟪	5 Sand mixtures - silty sand to sandy silt
🟨	6 Sands - clean sand to silty sand
🟨	7 Gravelly sand to dense sand
🟫	8 Very stiff sand to clayey sand
⬛	9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/22/2016
Operator: BR
Exhibit: A-4

CPT LOG NO. CPT-211

Page 2 of 2

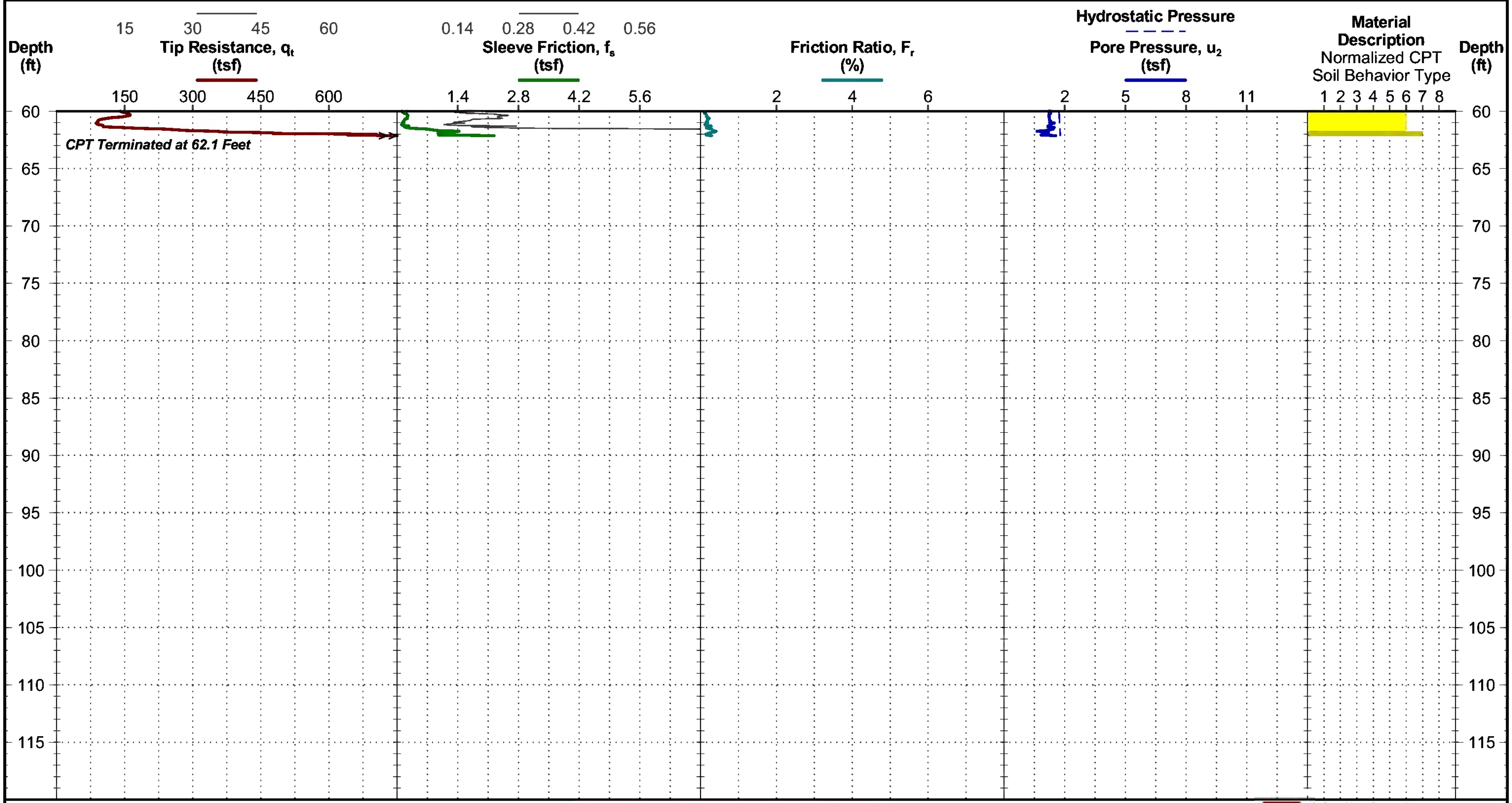
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.3459°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟦 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016	CPT Completed: 3/22/2016
Rig: Pagani TG73-200	Operator: BR
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. CPT-212

Page 1 of 1

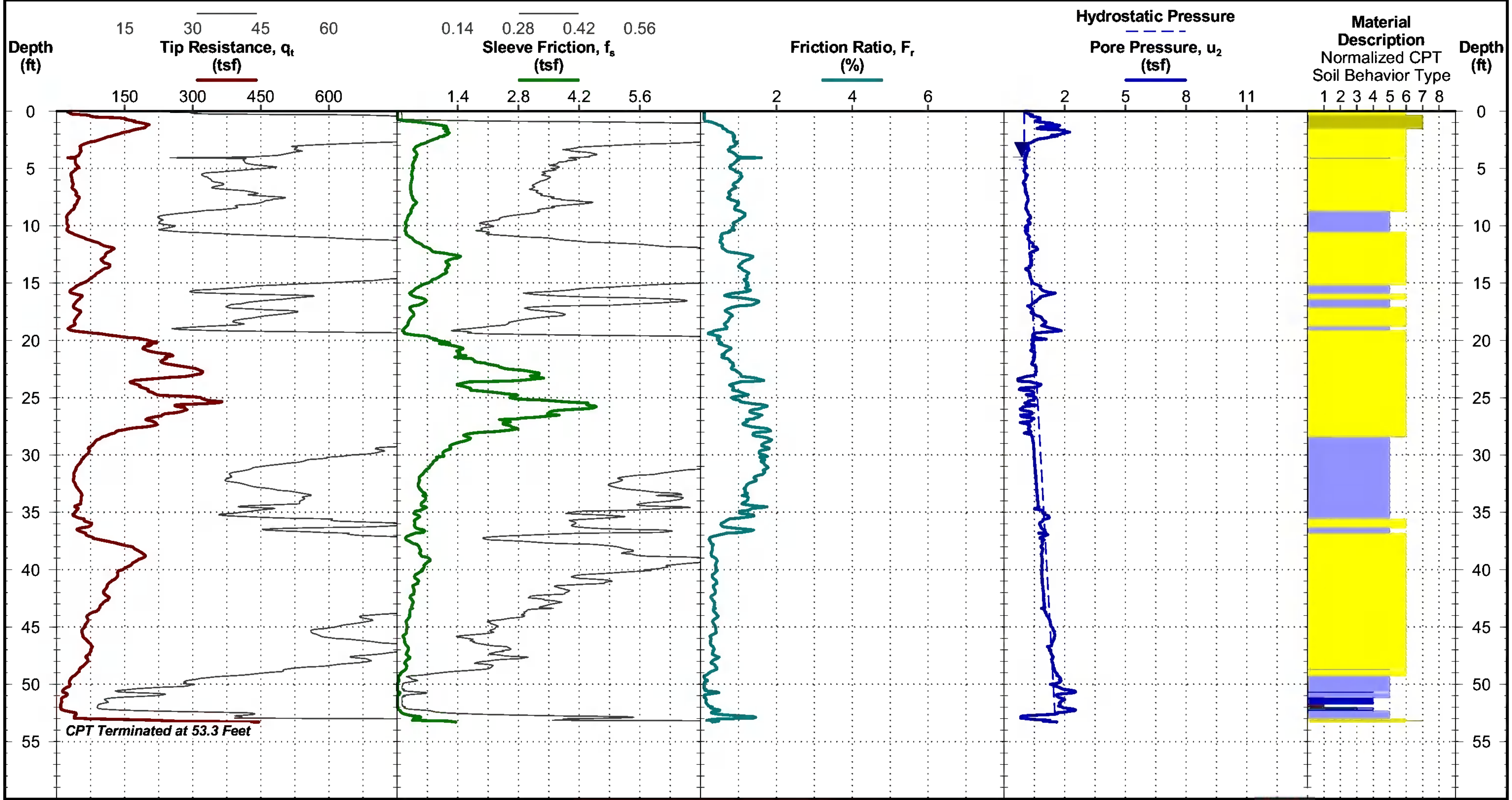
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3259°
Longitude: -79.352°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 4 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/22/2016
Operator: JB
Exhibit: A-4

CPT LOG NO. SCPT-213

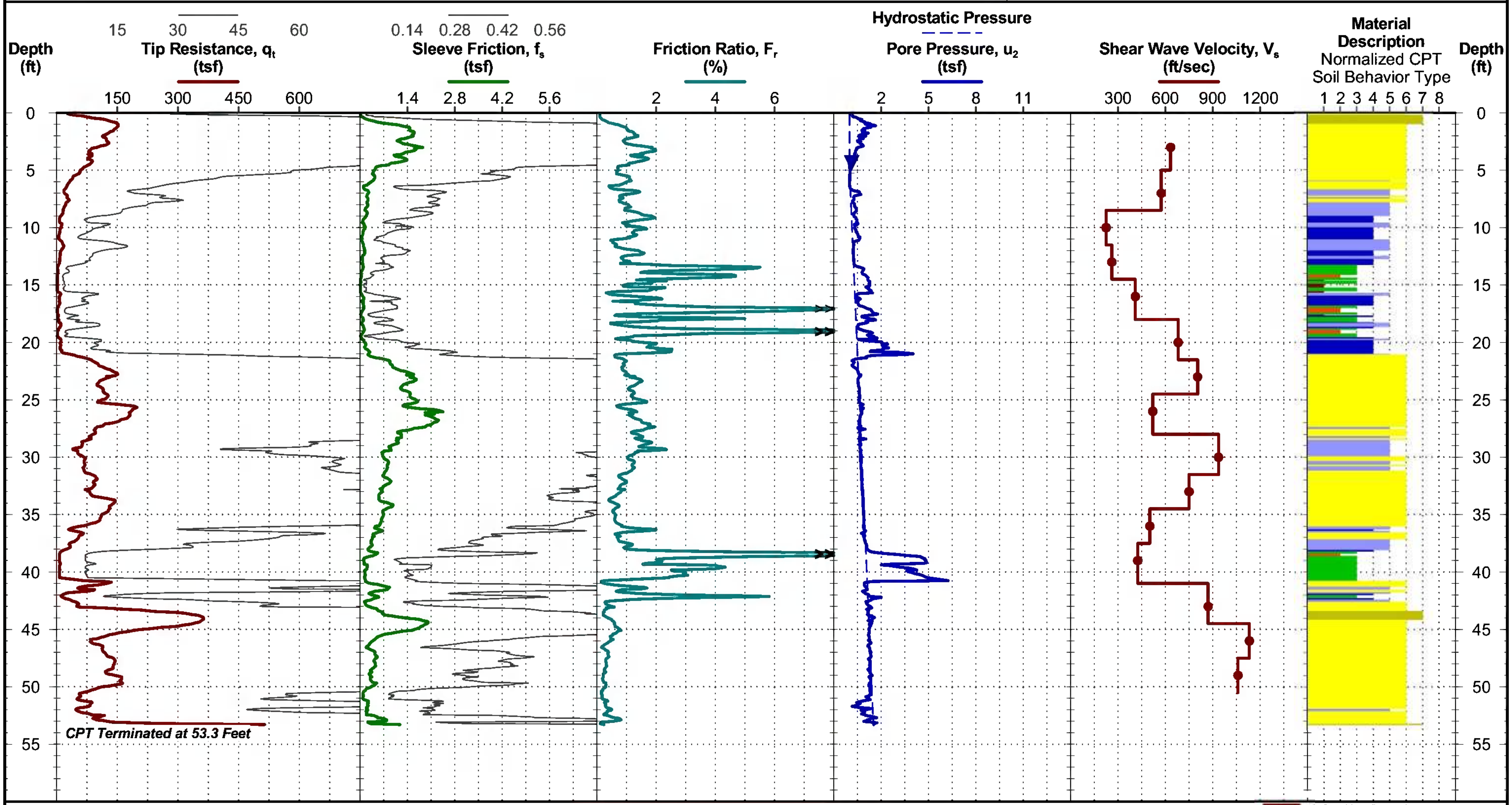
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3267°
Longitude: -79.3499°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

🟥 1 Sensitive, fine grained
🟧 2 Organic soils - clay
🟩 3 Clay - silty clay to clay
🟦 4 Silt mixtures - clayey silt to silty clay
🟦 5 Sand mixtures - silty sand to sandy silt
🟨 6 Sands - clean sand to silty sand
🟨 7 Gravelly sand to dense sand
🟫 8 Very stiff sand to clayey sand
🟪 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/22/2016
Operator: JB
Exhibit: A-4

CPT LOG NO. SCPT-214

Page 1 of 1

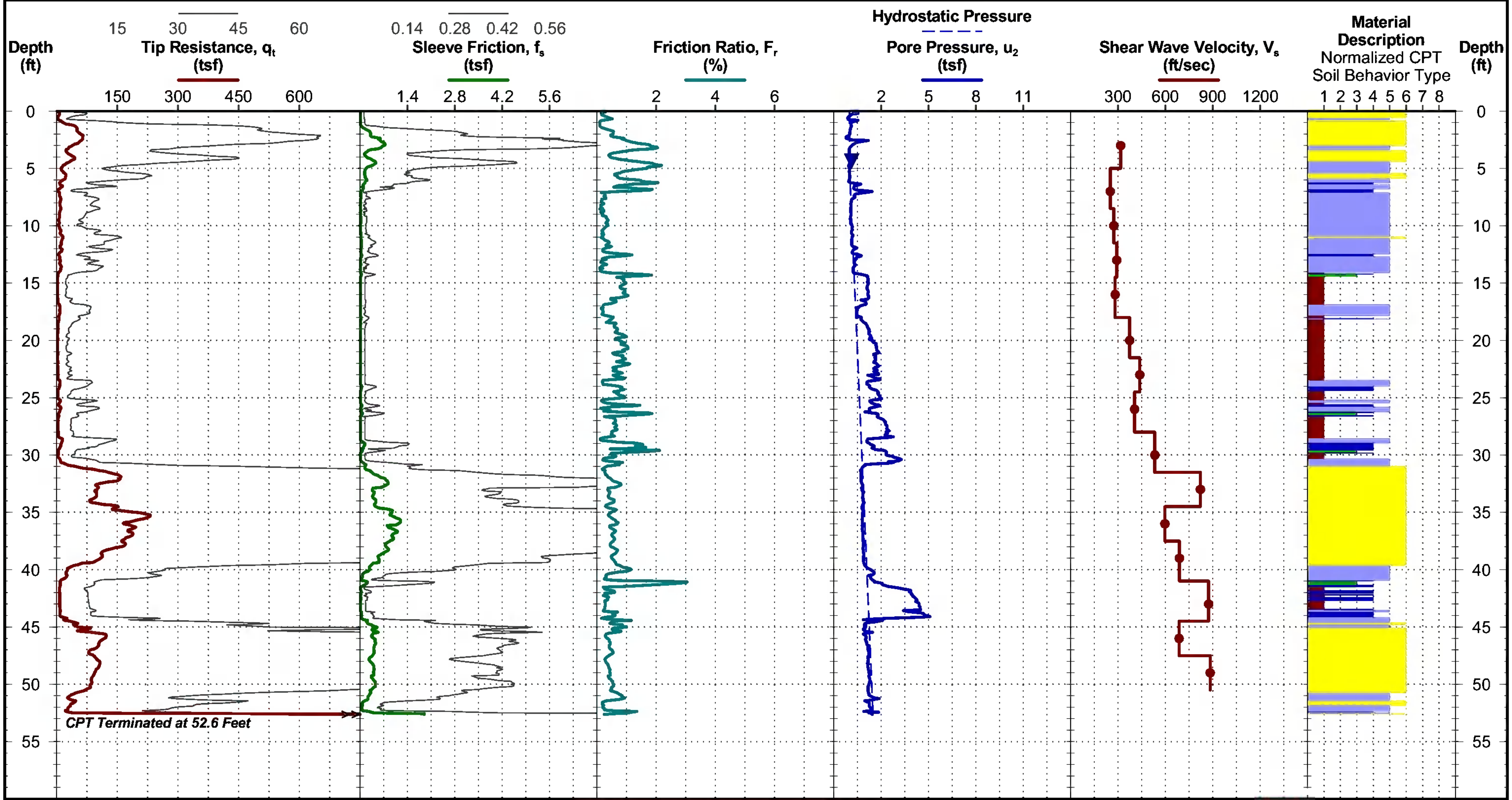
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3265°
Longitude: -79.3472°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016	CPT Completed: 3/22/2016
Rig: Pagani TG73-200	Operator: BR
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. CPT-215

Page 1 of 1

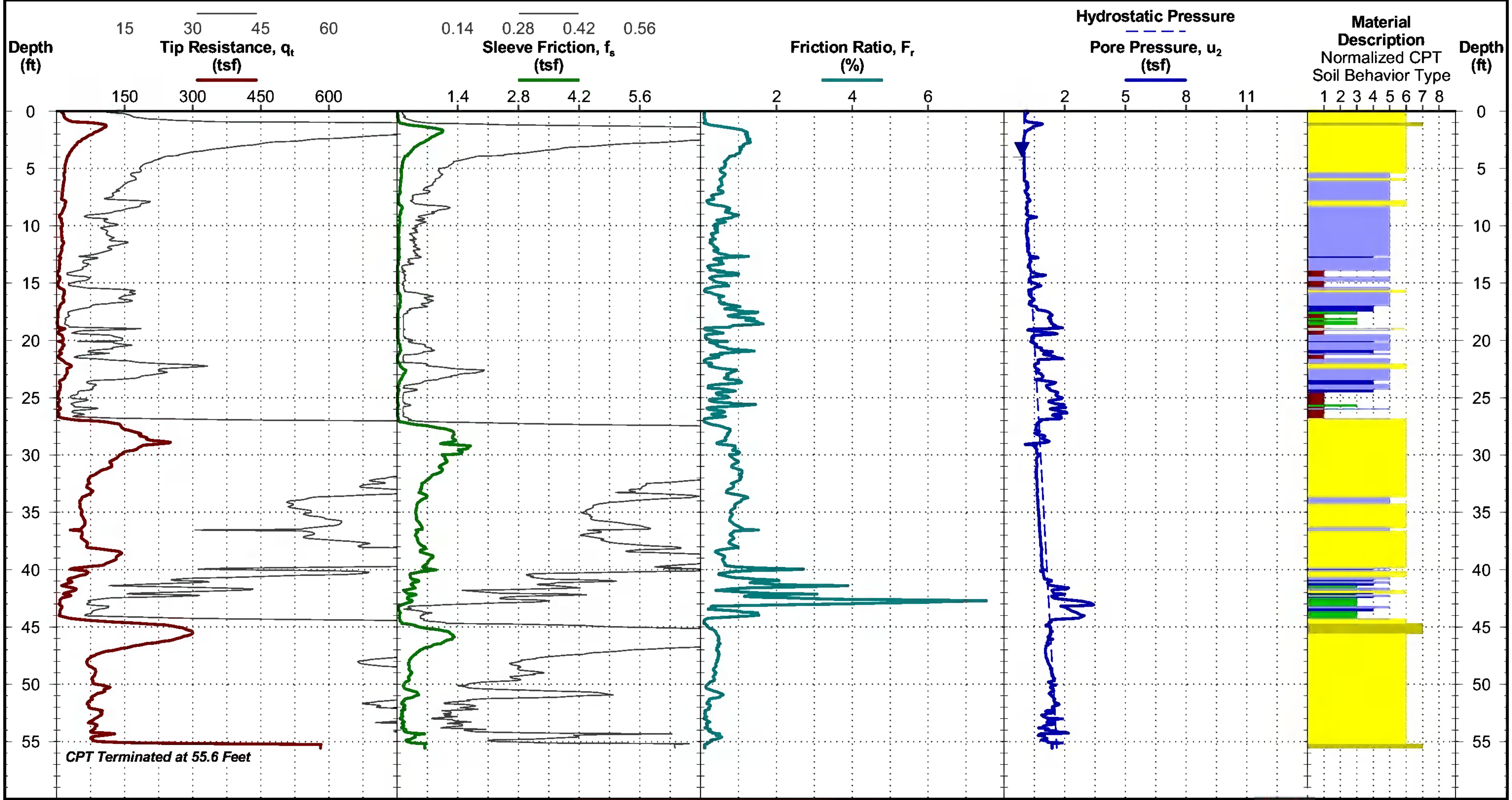
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3256°
Longitude: -79.3498°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

🟥	1	Sensitive, fine grained
🟧	2	Organic soils - clay
🟩	3	Clay - silty clay to clay
🟦	4	Silt mixtures - clayey silt to silty clay
🟪	5	Sand mixtures - silty sand to sandy silt
🟨	6	Sands - clean sand to silty sand
▨	7	Gravelly sand to dense sand
▨	8	Very stiff sand to clayey sand
▨	9	Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 4 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/22/2016
Operator: JB
Exhibit: A-4

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. CPT REPORT EN165065 WINYAH GENERATION STATION.GPJ TERRACON2015.GDT 5/11/16

CPT LOG NO. SCPT-216

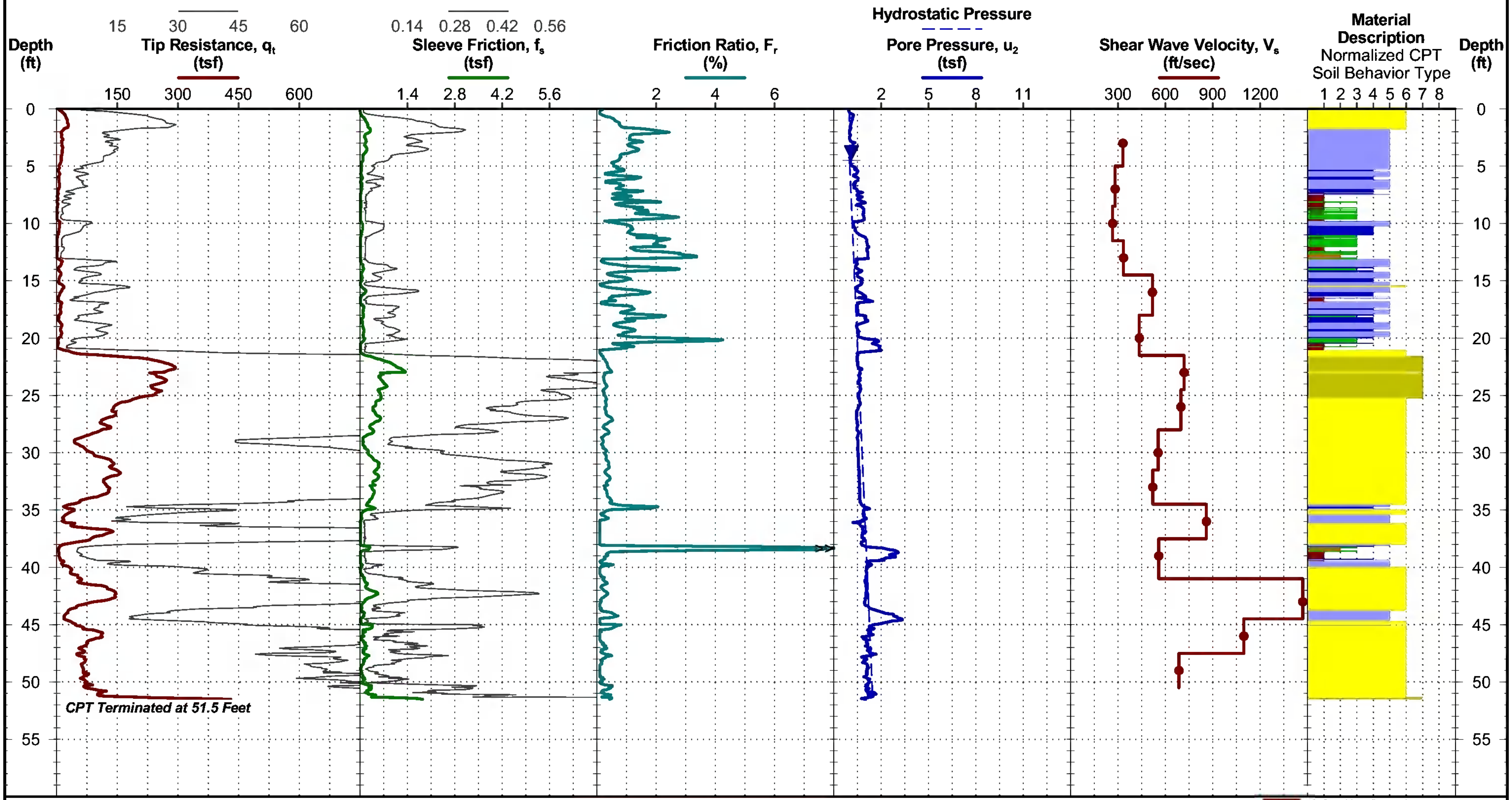
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3228°
Longitude: -79.3516°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟦 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 4.5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016	CPT Completed: 3/22/2016
Rig: Pagani TG73-200	Operator: JB
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. SCPT-217

Page 1 of 1

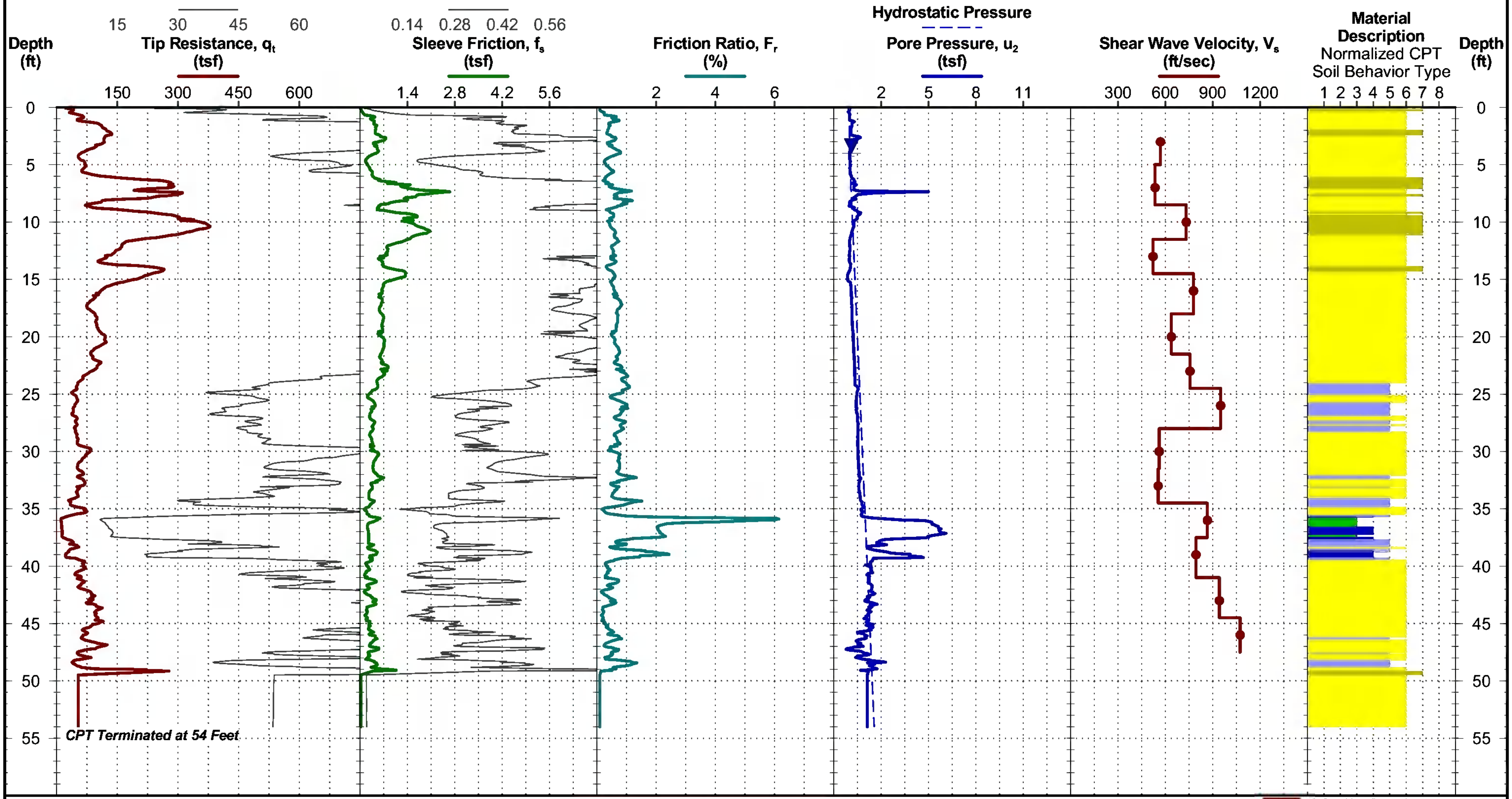
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3218°
Longitude: -79.3518°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟪 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟫 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 4 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016	CPT Completed: 3/22/2016
Rig: Pagani TG73-200	Operator: JB
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. CPT-218

Page 1 of 1

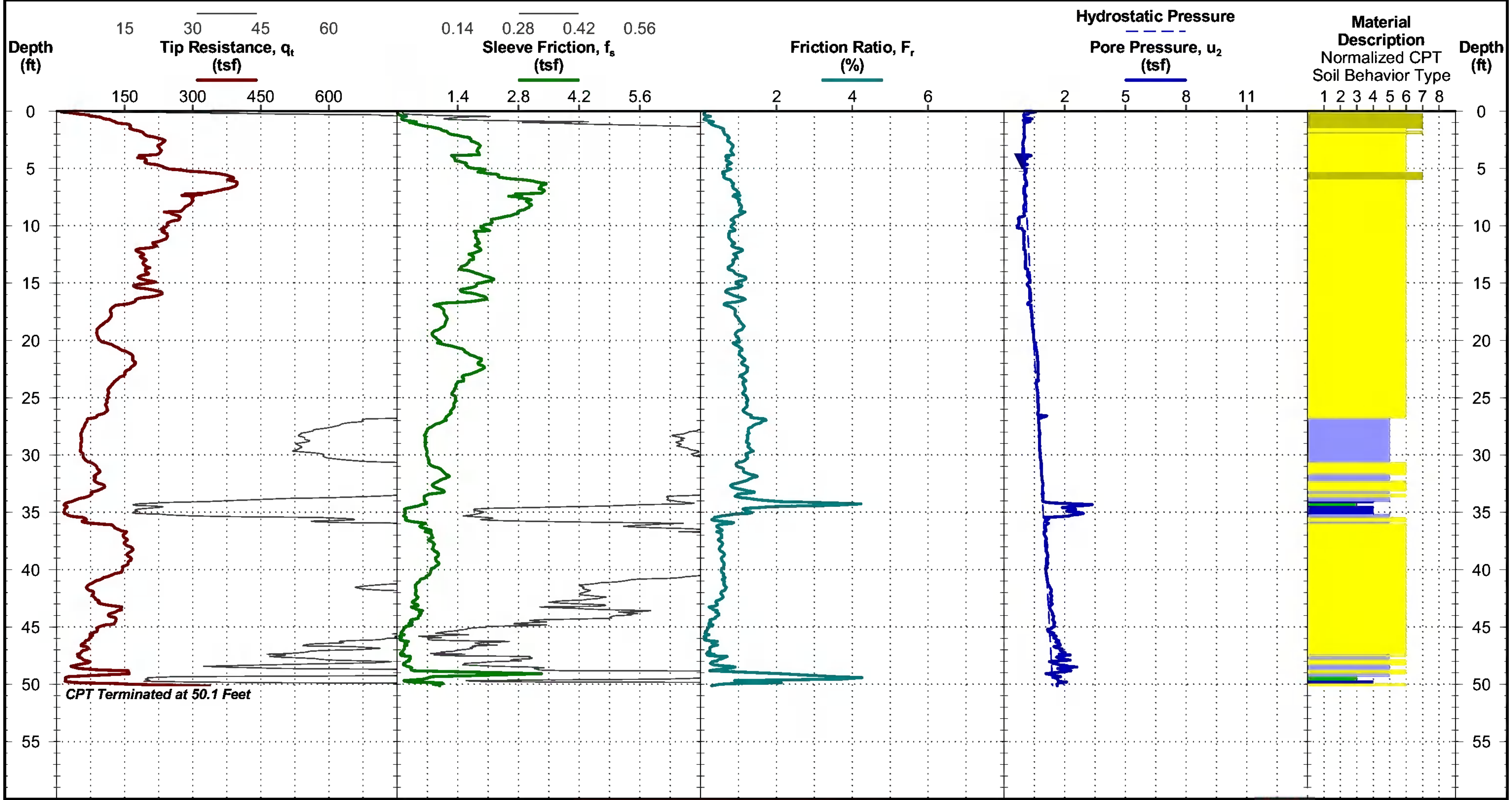
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3244°
Longitude: -79.3496°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟪 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/23/2016	CPT Completed: 3/23/2016
Rig: Pagani TG73-200	Operator: BR
Project No.: EN165065	Exhibit: A-4

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. CPT REPORT EN165065 WINYAH GENERATION STATION.GPJ TERRACON2015.GDT 5/11/16

CPT LOG NO. CPT-219

Page 1 of 1

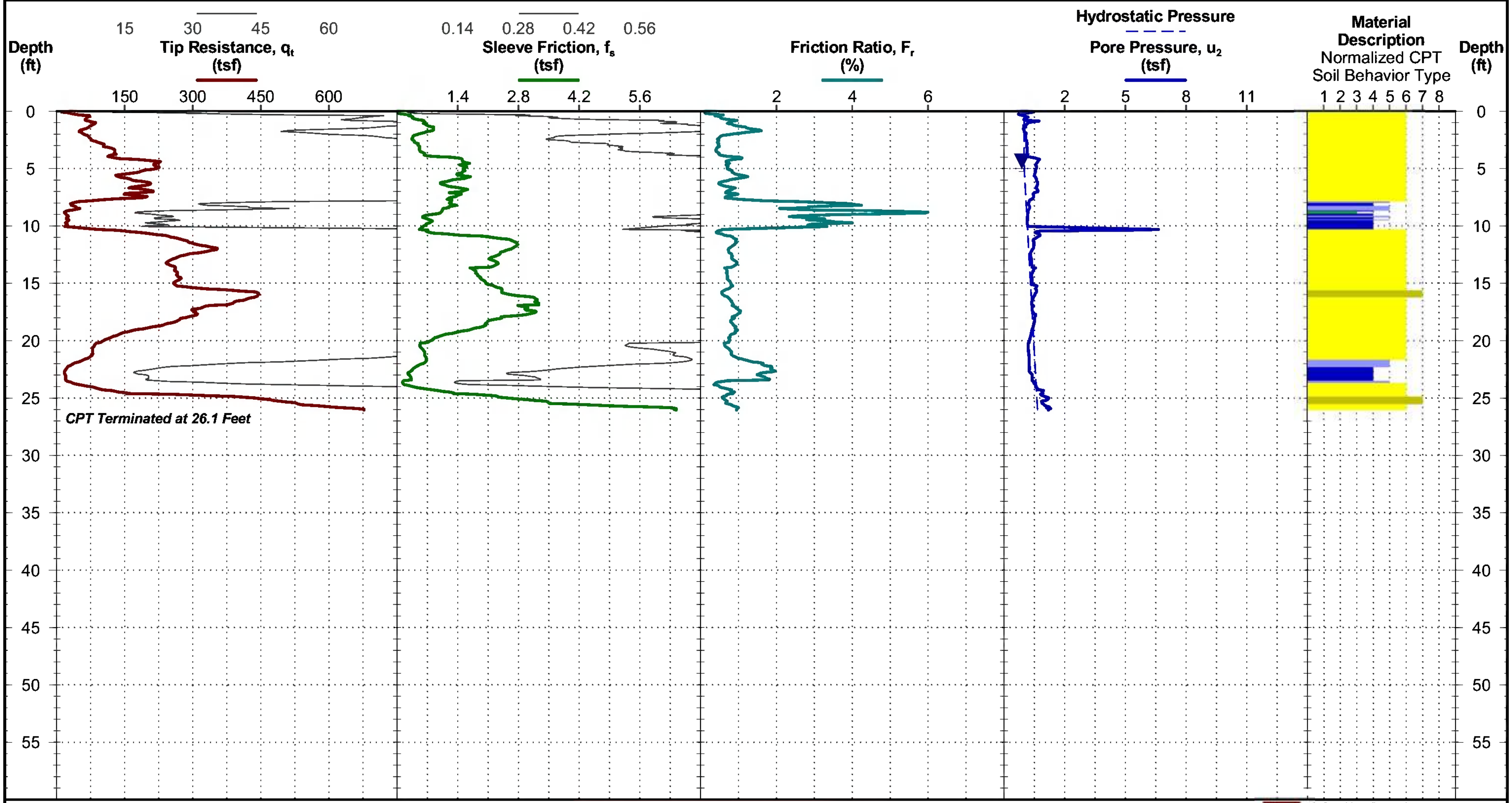
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3254°
Longitude: -79.3481°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟦 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 5 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm² and 150 cm²
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/23/2016	CPT Completed: 3/23/2016
Rig: Pagani TG73-200	Operator: BR
Project No.: EN165065	Exhibit: A-4

CPT LOG NO. SCPT-220

Page 1 of 1

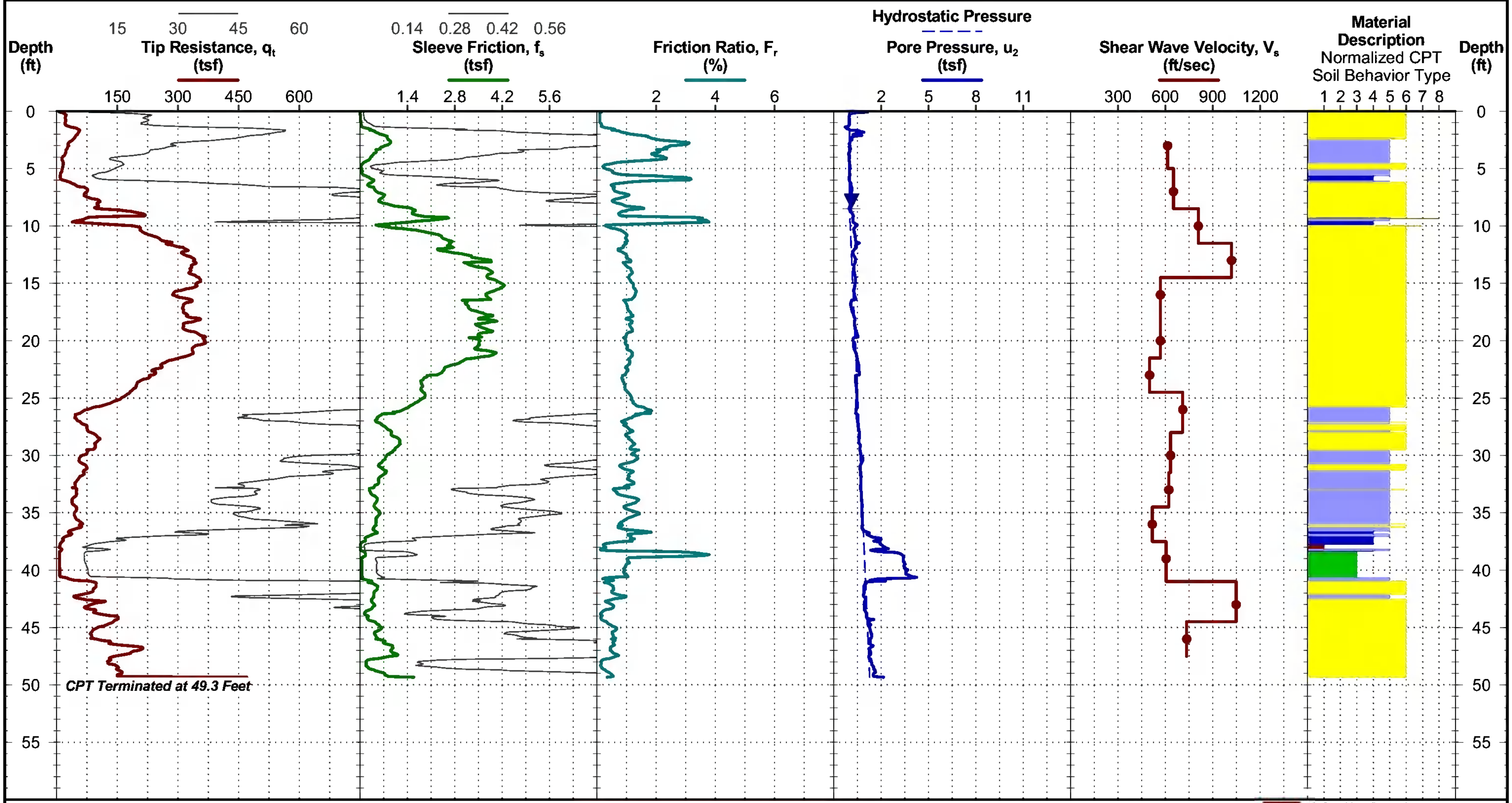
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3257°
Longitude: -79.3467°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravelly sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 8.5 ft measured water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/22/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/22/2016
Operator: BR
Exhibit: A-4

CPT LOG NO. CPT-228

Page 1 of 1

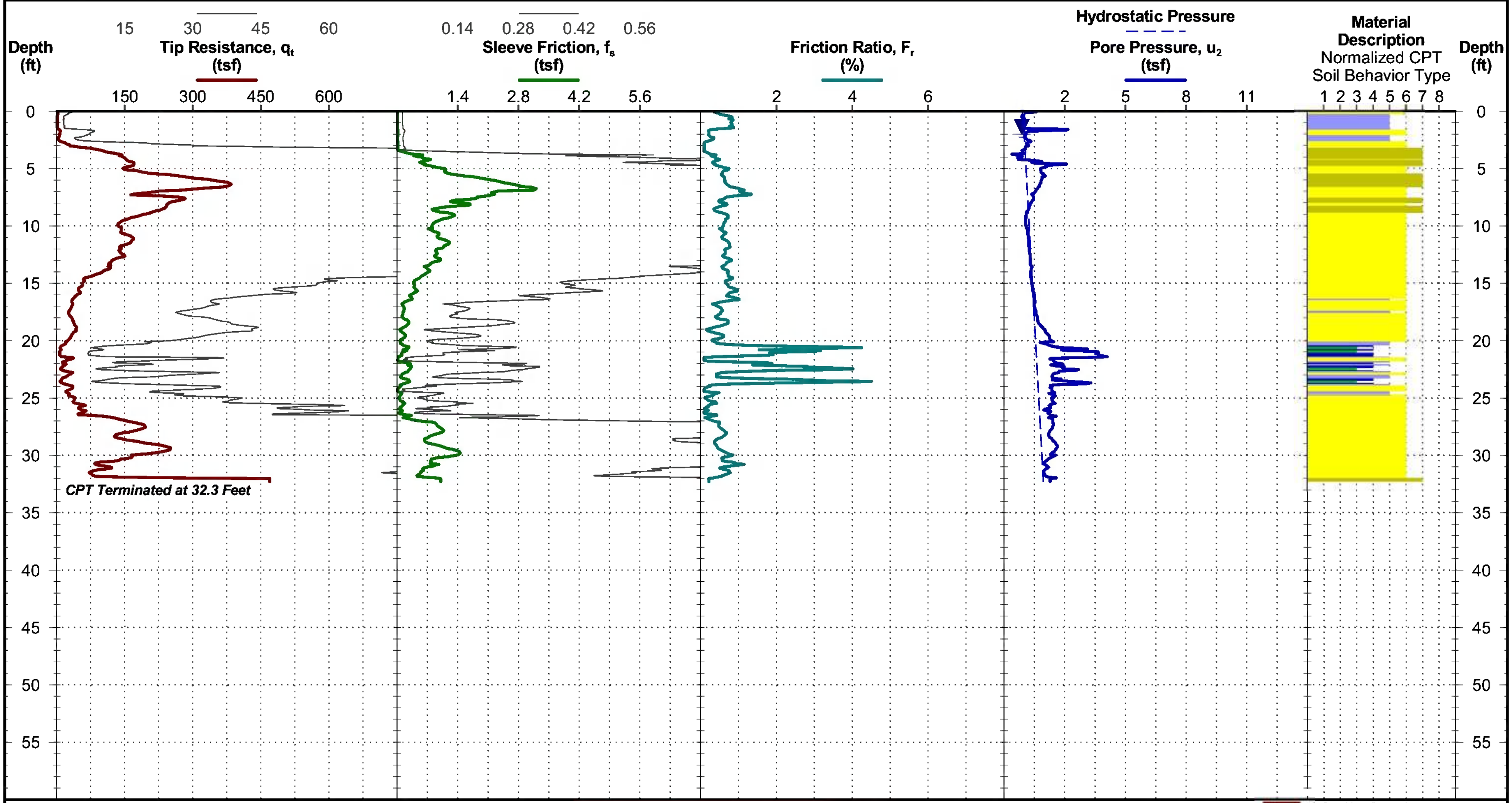
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3268°
Longitude: -79.3458°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

🟥	1 Sensitive, fine grained
🟧	2 Organic soils - clay
🟩	3 Clay - silty clay to clay
🟦	4 Silt mixtures - clayey silt to silty clay
🟪	5 Sand mixtures - silty sand to sandy silt
🟨	6 Sands - clean sand to silty sand
▨	7 Gravelly sand to dense sand
▨	8 Very stiff sand to clayey sand
▨	9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 2 ft measured water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/23/2016

CPT Completed: 3/23/2016

Rig: Pagani TG73-200

Operator: JB

Project No.: EN165065

Exhibit: A-4

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. CPT REPORT EN165065 WINYAH GENERATION STATION.GPJ TERRACON2015.GDT 5/11/16

CPT LOG NO. CPT-229

Page 1 of 1

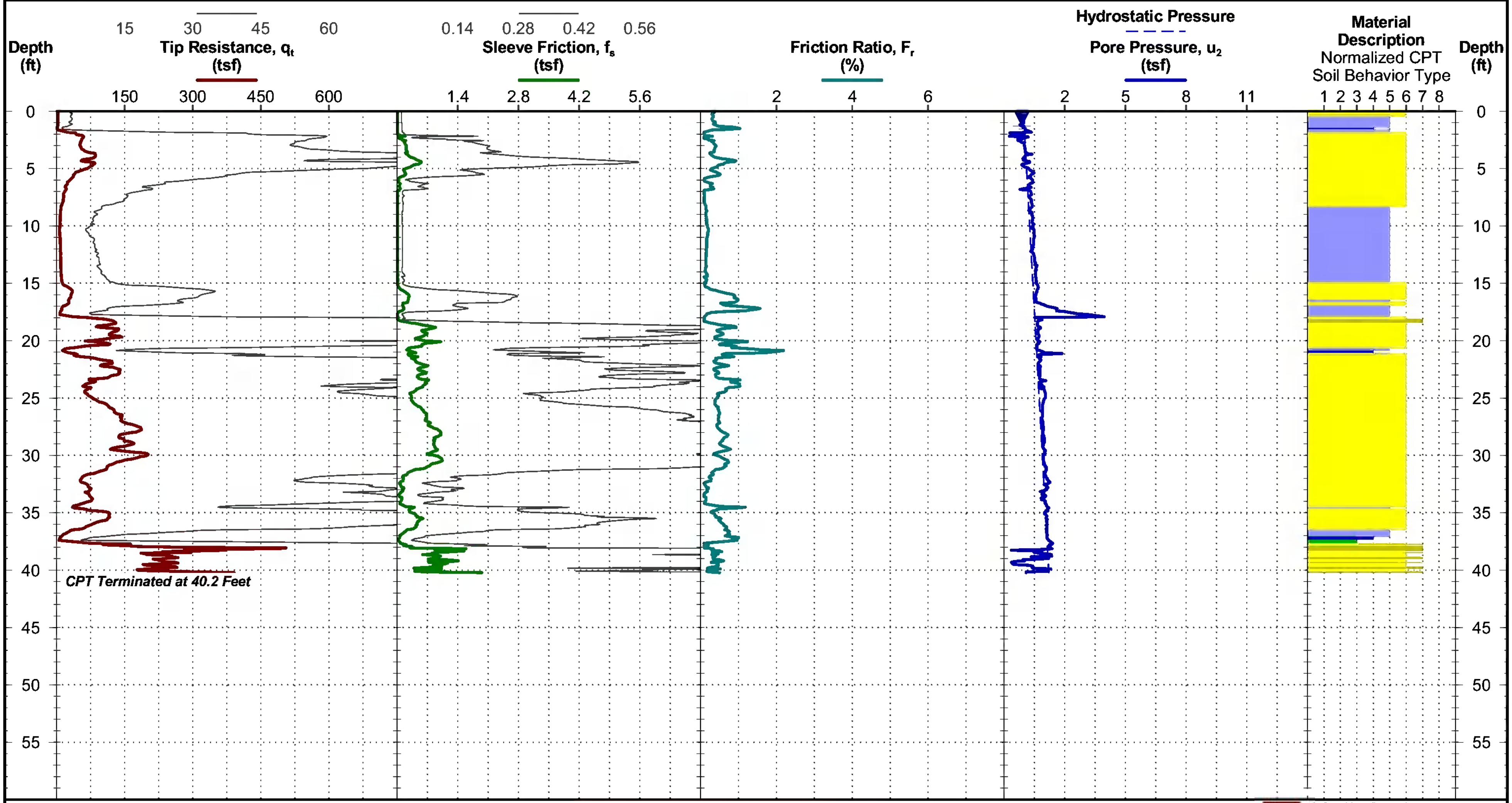
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.345°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

🟥 1 Sensitive, fine grained
🟧 2 Organic soils - clay
🟩 3 Clay - silty clay to clay
🟦 4 Silt mixtures - clayey silt to silty clay
🟪 5 Sand mixtures - silty sand to sandy silt
🟨 6 Sands - clean sand to silty sand
🟨 7 Gravelly sand to dense sand
🟫 8 Very stiff sand to clayey sand
⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 1.3 ft measured water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4675 with net area ratio of 0.84
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 10/22/2015
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

CPT Started: 3/24/2016
CPT Completed: 3/24/2016
Rig: Pagani TG73-200
Operator: BR
Project No.: EN165065
Exhibit: A-4

CPT LOG NO. CPT-229A

Page 1 of 1

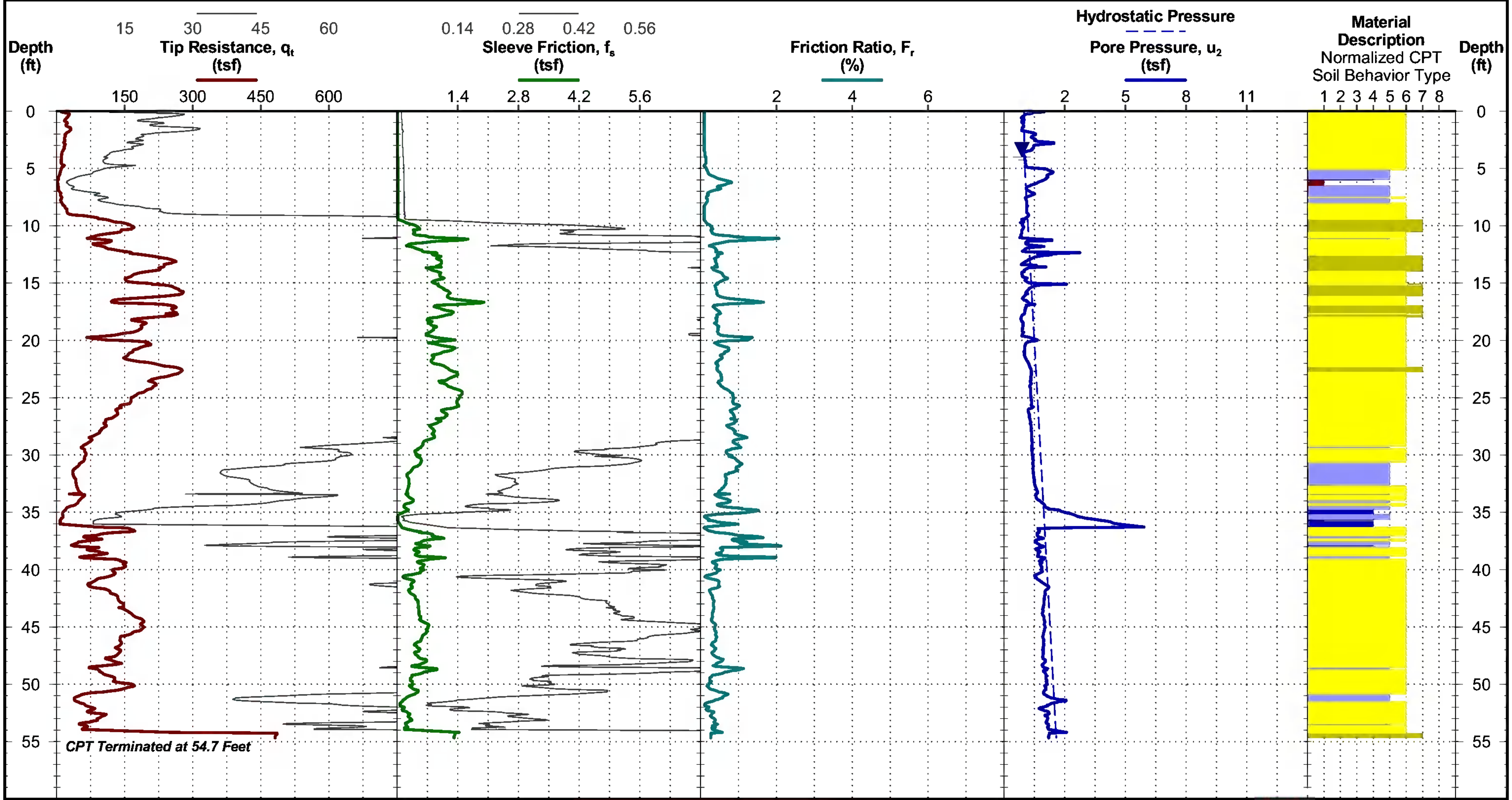
PROJECT: Winyah Generation Station

CLIENT: Santee Cooper
Moncks Corner, South Carolina

TEST LOCATION: See Exhibit A-2

SITE: Georgetown, South Carolina

Latitude: 33.3282°
Longitude: -79.3449°



See Exhibit A-3 for description of field procedures.
See Appendix C for explanation of symbols and abbreviations.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

- 🟥 1 Sensitive, fine grained
- 🟧 2 Organic soils - clay
- 🟩 3 Clay - silty clay to clay
- 🟦 4 Silt mixtures - clayey silt to silty clay
- 🟪 5 Sand mixtures - silty sand to sandy silt
- 🟨 6 Sands - clean sand to silty sand
- 🟨 7 Gravelly sand to dense sand
- 🟫 8 Very stiff sand to clayey sand
- ⬛ 9 Very stiff fine grained

WATER LEVEL OBSERVATION

▼ 4 ft estimated water depth
(used in normalizations and correlations; see Appendix C)

Probe no. 4526 with net area ratio of 0.83
U2 pore pressure transducer location
Manufactured by Geotech A.B.; calibrated 12/7/2015
Tip and sleeve areas of 10 cm² and 150 cm²
Ring friction reducer with O.D. of 1.875 in

Terracon
1450 Fifth St W
North Charleston, SC

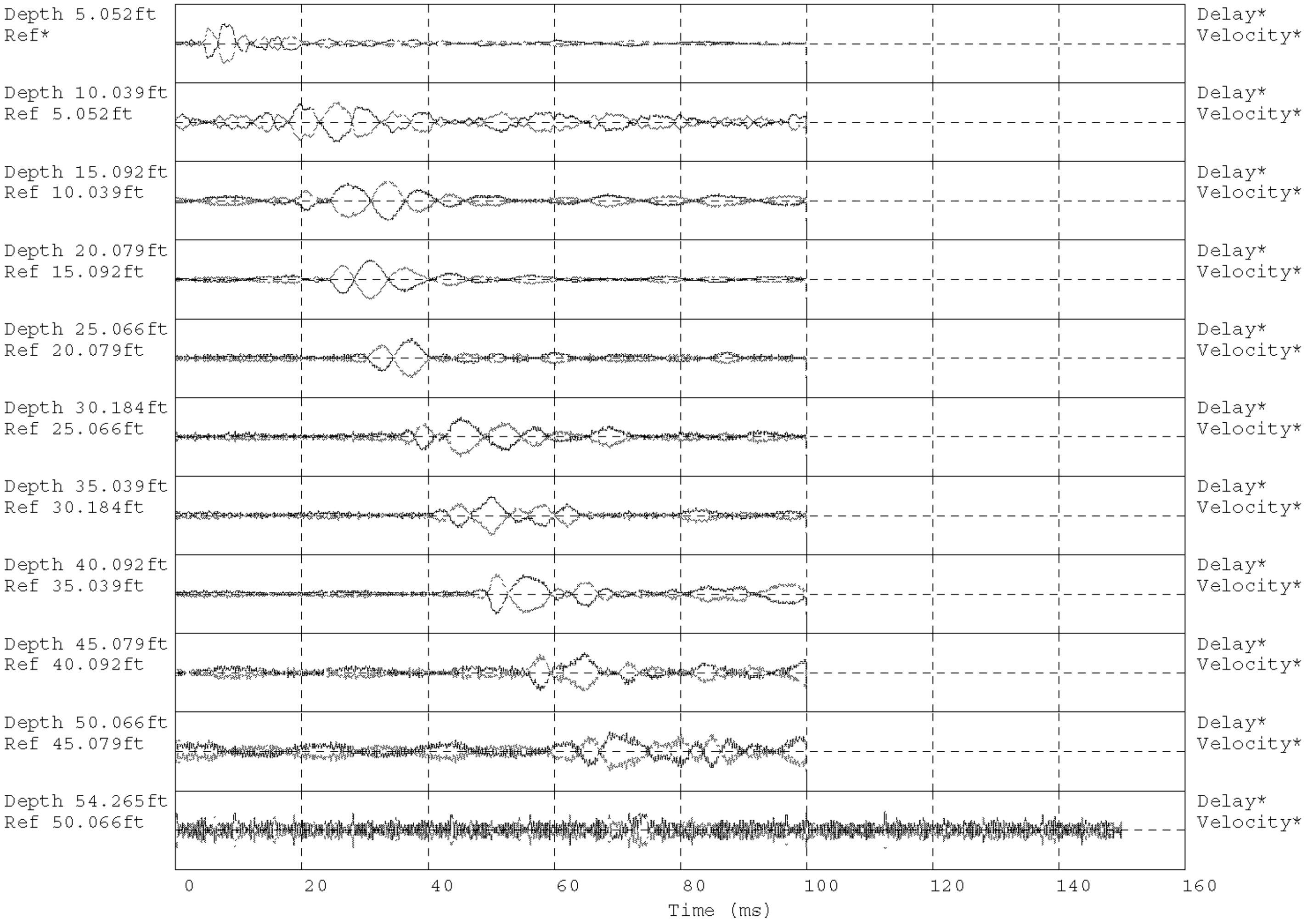
CPT Started: 3/23/2016
Rig: Pagani TG73-200
Project No.: EN165065

CPT Completed: 3/23/2016
Operator: JB
Exhibit: A-4

ATTACHMENT 3-B

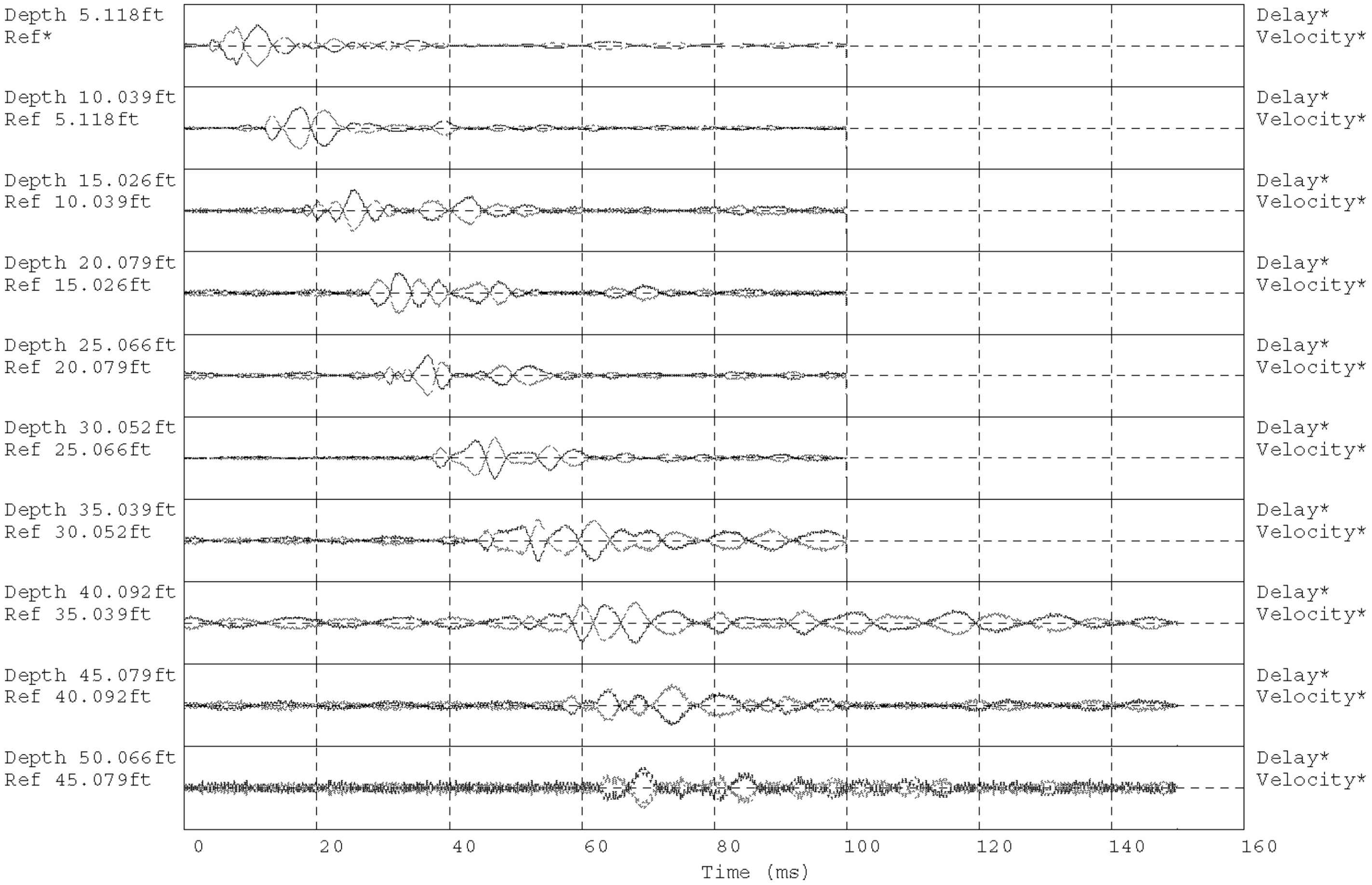
Shear Wave Velocity Test Data
(Provided by Mid-Atlantic Drilling)

Mid-Atlantic Drilling
CPT-137



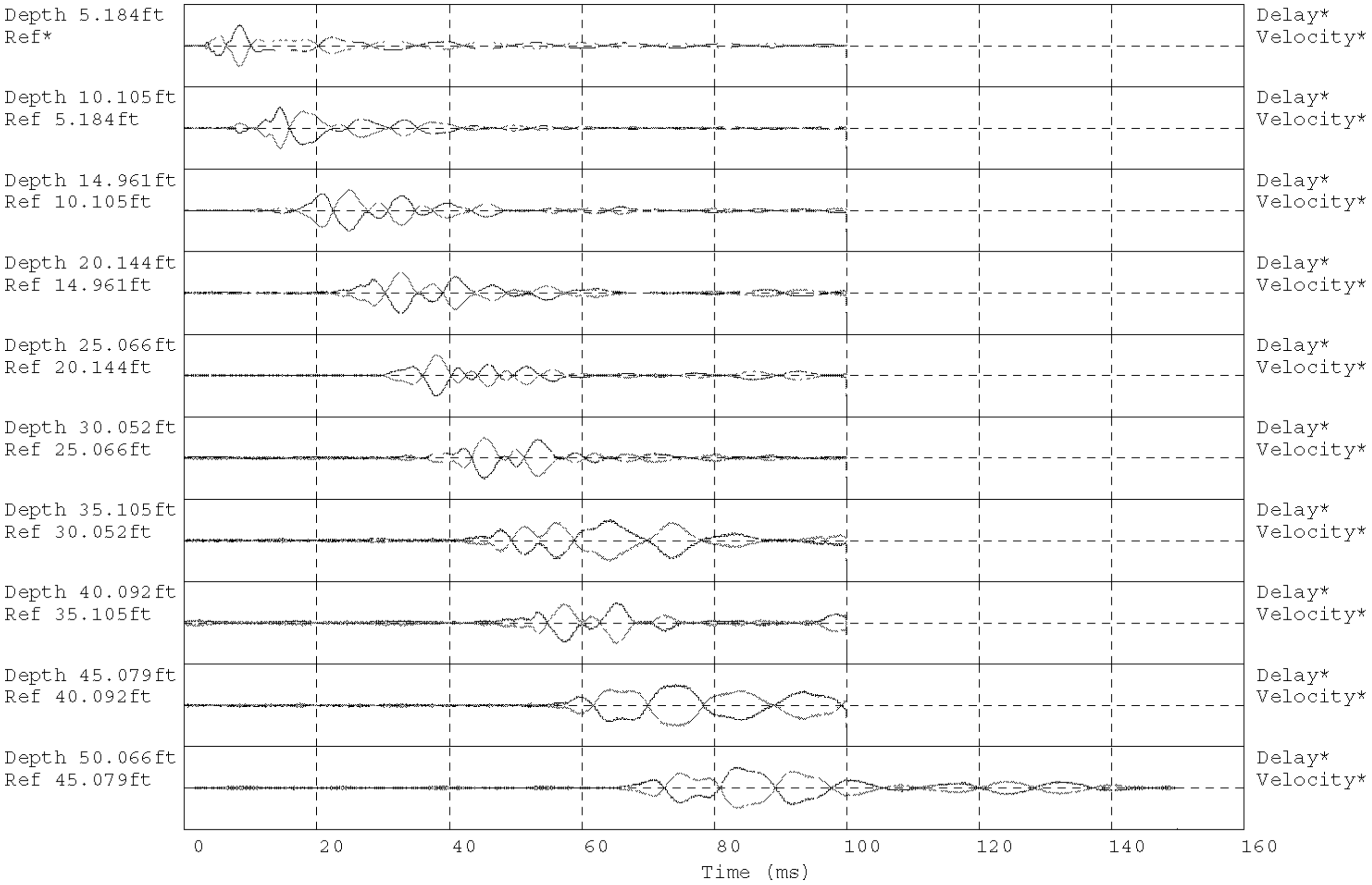
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-140



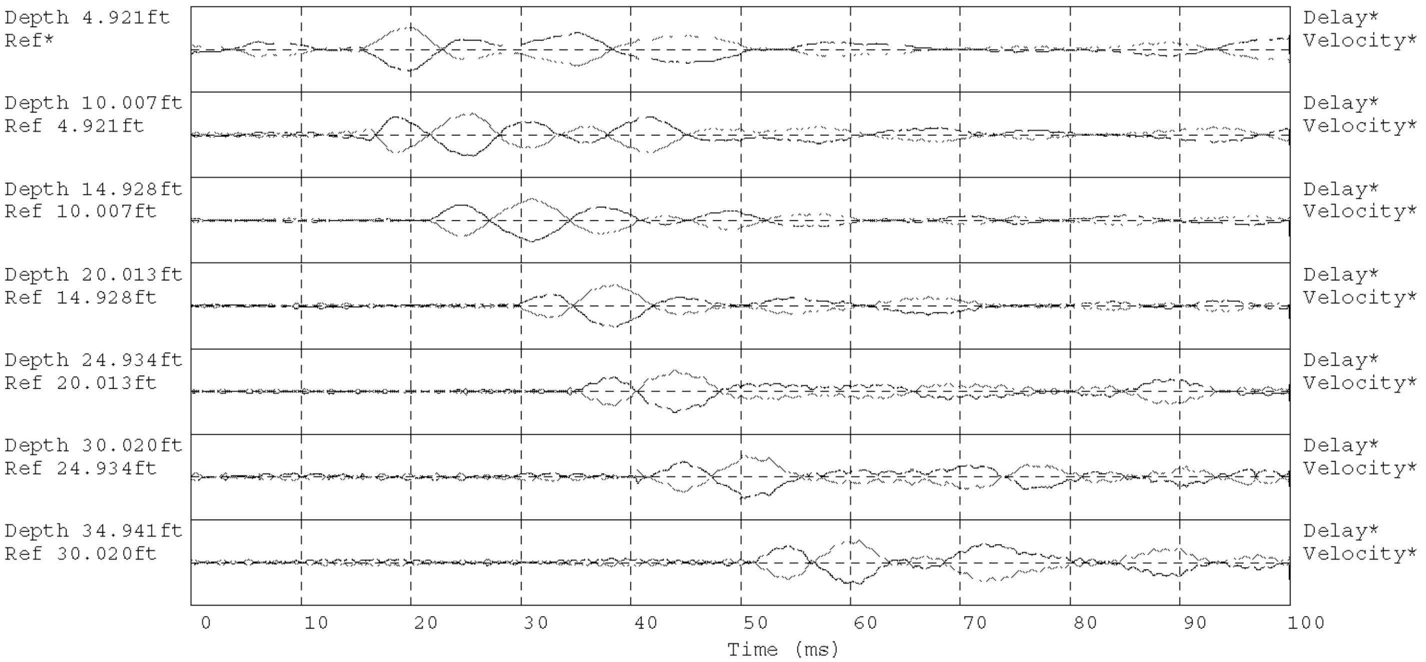
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-144



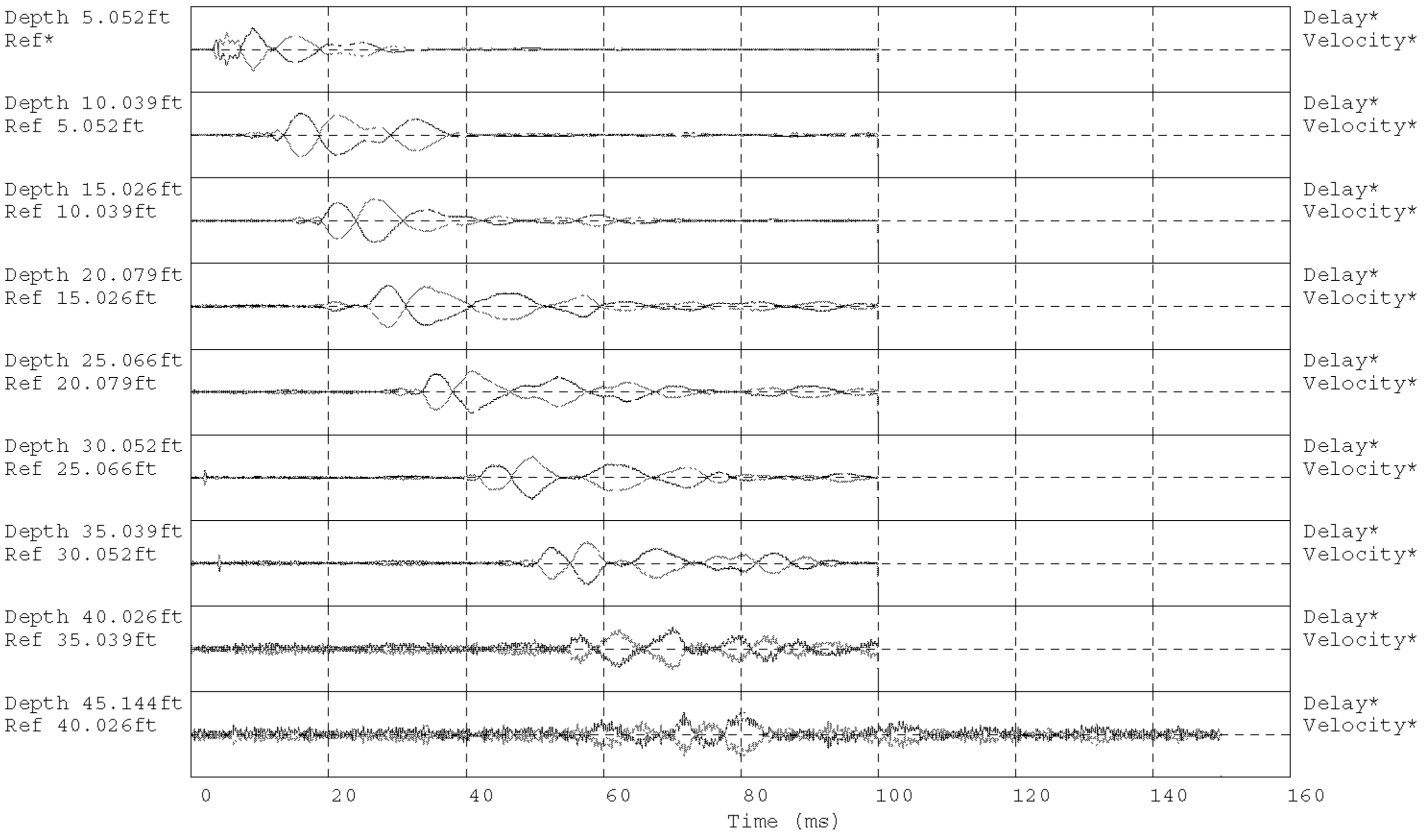
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-145



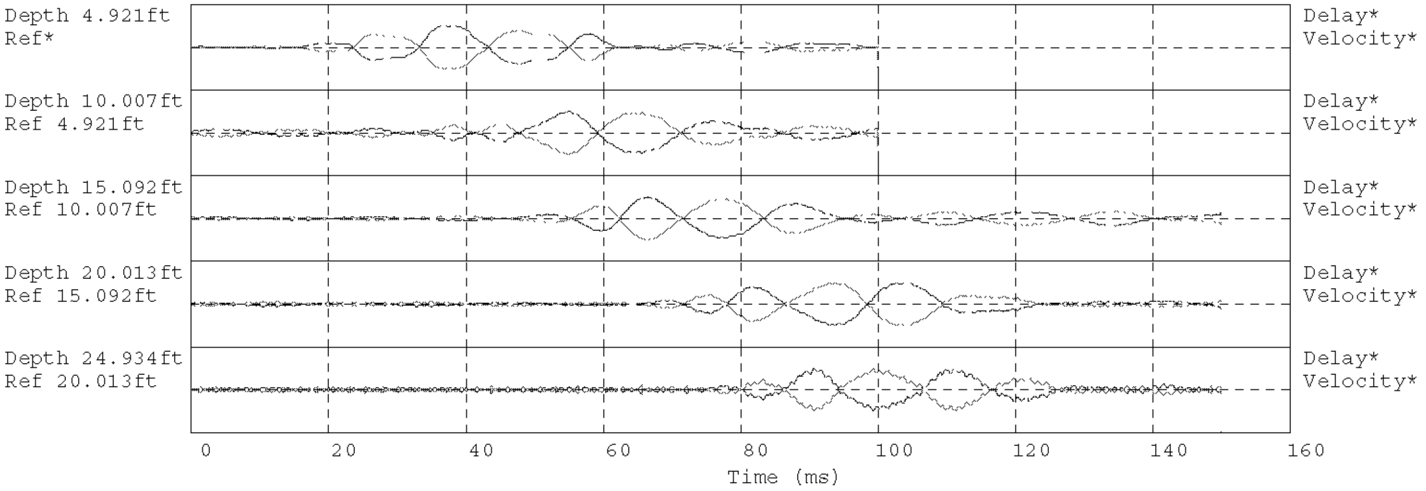
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-147



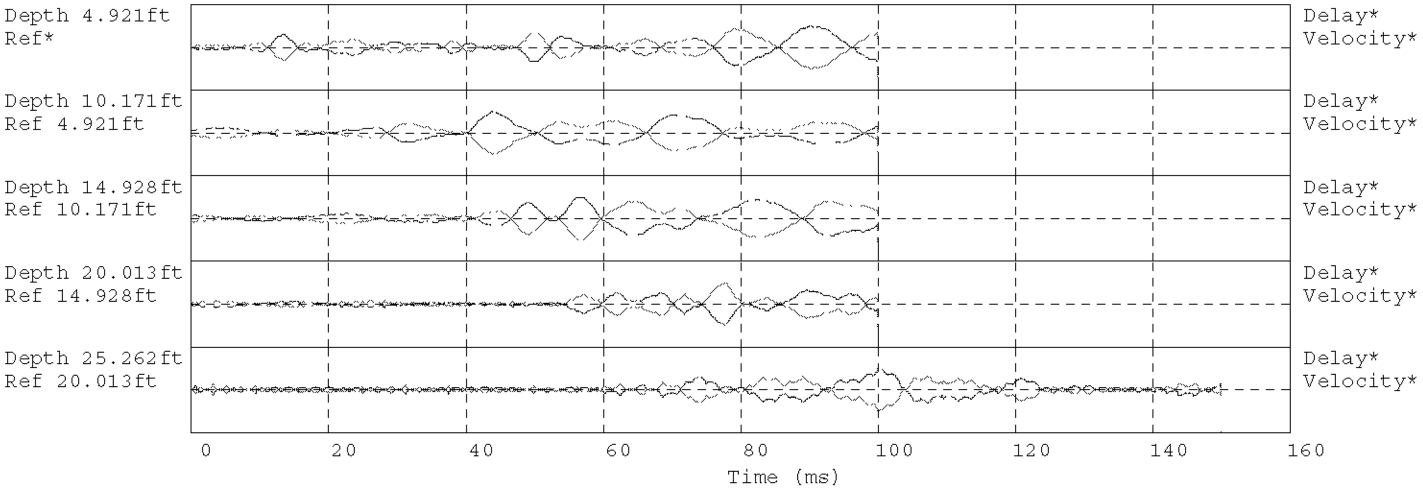
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-154



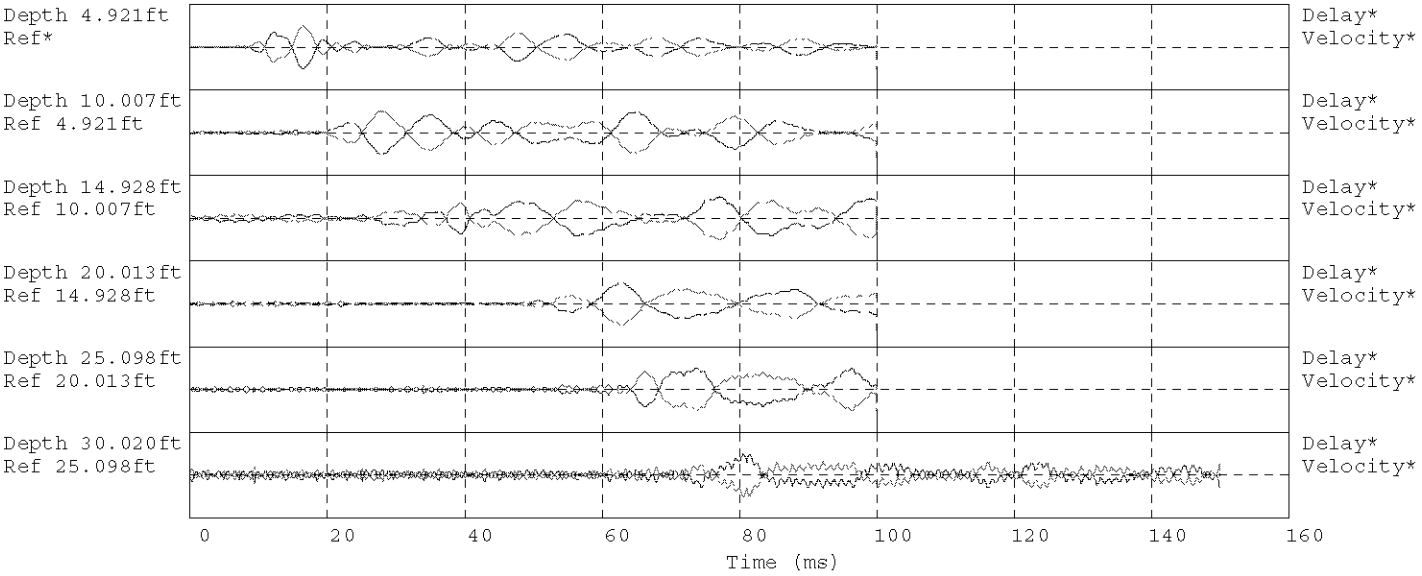
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-156



Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling
CPT-158



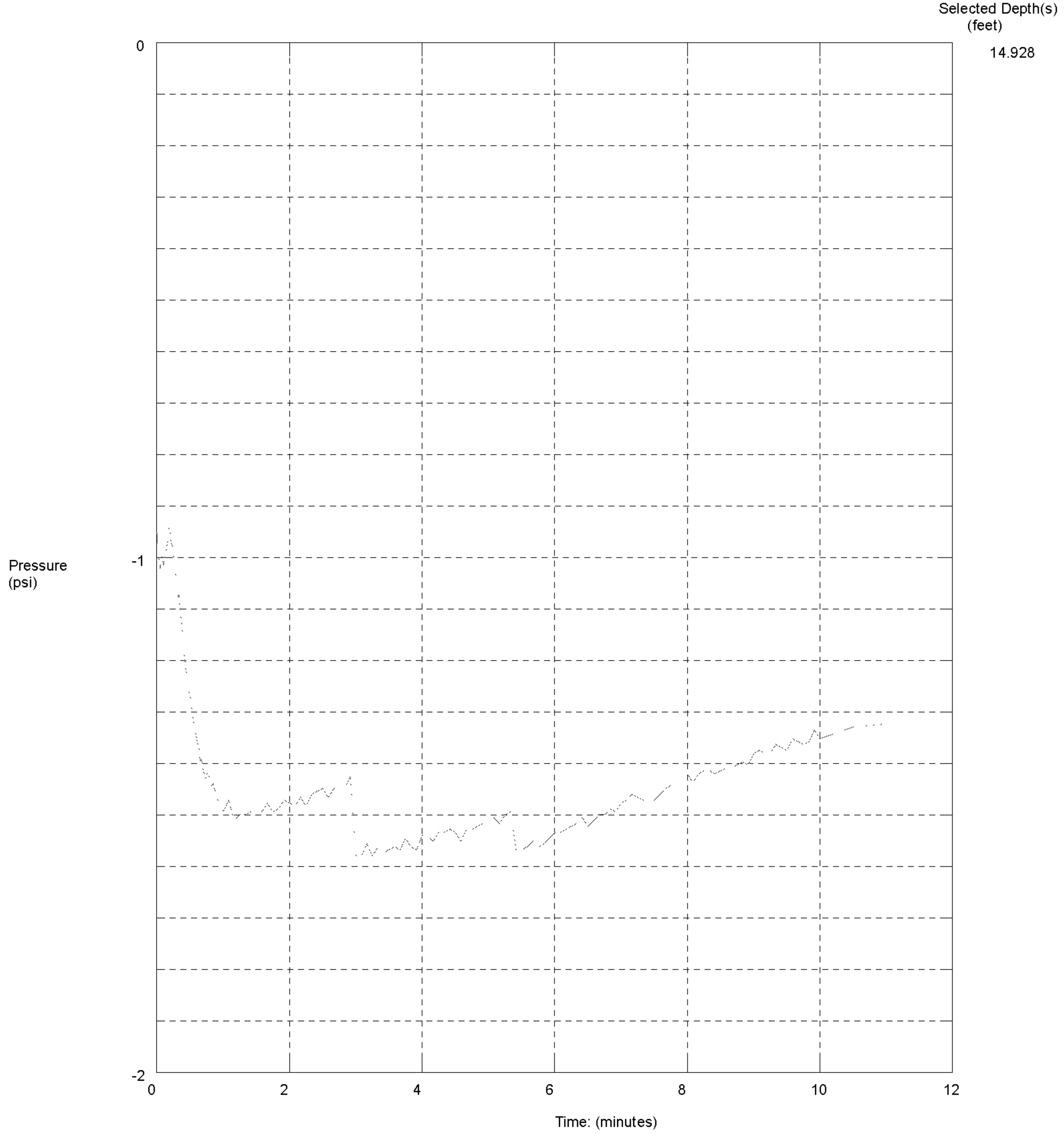
Hammer to Rod String Distance 1 (m)
* = Not Determined

ATTACHMENT 3-C

Dissipation Test Data
(Provided by Mid-Atlantic Drilling)

Mid-Atlantic Drilling Inc.

Operator Cory Robison	CPT Date/Time: 9/26/2013 10:36:07 AM
Sounding: CPT-138	Location: Georgetown S.C.
Cone Used: DDG1195	Job Number: GSC-5242



Maximum Pressure = -0.943 psi
Hydrostatic Pressure = 6.479 psi

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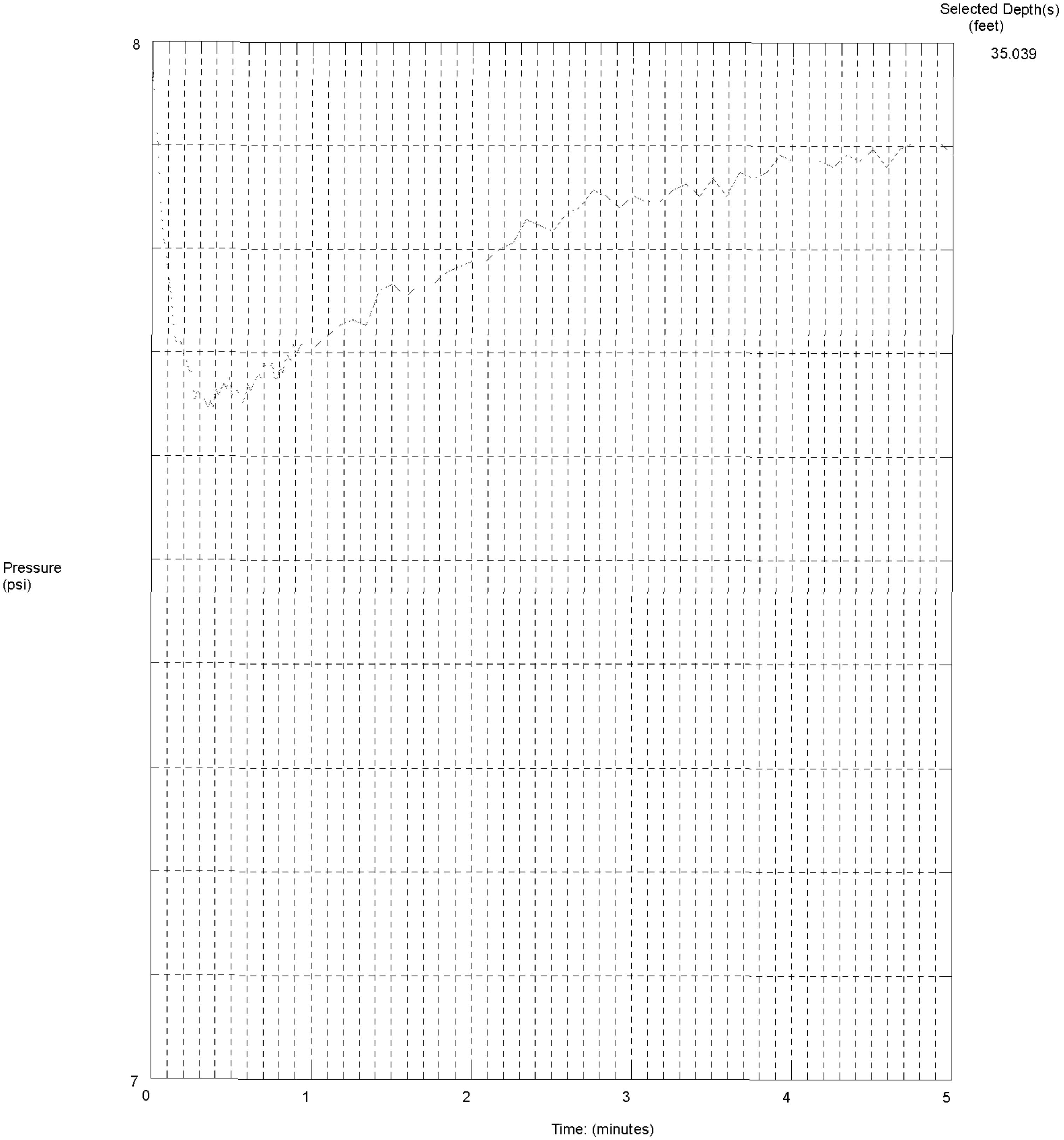
Geosyntec

..

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-138A
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 5:53:18 PM
Location: Georgetown S.C.
Job Number: GSC-5242

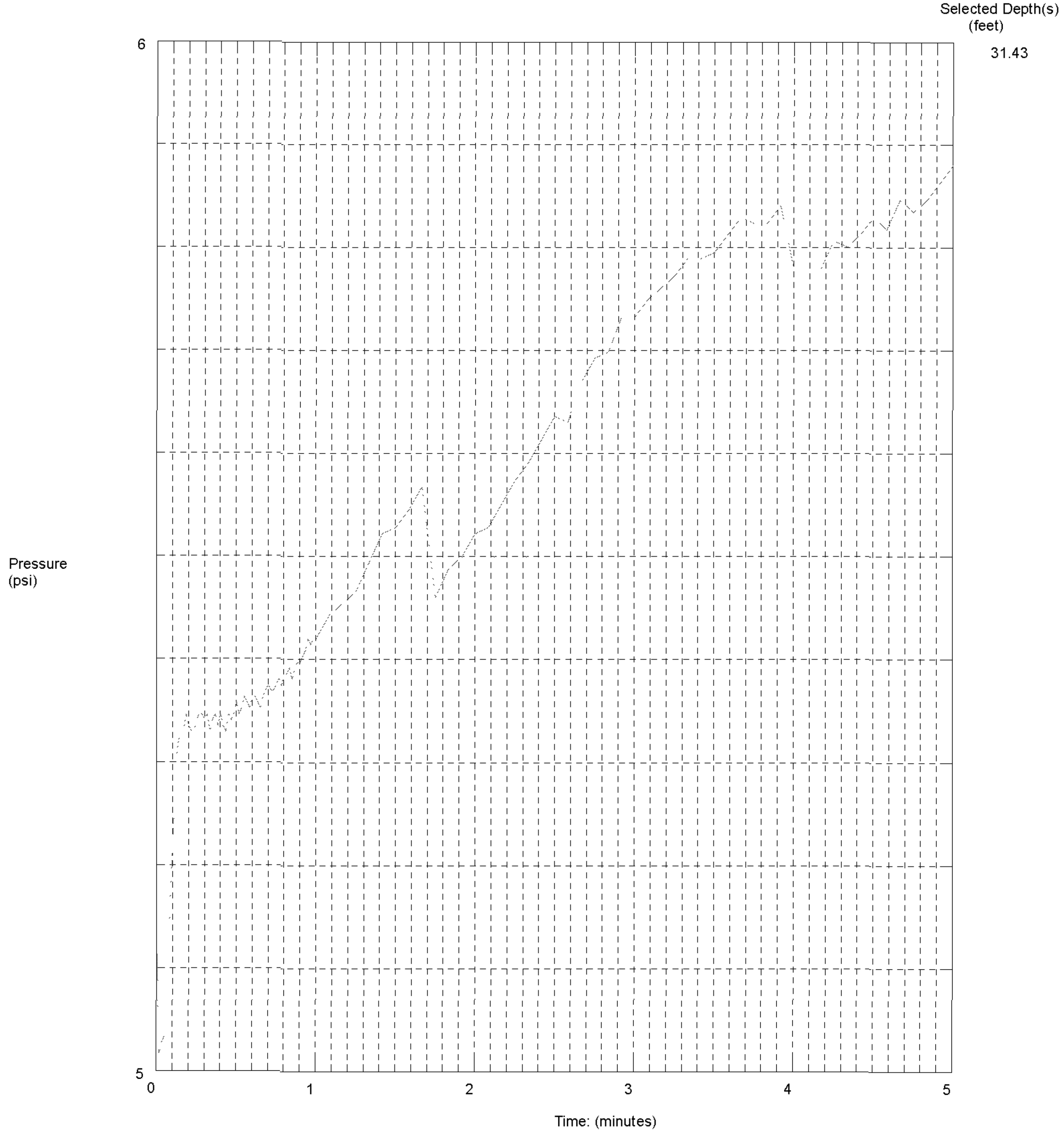


Maximum Pressure = 7.971 psi
Hydrostatic Pressure = 15.207 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-143
Cone Used: DDG1195

CPT Date/Time: 10/1/2013 1:34:00 PM
Location: Georgetown S.C.
Job Number: GSC-5242

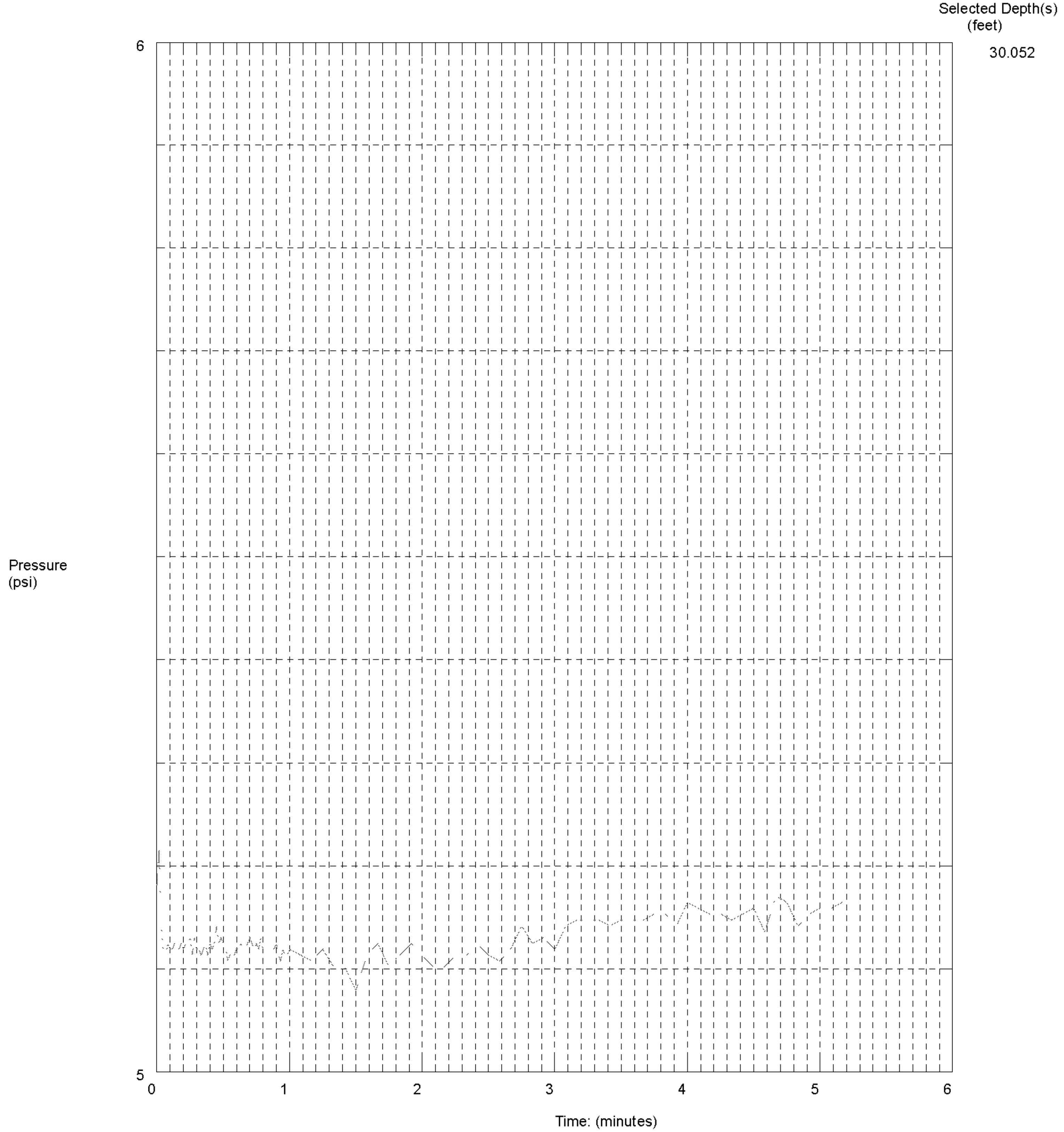


Maximum Pressure = 5.88 psi
Hydrostatic Pressure = 13.641 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-146
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 8:03:00 AM
Location: Georgetown S.C.
Job Number: GSC-5242

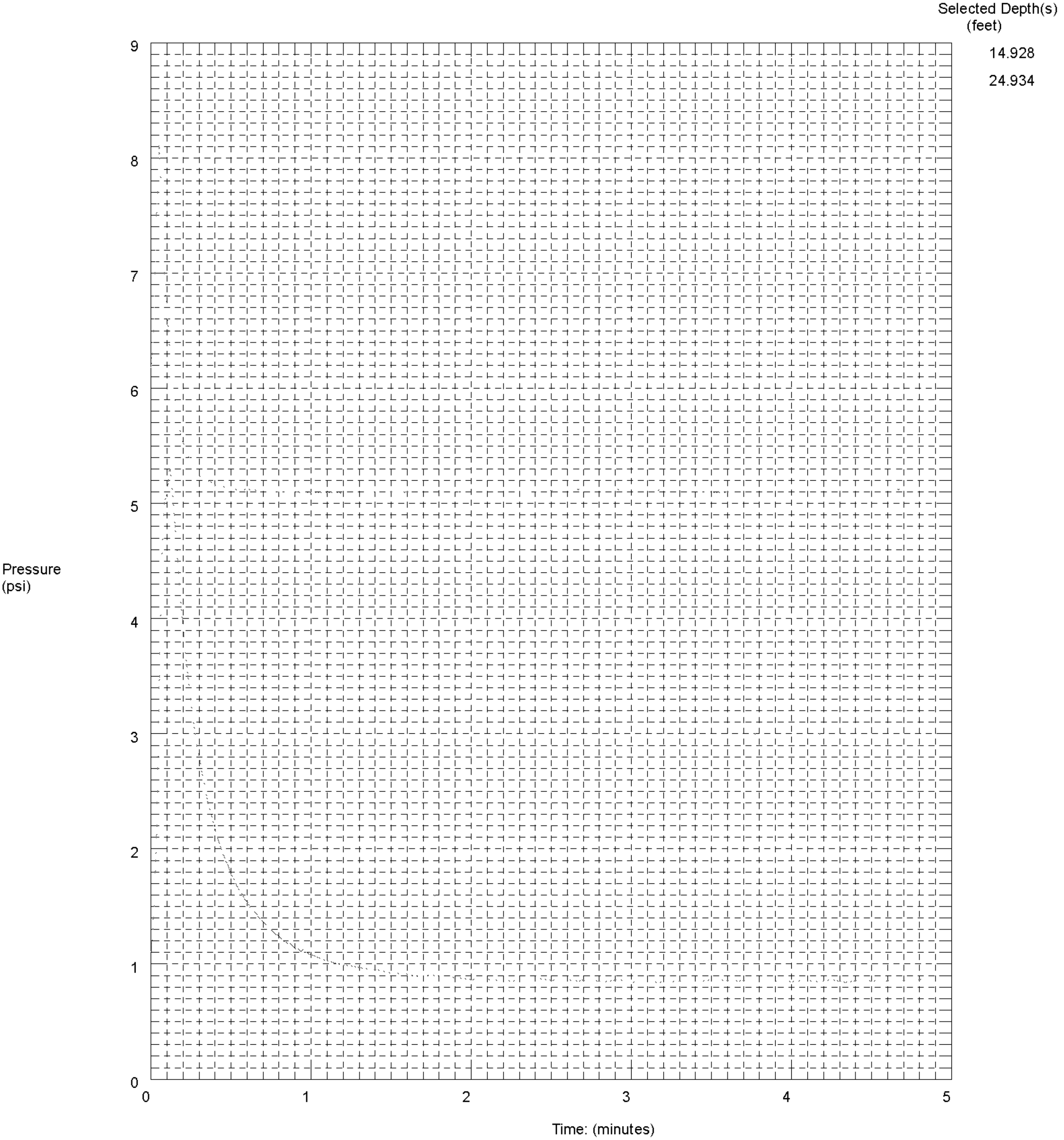


Maximum Pressure = 5.216 psi
Hydrostatic Pressure = 13.043 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-155
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 5:48:34 PM
Location: Georgetown S.C.
Job Number: GSC-5242

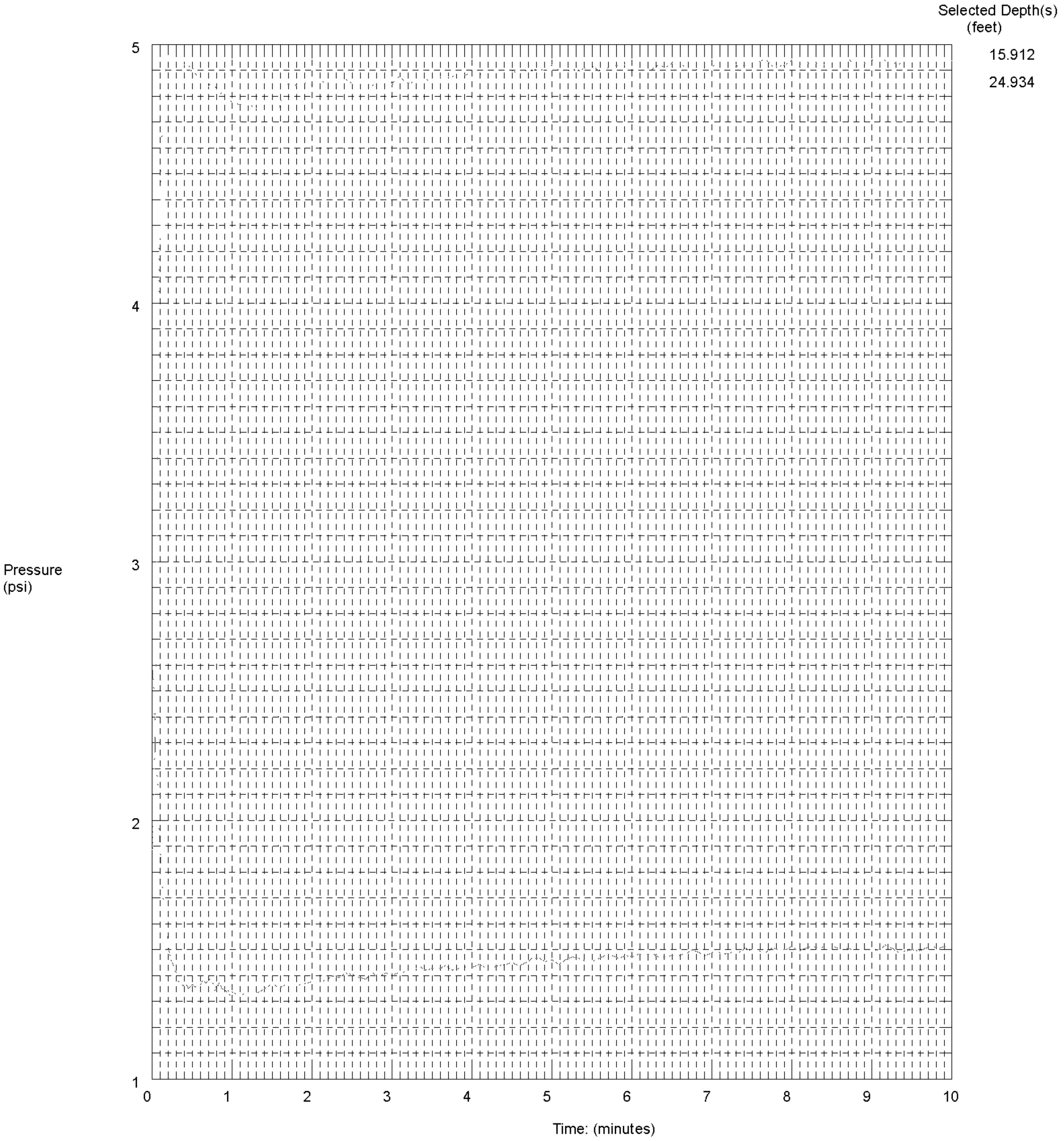


Maximum Pressure = 8.125 psi
Hydrostatic Pressure = 10.821 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-157a
Cone Used: DDG1195

CPT Date/Time: 9/25/2013 3:21:59 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Pressure = 4.977 psi
Hydrostatic Pressure = 10.821 psi

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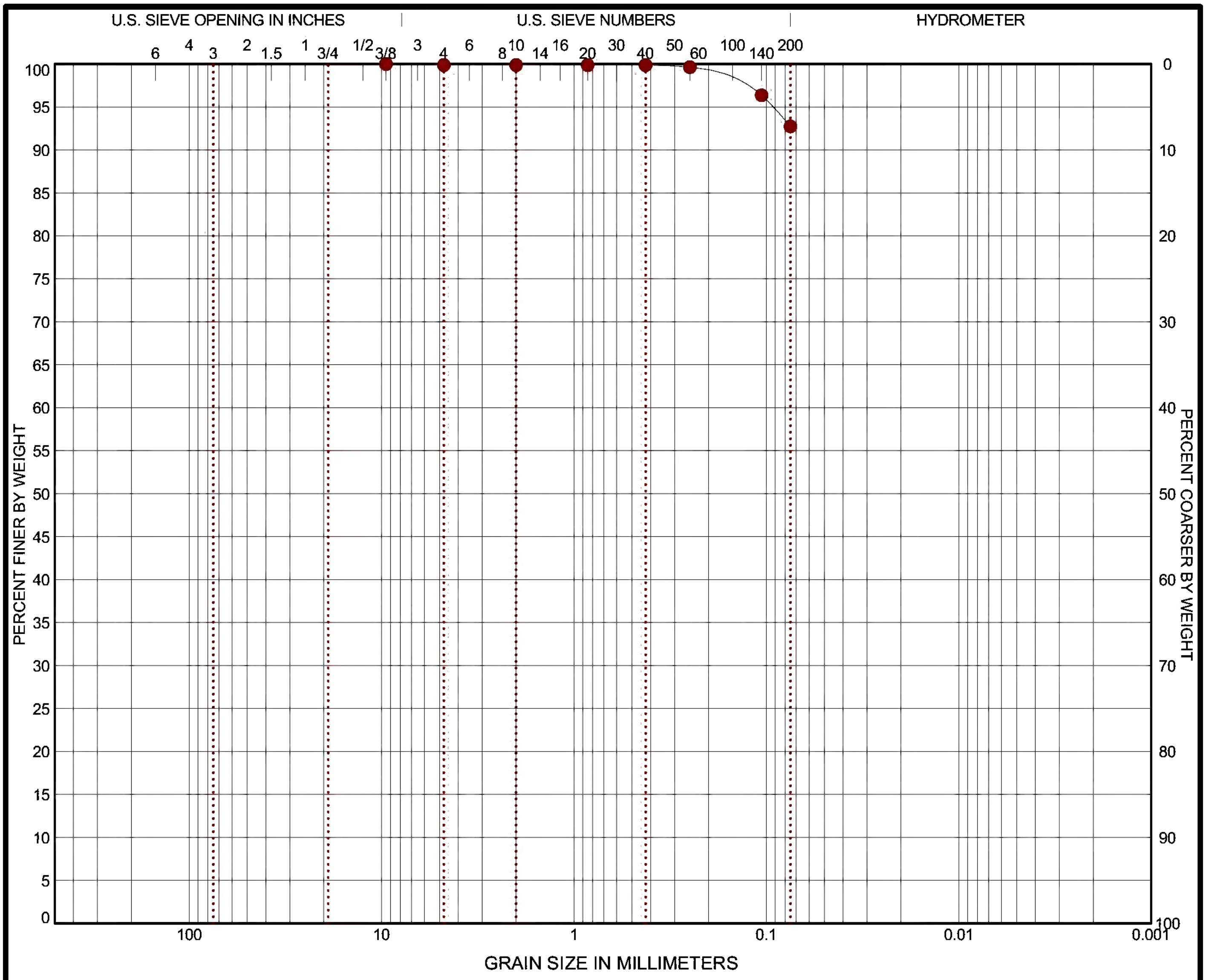
Geosyntec

ATTACHMENT 4

Laboratory Testing Results (provided by Excel Geotechnical Testing and Terracon)

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-304**	**13.5 - 15**	**0.0**	**0.1**	**7.1**		**92.7**		**ML**

	GRAIN SIZE		
	●		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILT (ML)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

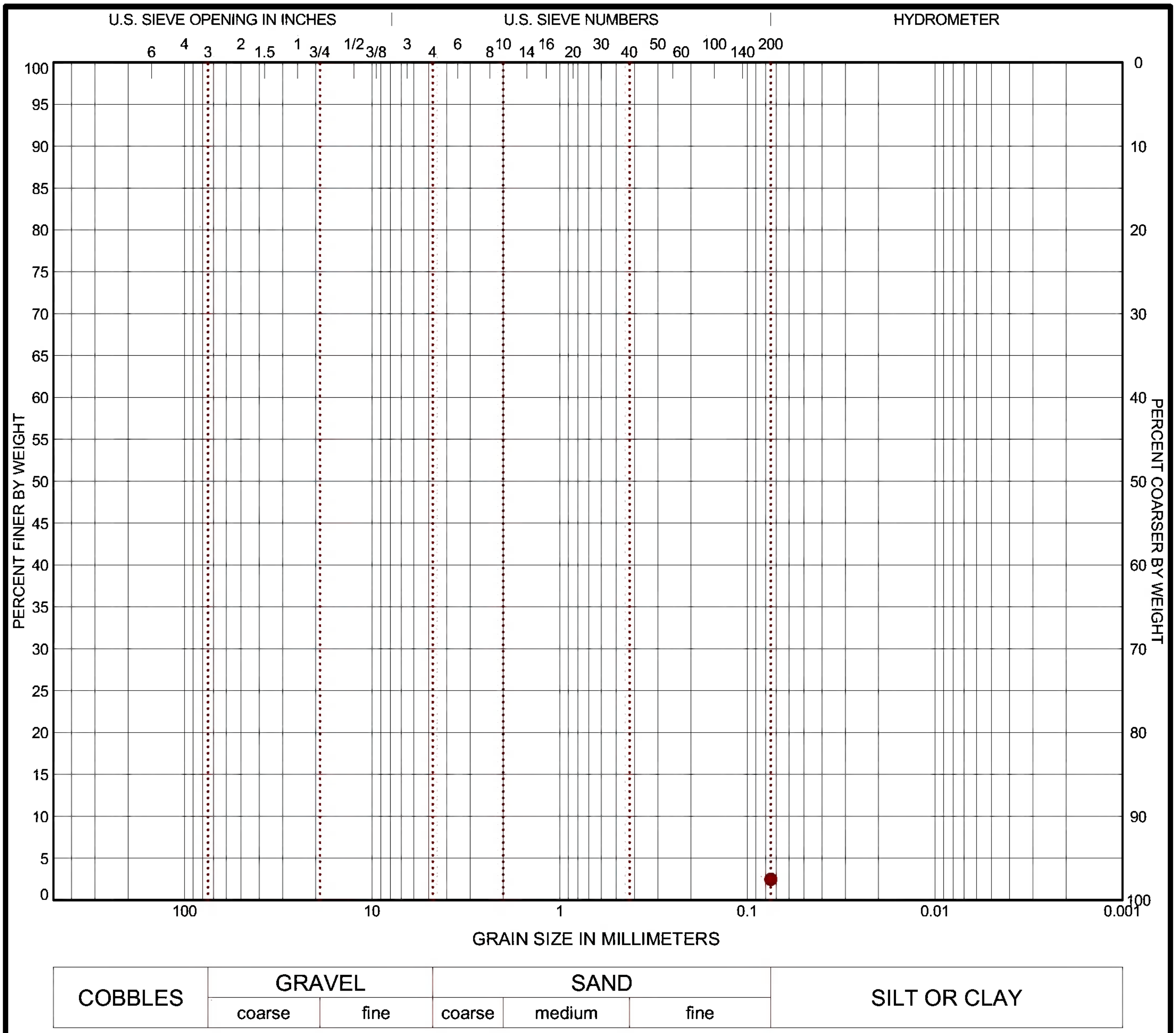
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-11

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-304**	**30 - 32**					**2.5**		

	GRAIN SIZE		
	●		
D₆₀ D₃₀ D₁₀			
	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER ●		
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
#4			
#10			
#20			
#40			
#60			
#100			
#200	**2.48**		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

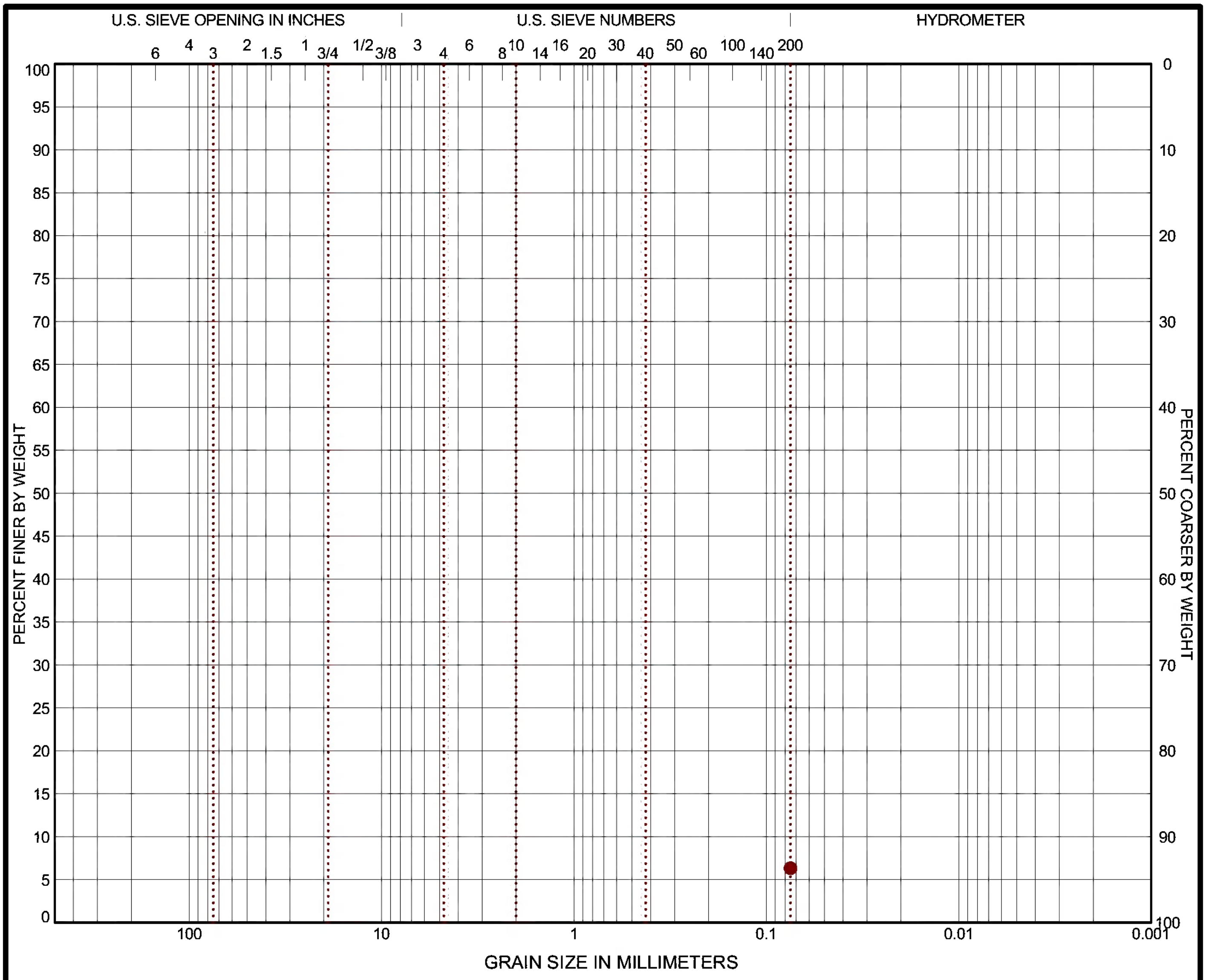
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-12

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-304**	**38 - 40**					**6.3**		

	GRAIN SIZE		
	●		
D_{60} D_{30} D_{10}			
	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
#4			
#10			
#20			
#40			
#60			
#100			
#200	**6.33**		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

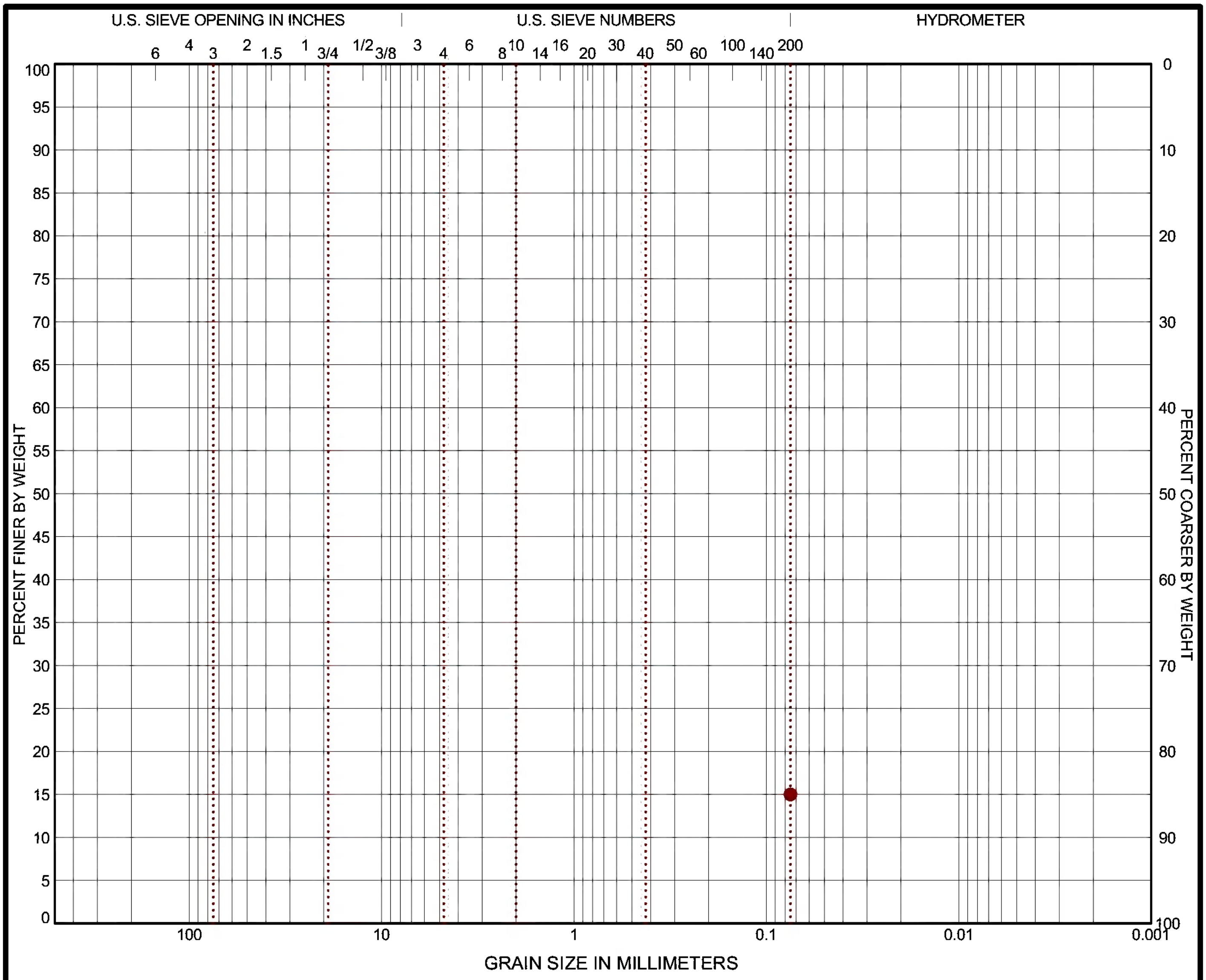
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-13

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-304**	**48.5 - 50**					**15.0**		**SM**

	GRAIN SIZE		
	●		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILTY SAND (SM)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

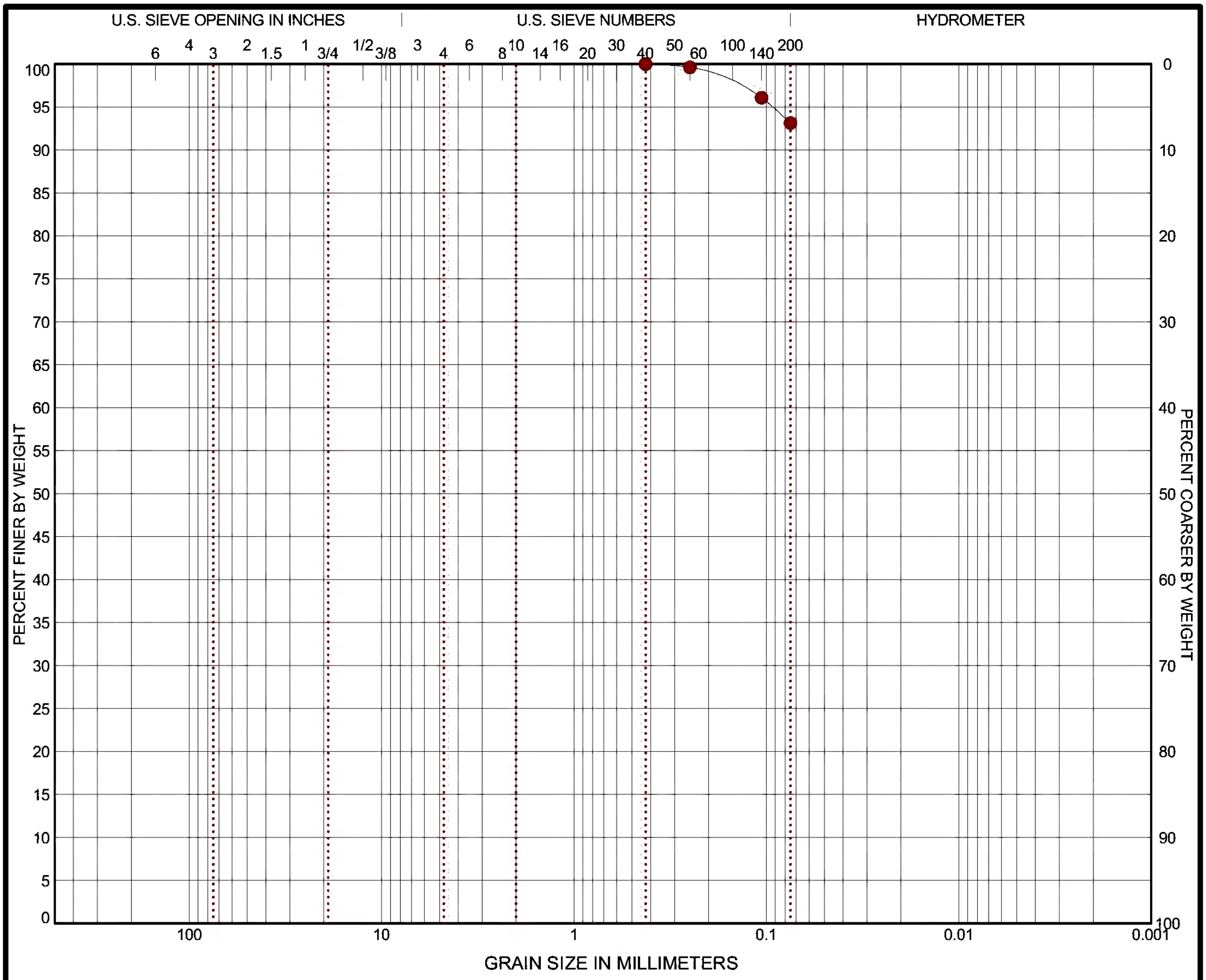
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-14

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-305**	**13.5 - 15**	**0.0**	**0.0**	**6.9**		**93.1**		**ML**

	GRAIN SIZE		
	●		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILT (ML)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

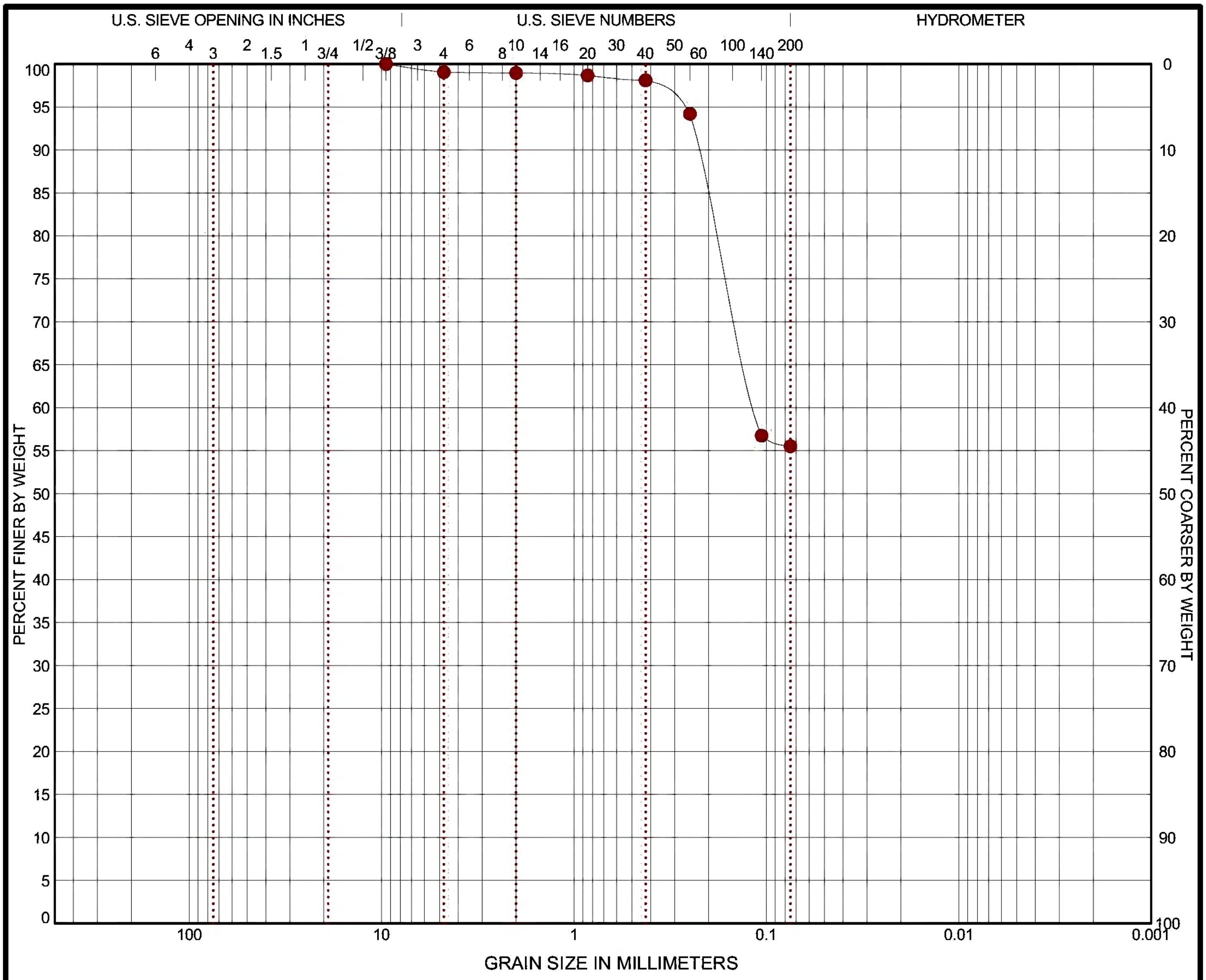
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-15

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-305**	**23.5 - 25**	**0.0**	**1.0**	**43.5**		**55.5**		**ML**

	GRAIN SIZE		
☒	●		
D_{60}	**0.114**		
D_{30}			
D_{10}			
☒	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

<u>SOIL DESCRIPTION</u>

● SANDY SILT (ML)

<u>REMARKS</u>

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

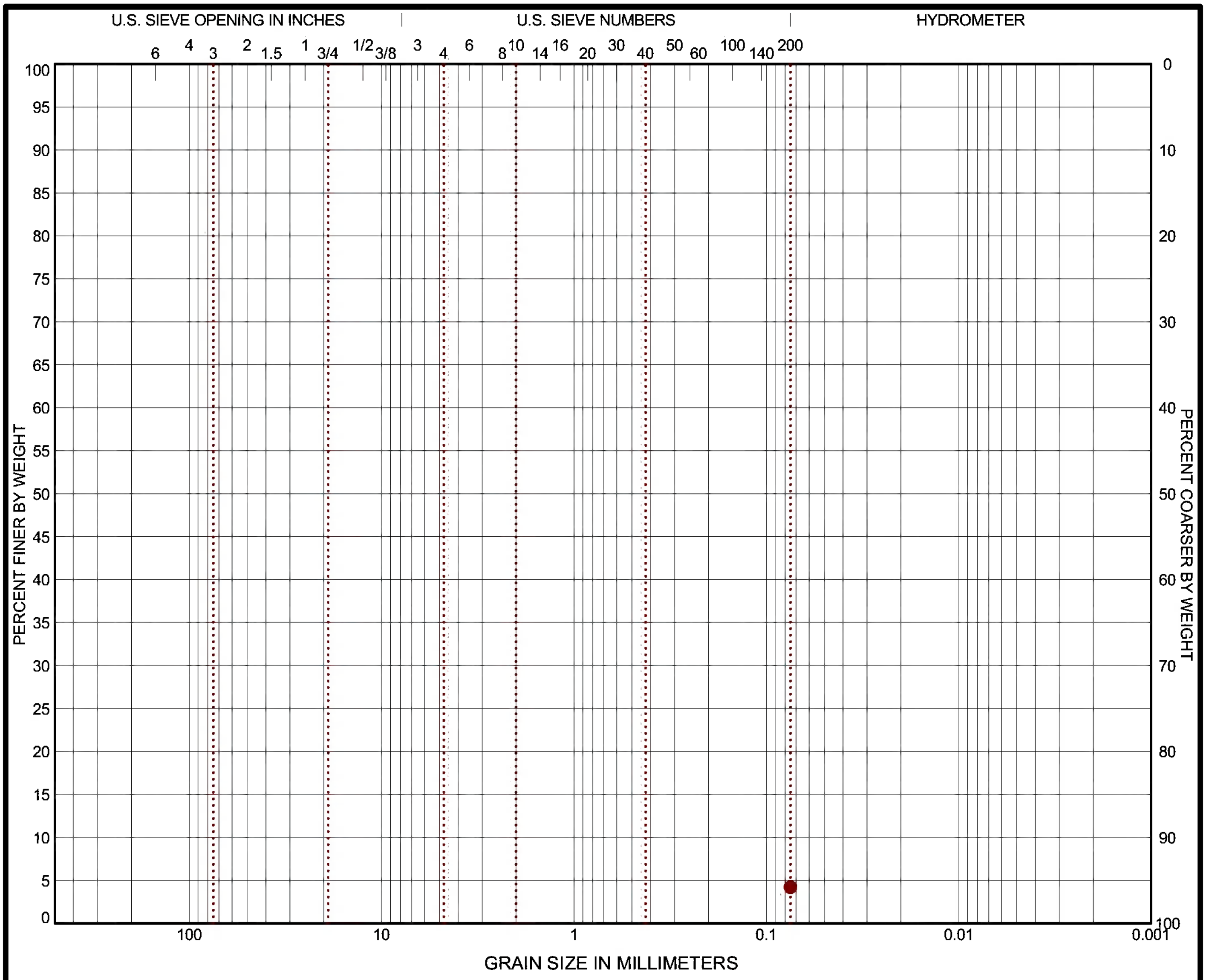
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-305**	**30 - 32**					**4.2**		

	GRAIN SIZE		
☒	●		
D_{60} D_{30} D_{10}			
☒	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

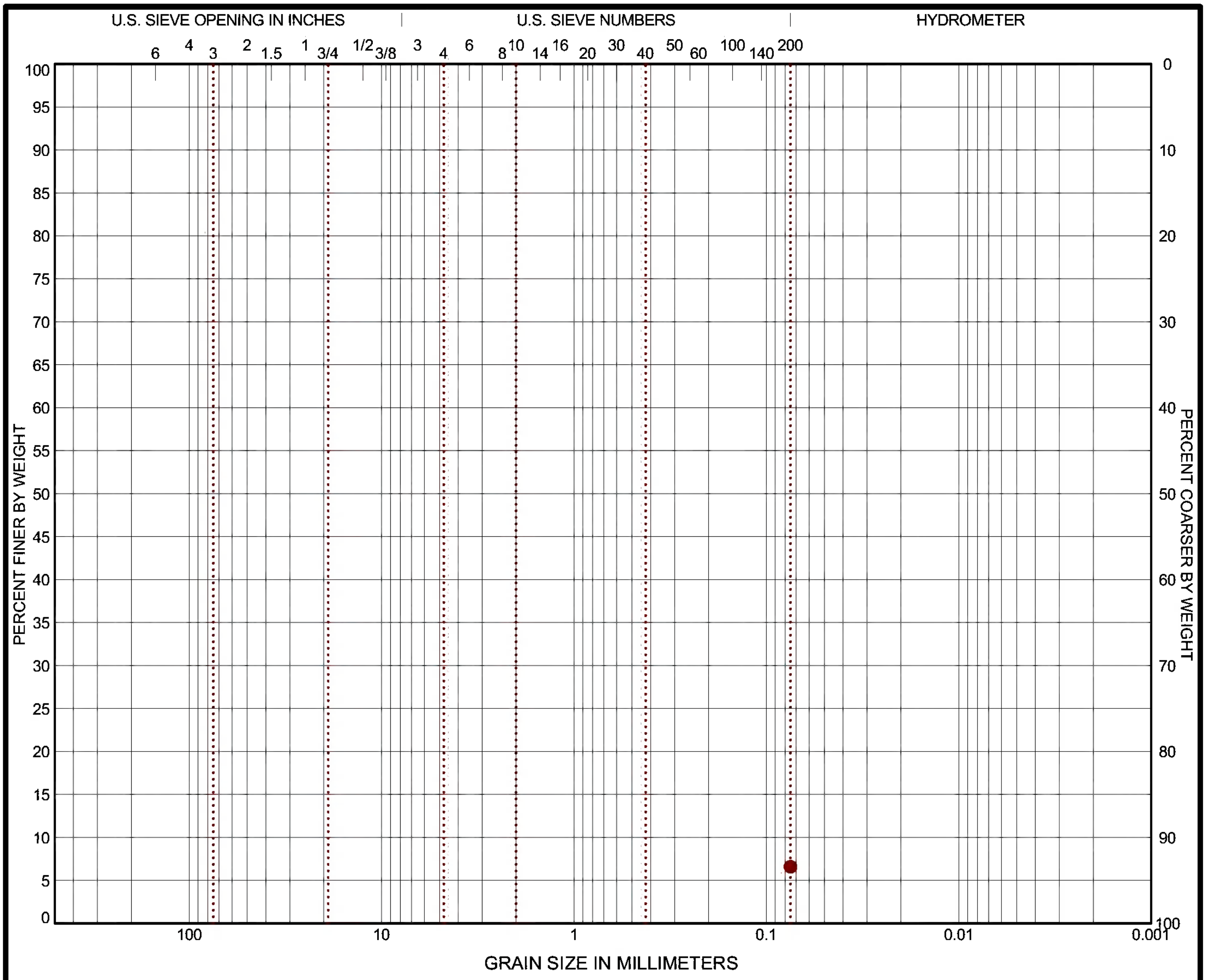
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-17

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-305**	**38 - 40**					**6.6**		

	GRAIN SIZE		
	●		
D_{60} D_{30} D_{10}			
	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

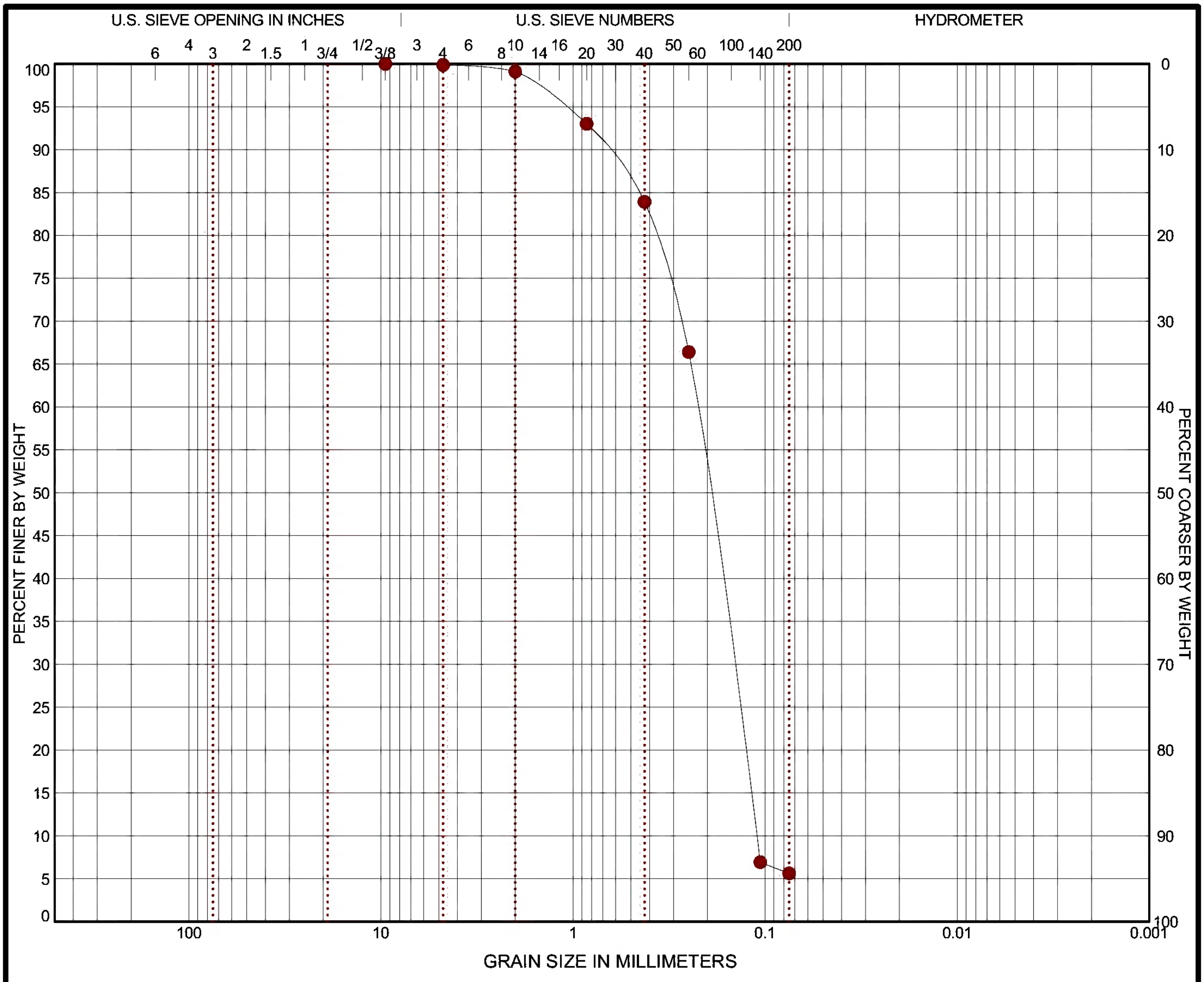
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-18

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-305**	**48 - 50**	**0.0**	**0.1**	**94.2**		**5.6**		

	GRAIN SIZE		
	●		
D_{60}	**0.228**		
D_{30}	**0.148**		
D_{10}	**0.111**		
	COEFFICIENTS		
C_C	**0.87**		
C_U	**2.06**		

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

1450 Fifth St W
North Charleston, SC

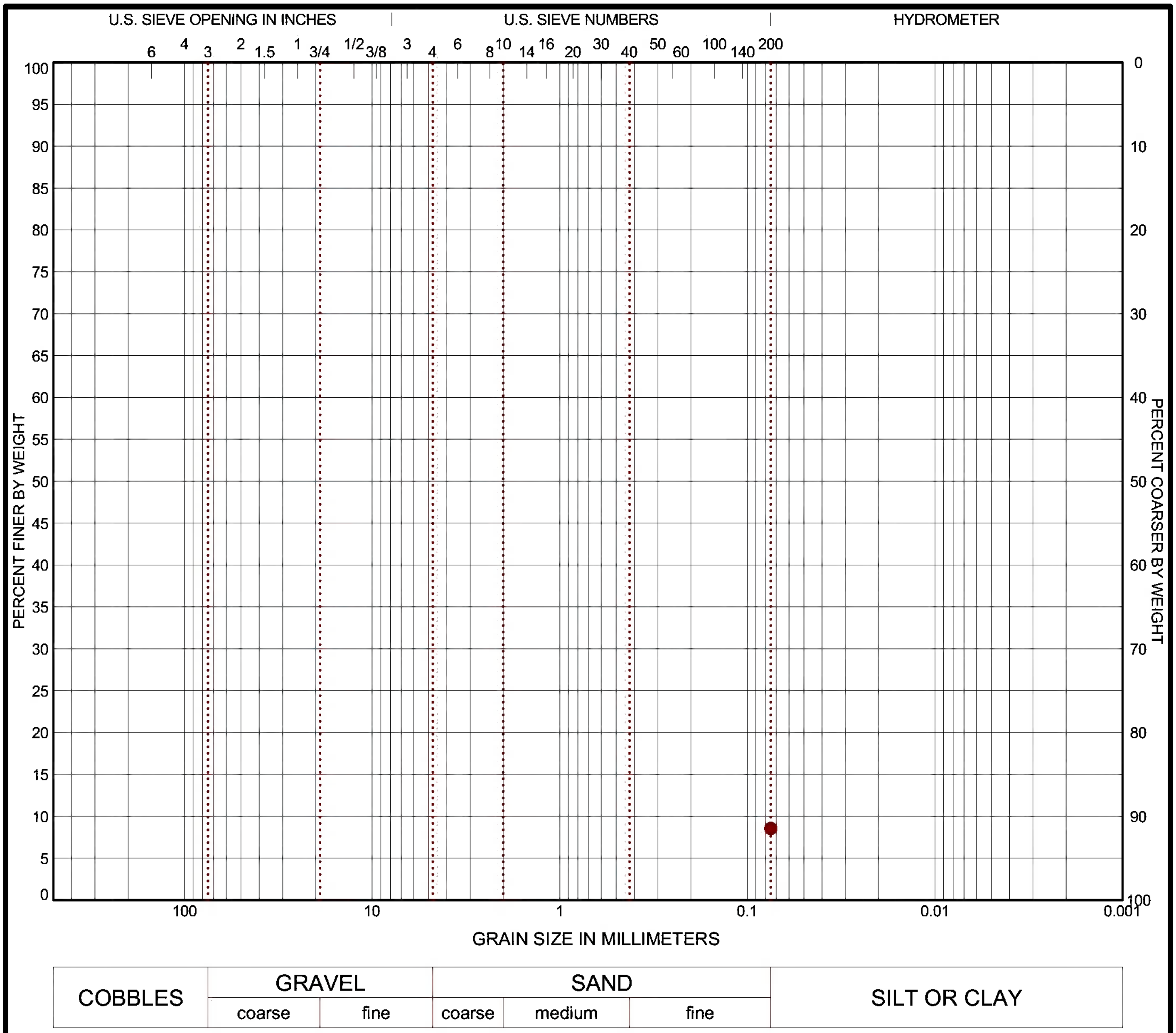
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-19

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**23.5 - 25**					**8.6**		

	GRAIN SIZE		
☒	●		
D_{60} D_{30} D_{10}			
☒	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

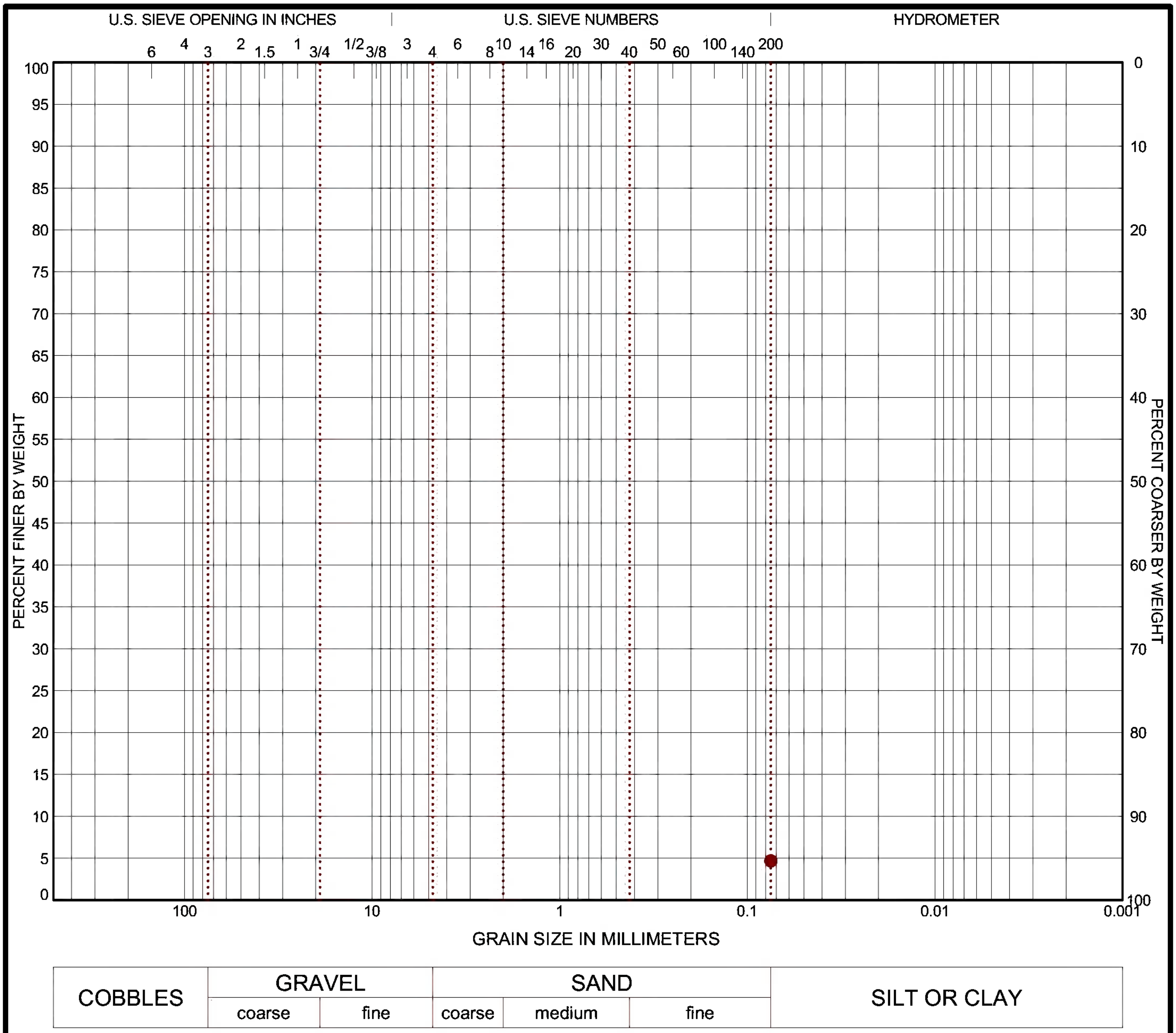
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-20

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**32 - 34**					**4.7**		

	GRAIN SIZE		
	●		
D$_{60}$			
D$_{30}$			
D$_{10}$			
	COEFFICIENTS		
C$_{C}$			
C$_{U}$			

SIEVE (size)	PERCENT FINER		
	●		
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
#4			
#10			
#20			
#40			
#60			
#100			
#200	**4.69**		

SOIL DESCRIPTION

●

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

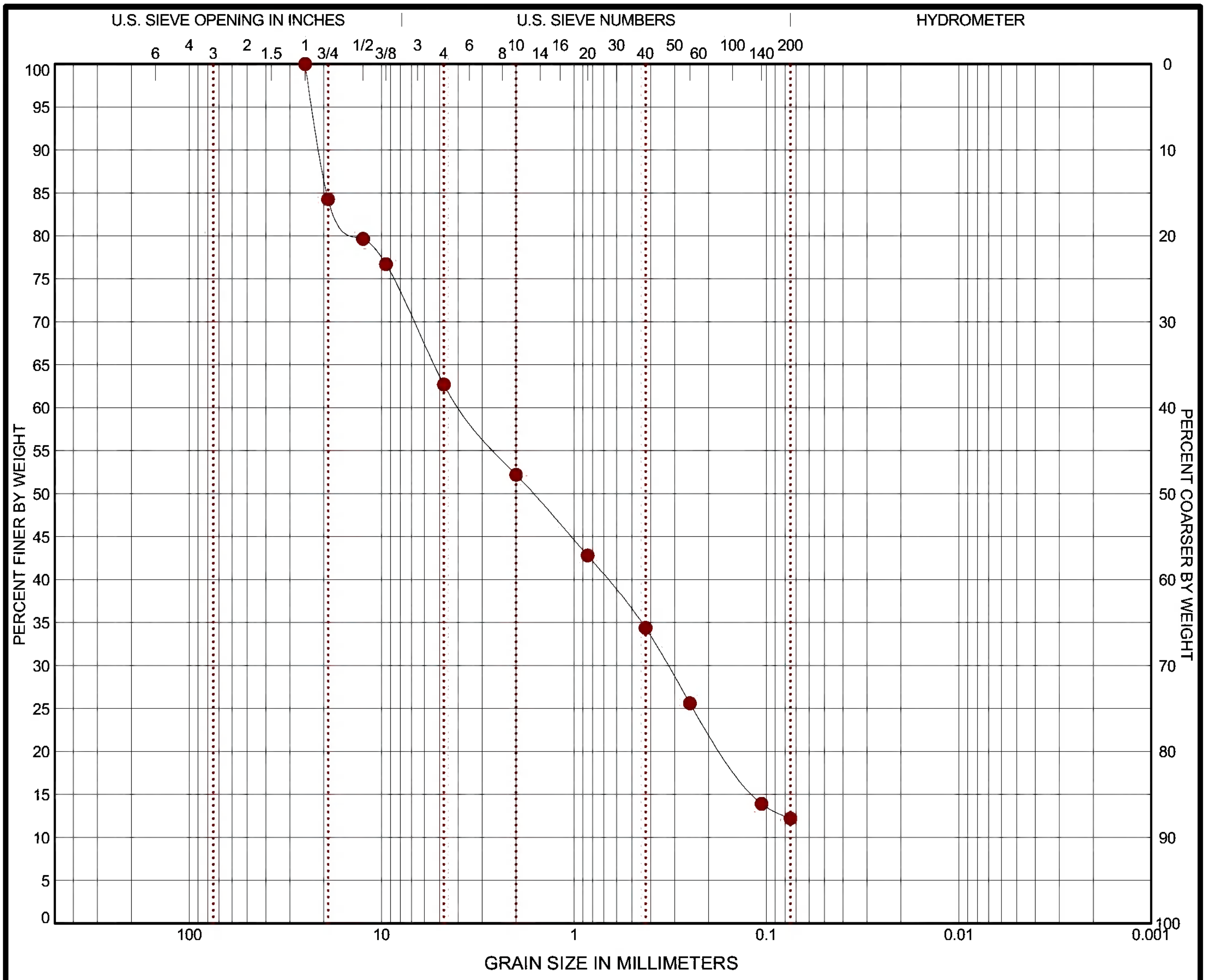
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-21

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>coarse</td><td>fine</td><td>coarse</td><td>medium</td><td>fine</td></tr>
</table>

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**53.5 - 55**	**0.0**	**37.3**	**50.5**		**12.2**		**SM**

	GRAIN SIZE		
	●		
D_{60}	**3.802**		
D_{30}	**0.326**		
D_{10}			
	COEFFICIENTS		
C_C	**0.58**		
C_U	**79.29**		

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILTY SAND with GRAVEL (SM)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

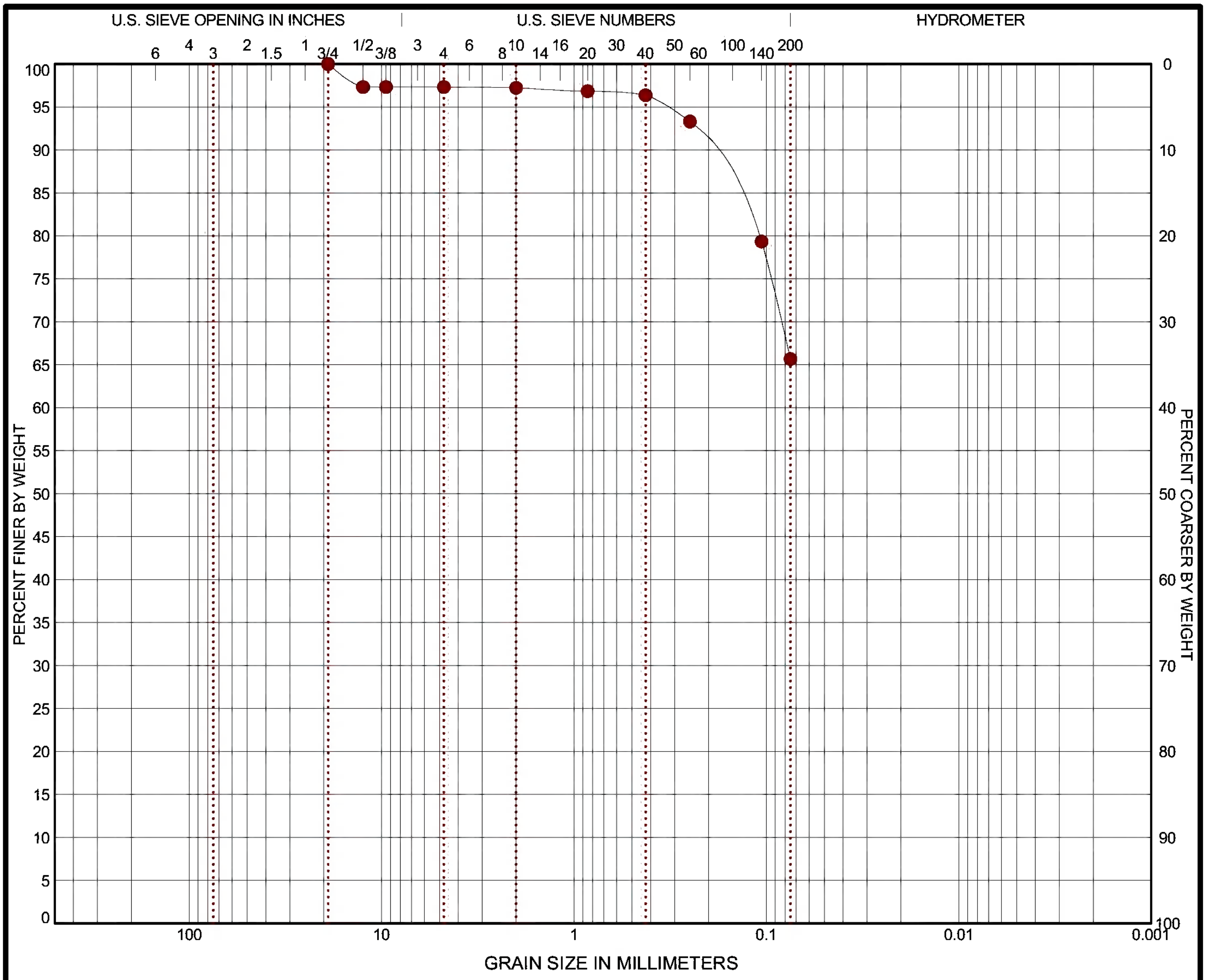
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-22

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	SPT-306	73.5 - 75	0.0	2.7	31.7		65.7		CH

	GRAIN SIZE		
	●		
D_{60} D_{30} D_{10}			
	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SANDY FAT CLAY (CH)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

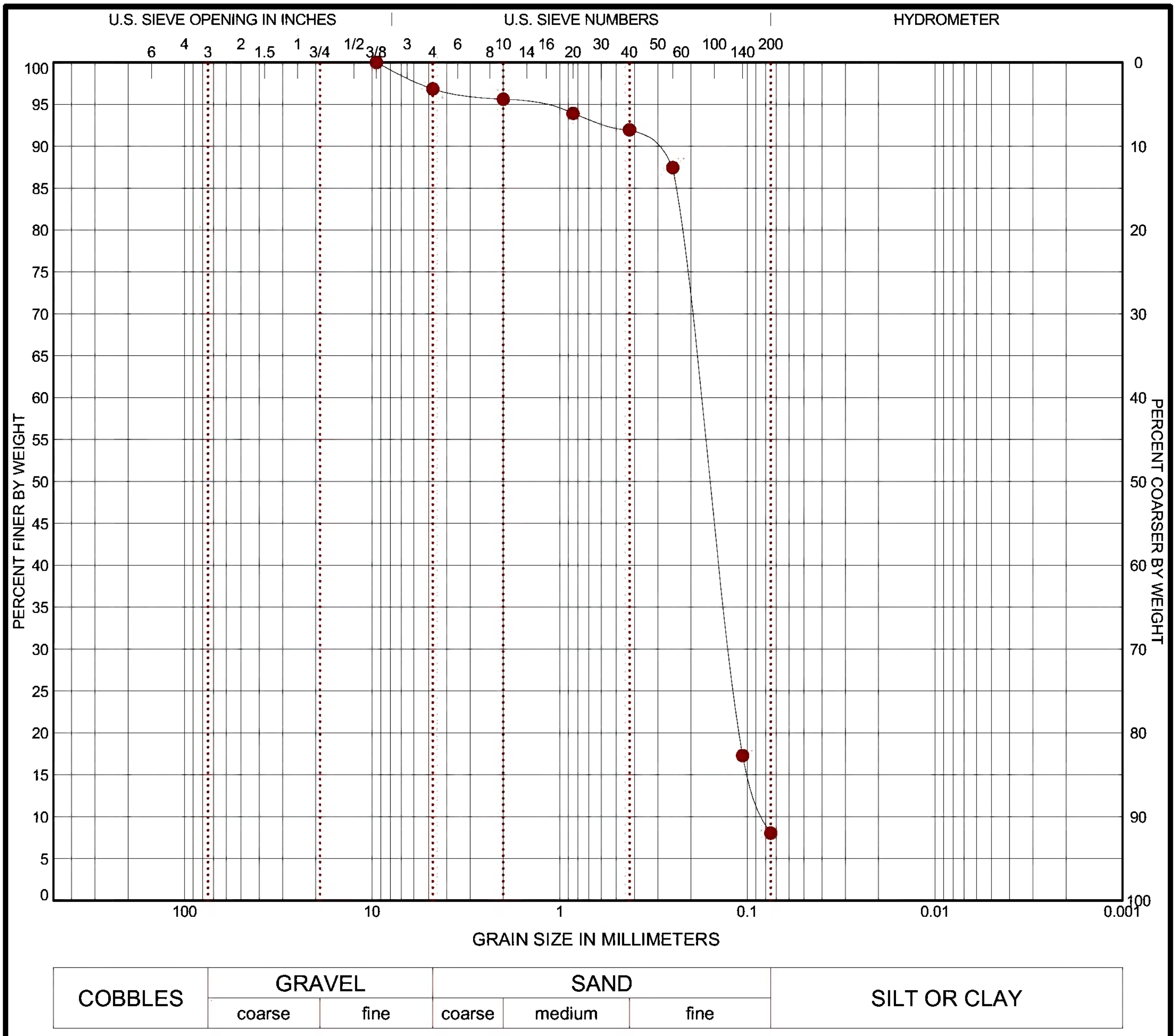
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-23

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**93.5 - 95**	**0.0**	**3.2**	**88.8**		**8.0**		**SP-SM**

	GRAIN SIZE		
	●		
D_{60}	**0.179**		
D_{30}	**0.124**		
D_{10}	**0.081**		
	COEFFICIENTS		
C_C	**1.06**		
C_U	**2.21**		

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● POORLY GRADED SAND with SILT (SP-SM)

REMARKS

●

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

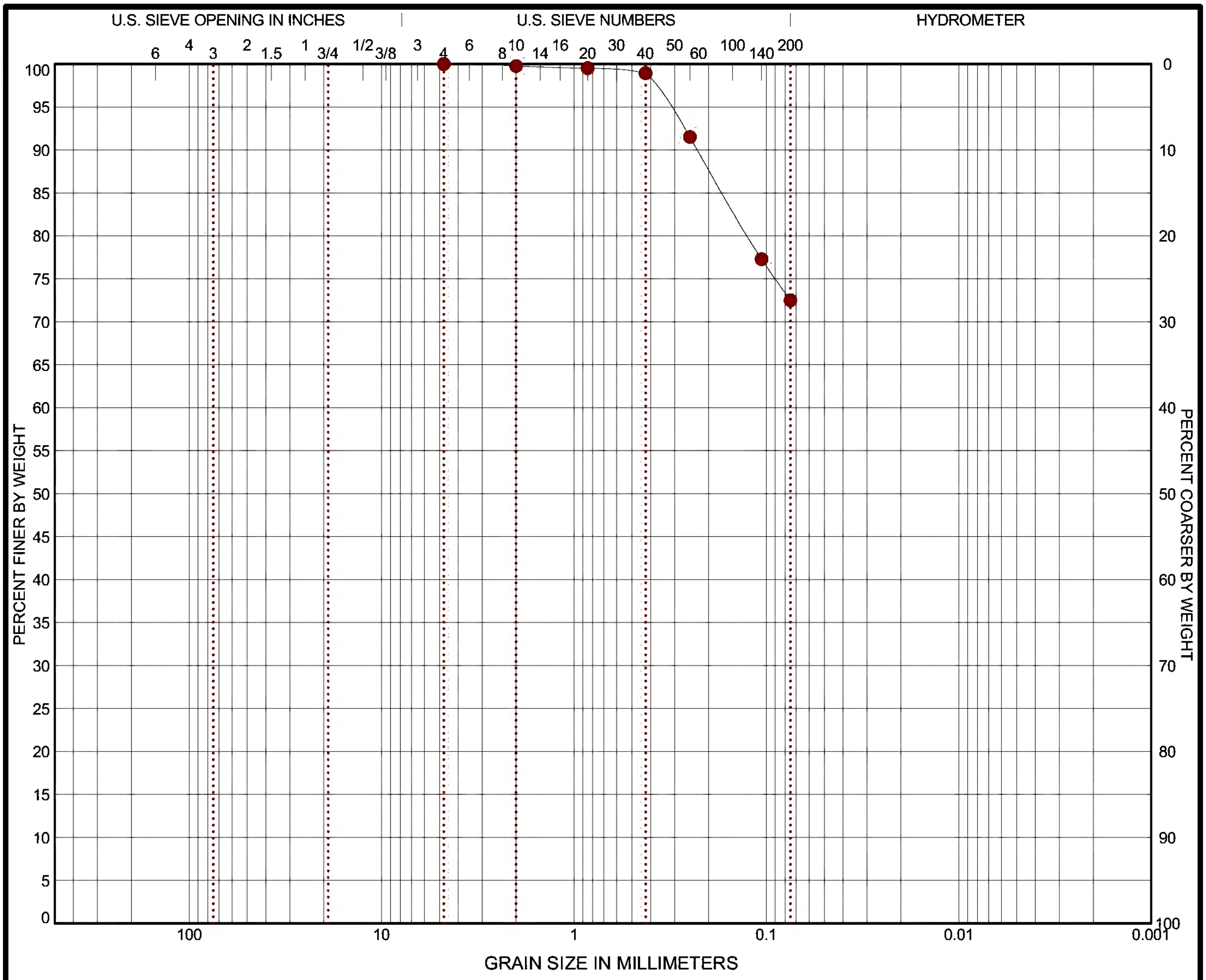
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-24

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	SPT-306	108.5 - 110	0.0	0.0	27.5		72.5		MH

	GRAIN SIZE		
	●		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● ELASTIC SILT with SAND (MH)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

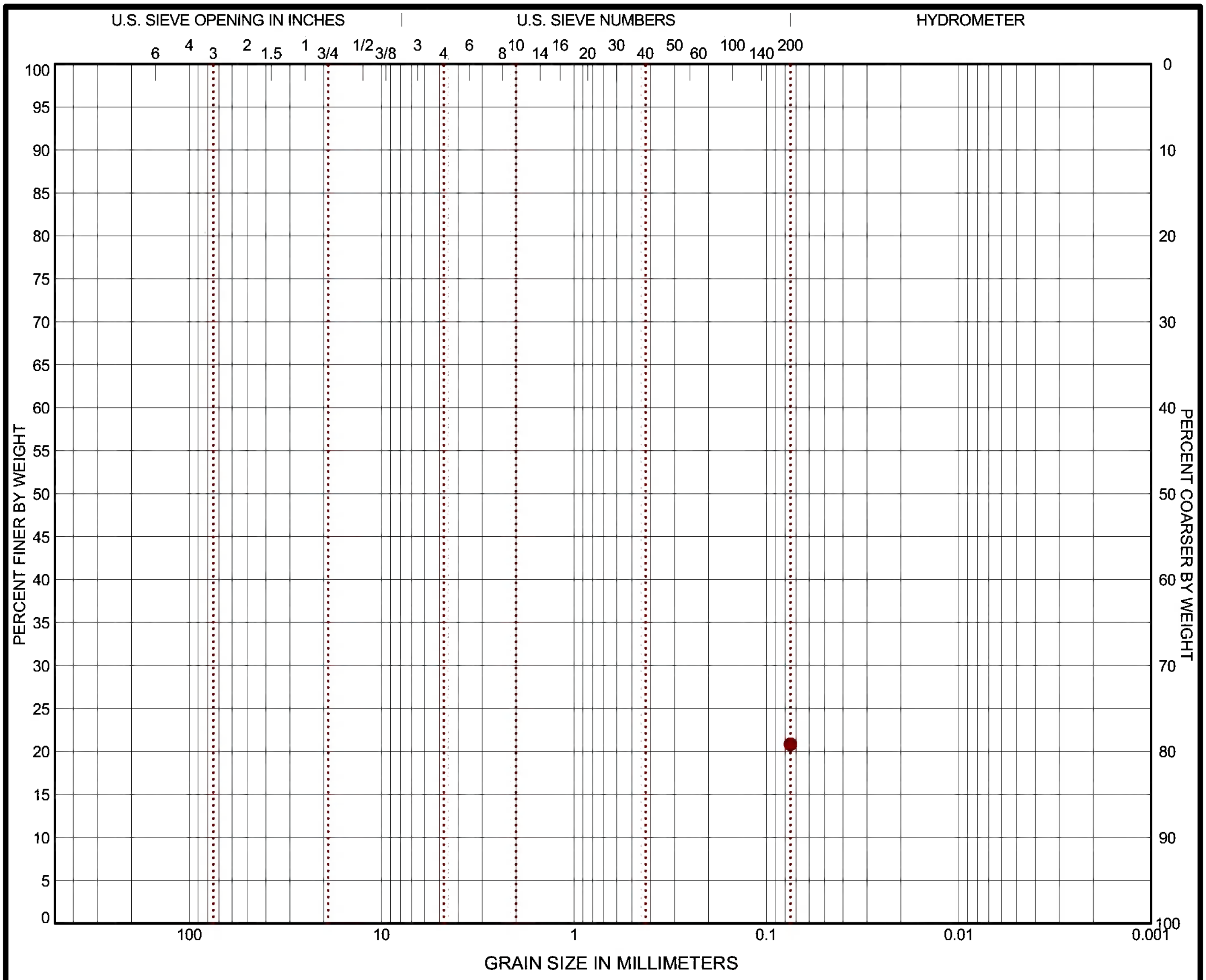
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-25

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**123.5 - 125**					**20.8**		**SM**

	GRAIN SIZE		
☒	●		
D_{60}			
D_{30}			
D_{10}			
☒	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILTY SAND (SM)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

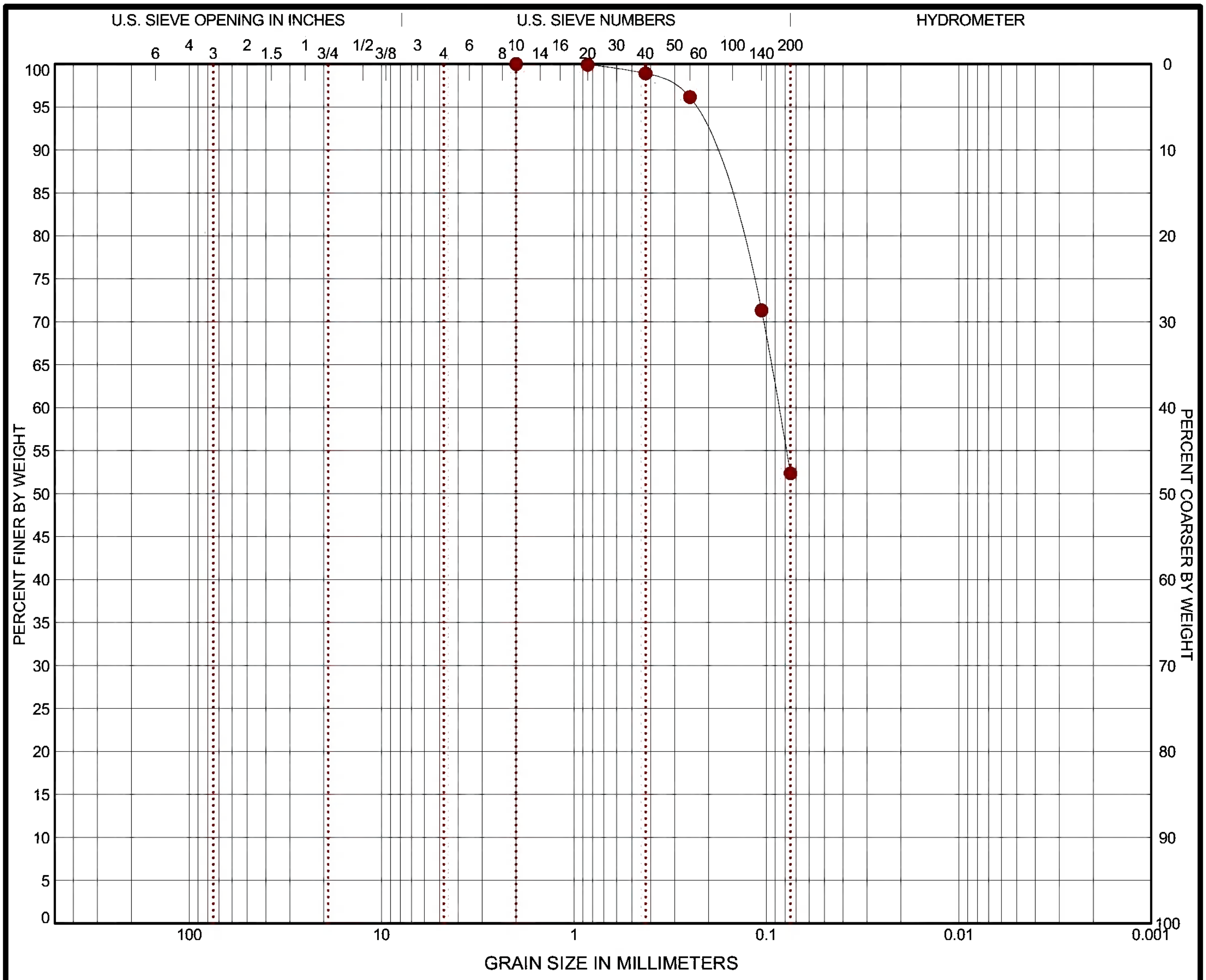
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-26

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**138.5 - 140**	**0.0**	**0.0**	**47.6**		**52.4**		**CL**

	GRAIN SIZE		
	●		
D_{60} D_{30} D_{10}	**0.086**		
	COEFFICIENTS		
C_C C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SANDY LEAN CLAY (CL)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

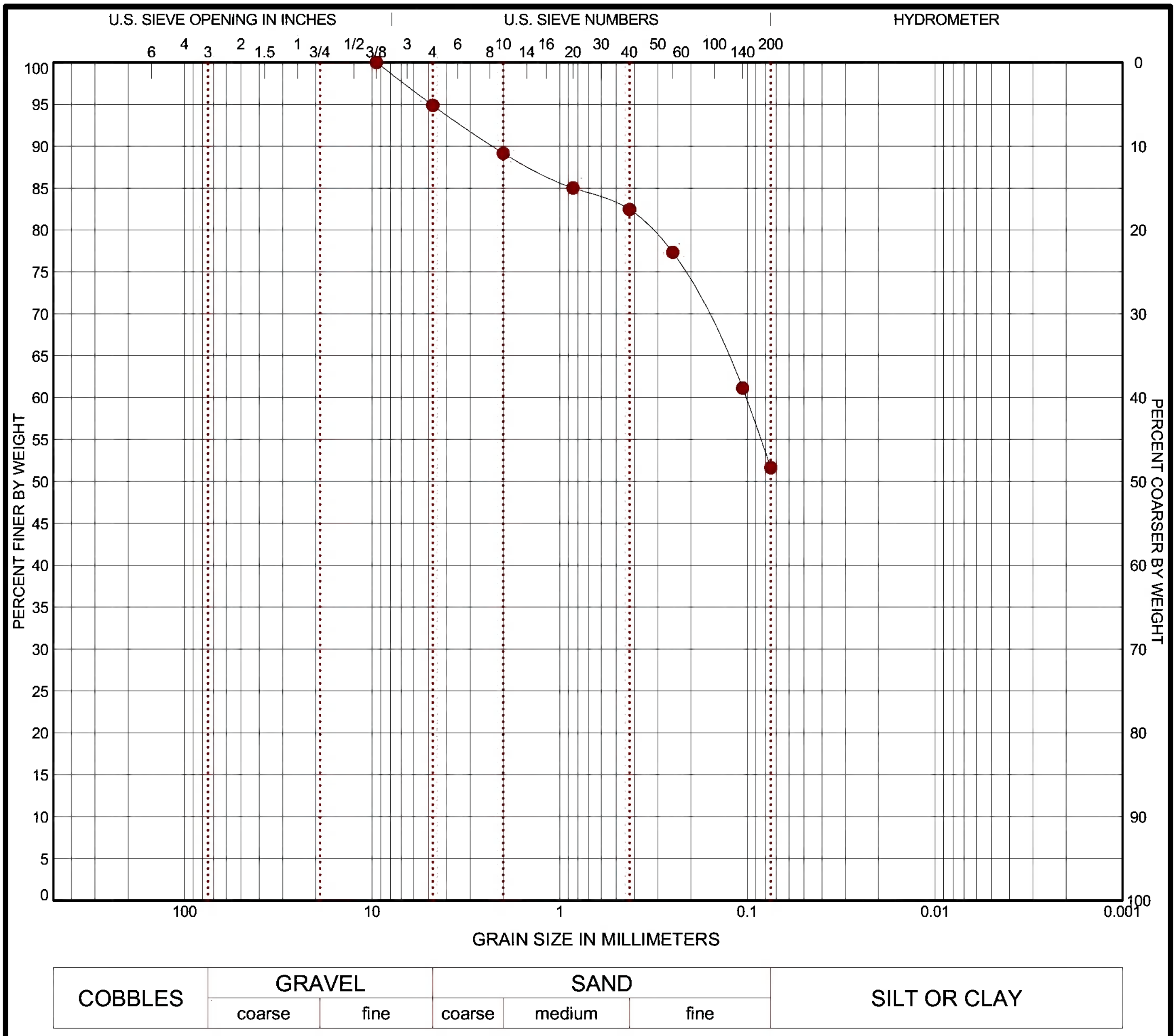
PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-27

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**148.5 - 150**	**0.0**	**5.1**	**43.2**		**51.6**		**ML**

	GRAIN SIZE		
	●		
D_{60}	**0.102**		
D_{30}			
D_{10}			
	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SANDY SILT (ML)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

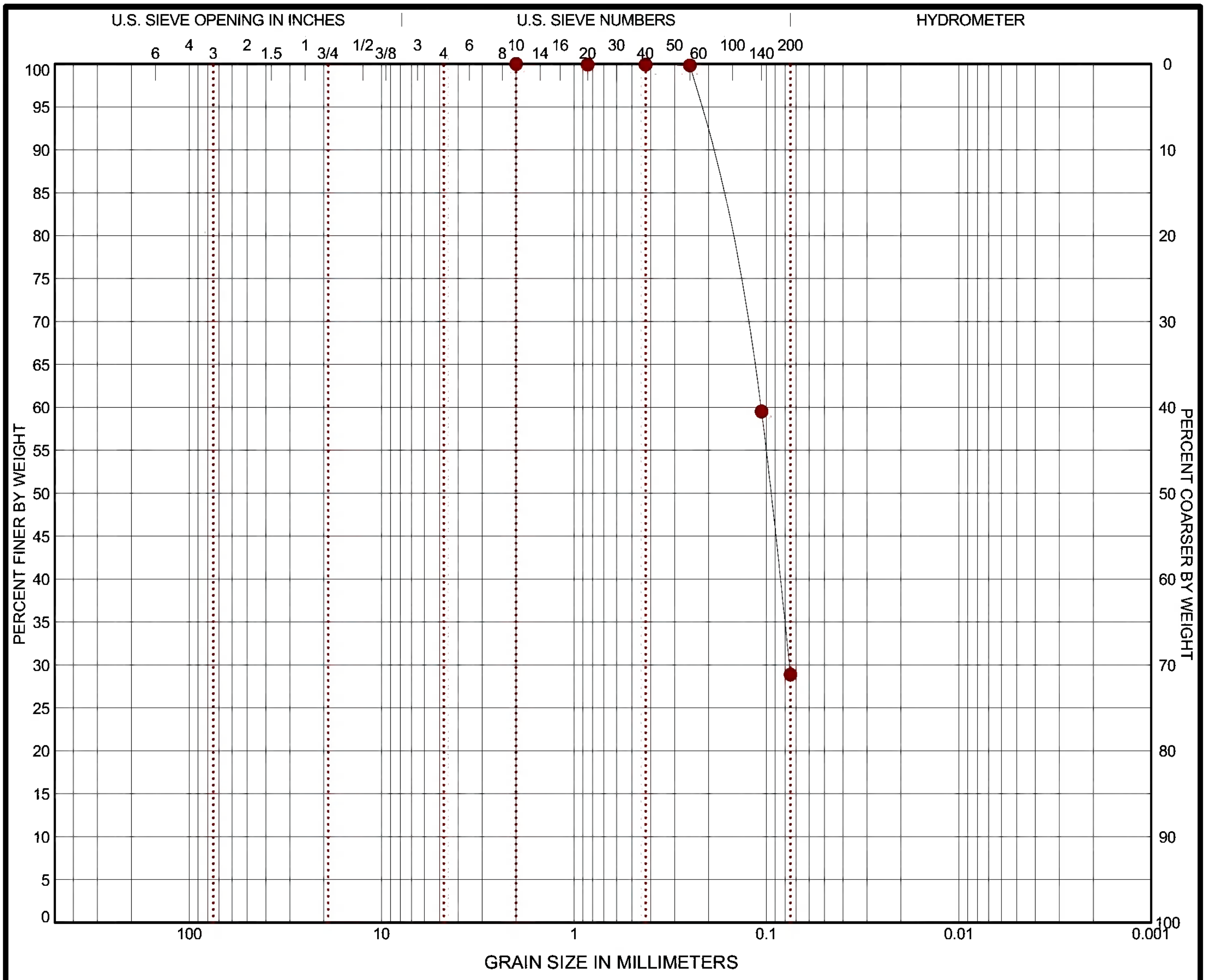
CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-28

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**158.5 - 160**	**0.0**	**0.0**	**71.1**		**28.9**		**SM**

	GRAIN SIZE		
	●		
D_{60}	**0.107**		
D_{30}	**0.076**		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE (size)	PERCENT FINER		
	●		

SOIL DESCRIPTION

● SILTY SAND (SM)

REMARKS

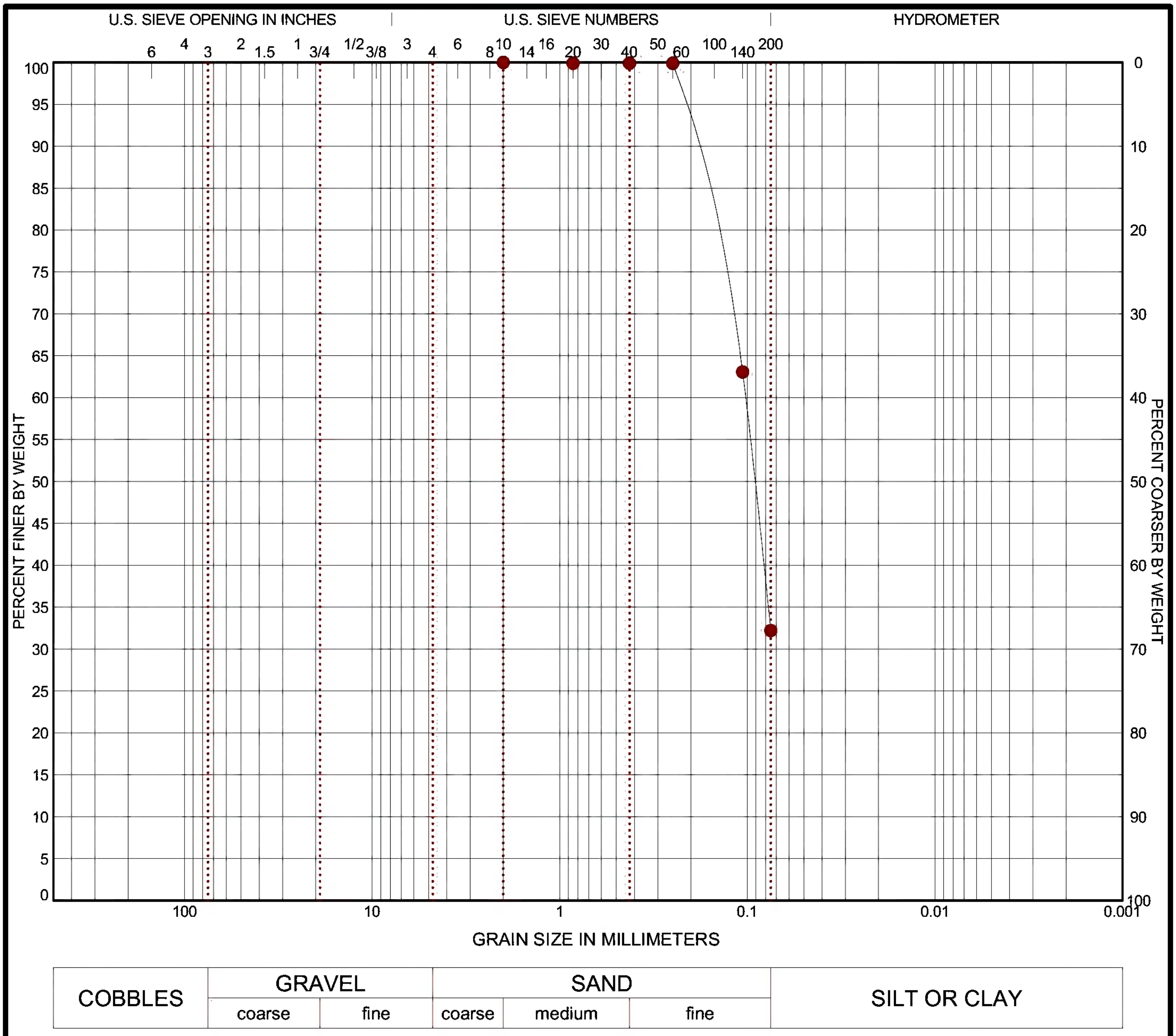
●

PROJECT: Winyah Generation Station	Terracon	PROJECT NUMBER: EN165065
SITE: Georgetown, South Carolina	1450 Fifth St W North Charleston, SC	CLIENT: Santee Cooper Moncks Corner, South Carolina
		EXHIBIT: B-29

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS 1 EN165065 WINYAH GENERATION STATION.GPJ 5/13/16

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**SPT-306**	**163.5 - 165**	**0.0**	**0.0**	**67.8**		**32.2**		**SM**

	GRAIN SIZE		
	●		
D_{60}	**0.102**		
D_{30}			
D_{10}			
	COEFFICIENTS		
C_C			
C_U			

SIEVE (size)	PERCENT FINER		
	●		
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
#4			
#10	**100.0**		
#20	**99.9**		
#40	**99.9**		
#60	**99.9**		
#100			
#200	**32.21**		

SOIL DESCRIPTION

● SILTY SAND (SM)

REMARKS

●

PROJECT: Winyah Generation Station

SITE:
Georgetown, South Carolina

Terracon
1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-30

ATTACHMENT 5

Subsurface Stratigraphy and Material Properties

Geosyntec
consultants

CALCULATION PACKAGE COVER SHEET

Client: Santee Cooper **Project:** Winyah Generating Station **Project No.** GSC5242

TITLE OF PACKAGE: SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES: ASH POND A

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Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES: ASH POND A

INTRODUCTION

This calculation package was prepared to present the subsurface stratigraphy and material properties supporting geotechnical analyses for Ash Pond A located at Winyah Generating Station (WGS or "Site"), which is owned and operated by Santee Cooper. This calculation package is an attachment to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* (Safety Factor Assessment Report) prepared by Geosyntec Consultants (Geosyntec). The remainder of this calculation package presents the: (i) site investigations; (ii) subsurface stratigraphy; (iii) interpretation of the phreatic surface and current water levels; (iv) standard penetration test (SPT) and cone penetration test (CPT) interpretation; (v) laboratory testing program and interpretation; (vi) in-situ testing interpretation; and (vii) selected material properties for analysis.

SITE INVESTIGATIONS

In October 2013, Geosyntec collected geotechnical subsurface data at WGS through soil borings and CPT soundings in support of evaluating preliminary and conceptual closure alternatives for each CCR surface impoundment. The subsurface investigation was focused in the vicinity of the South Ash Pond, Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B. In the Ash Pond A area, Geosyntec advanced seven soil borings by the mud rotary wash drilling method in general accordance with recommendations made by Idriss and Boulanger (2008) (Table 1) and sixteen CPT soundings. The location of each soil boring and CPT sounding in the area of Ash Pond A is shown in Figure 1. Soil Consultants, Inc. (SCI) of Charleston, South Carolina, was the drilling contractor during this investigation. Mid-Atlantic Drilling, Inc. (MAD) from Wilmington, North Carolina performed the CPT soundings. One soil boring and four CPT soundings were advanced within the interior of Ash Pond A and were terminated once native or foundation materials were encountered. The remaining soil borings and CPT soundings were performed on the perimeter and divider dikes and were terminated once refusal was encountered, which was defined as a SPT blow count of 50 blows per foot over an advancement of 6" or the inability to further advance the cone.

During each soil boring, split spoon samples were collected and SPT blow counts (i.e., N-values) were recorded typically in 5-ft depth intervals. Three Shelby tubes were pushed to collect samples in the cohesive foundation soils located in the northwest corner of Ash Pond A. Several Shelby tubes were pushed to attempt to collect samples within the Ash Pond A perimeter dikes; however, the recompacted dike fill soils were found to be dense and cohesionless and thus, undisturbed samples were unable to be collected. In one soil boring (SPT-117), SCI utilized a tri-cone rotary wash drill bit instead of the side discharge flat drilling bit once the Chicora Member stratum was encountered to penetrate the unit and advance into the underlying formation (i.e., Williamsburg Formation Clay). In SPT-117, a Shelby tube was pushed to collect a sample of the underlying stiff clay for geotechnical laboratory testing. During this geotechnical subsurface investigation, shear wave velocities (V_s) were measured in 5-ft depth intervals at seven CPT soundings (CPT-135, 137, 140, 144, 145, 147, and 150). Additionally, dissipation tests were performed at five CPT soundings (CPT-138, 143, 146, 155, and 157) to evaluate the phreatic surface through the perimeter dikes and within Ash Pond A at the time of the investigation. Soil boring logs and CPT sounding data, including V_s and dissipation tests, are provided in Attachments 2 and 3, respectively.

In November 2013, Geosyntec installed piezometers as part of the development of a hydrogeological model at WGS. Two piezometers (PPZW-8D and PPZW-9D) were installed adjacent to Site monitoring wells (WAP-8 and WAP-9). Prior to installing these piezometers, subsurface soils were collected using a split spoon sampler and logged by a Geosyntec geologist. SPT N-values measured during this installation were interpreted and utilized as a part of this subsurface assessment.

In the spring of 2016, Geosyntec performed a geotechnical subsurface investigation predominantly within the interior of Ash Pond A, Ash Pond B, and the Unit 2 Slurry Pond to collect information in support of the design of closure options for each surface impoundment. Within the Ash Pond A interior and along the divider dike, Terracon was subcontracted and performed twelve CPT soundings to evaluate the subsurface stratigraphy underlying the surface impoundments. Three additional CPT soundings (CPT-228, CPT-229, and CPT-229A) were advanced at the perimeter dike crest and dike toe adjacent to the Cooling Pond (east side of Ash Pond A). Additionally, Terracon advanced three soil borings (SPT-304, SPT-305, and SPT-306) within the Ash Pond A interior to collect soil samples for laboratory testing. The laboratory testing program for soil samples collected during this investigation consisted of particle size distribution analysis, moisture content tests, and Atterberg limits tests.

SUBSURFACE STRATIGRAPHY AND RESIDUAL MATERIALS

Subsurface Stratigraphy

The subsurface stratigraphy at WGS was developed based on information collected during Geosyntec's geotechnical site investigations and from site wide geotechnical data from previous subsurface investigations. Boring logs from the Geosyntec investigation are provided in Attachment 2 of this Safety Factor Assessment Report. The general subsurface stratigraphy is described as follows:

- Dike Fill Soils: Dike fill soils were generally observed to be medium dense to very dense, poorly graded silty sands with uncorrected SPT blow counts typically ranging between 7 and 66 blows per foot and measured CPT tip resistances typically ranging between 100 and 450 tsf. Grain size distribution analyses indicated that the dike fill soils typically consist of 72 percent to 87 percent sand-sized particles (smaller than a No. 4 sieve but greater than a No. 200 sieve and 6 percent to 28 percent silt and clay-sized particles (i.e., "fines" with diameters smaller than a No. 200 sieve), with most samples containing less than 15 percent fines.
- Foundation Soils: Foundation soils were observed to be variable across the Ash Pond A footprint. The foundation materials consist primarily of poorly graded silty sands with shells

and a few isolated seams of clayey sand or high plasticity clay. Uncorrected SPT blow counts within foundation soils ranged between 0 and 61 blows per foot, with clayey material generally having a lower measured blow count than sandy material. Tip resistances generally ranged between 25 and 300 tsf (generally below 50 tsf).

- Chicora Member: A layer of dense to very dense soil consisting of partially cemented to heavily cemented shells was encountered beneath the foundation soils during subsurface investigations at WGS. SPT blow counts in this layer exceeded 50 blows over less than 6 in. of advancement with minimal sample recovery. The thickness of this layer, particularly the cemented layers of the material, varied across the Site. Based on review of historical and existing data (Doar, 2012), this layer is the upper portion of the overall Williamsburg Formation and is referred to as the "Chicora Member", "Coquina", or "Shell Hash". The term "Chicora Member" or "Chicora" is used to refer to this soil unit throughout the Safety Factor Assessment Report. Boring and CPT refusal was typically encountered at the top of this stratum, though two soil borings within the Ash Pond A area penetrated this stratum.

- Williamsburg Formation Clay: The Williamsburg Formation Clay stratum was encountered beneath the Chicora Member. The Williamsburg Formation Clay is described as stiff to very hard, dark gray to black, medium to high plasticity clay or silt with sand. The Williamsburg Formation Clay has historically been referred to as "Black Mingo Clay" or the "Black Mingo Formation" at the Site. The term "Williamsburg Formation Clay" is the most recent geological term for this stratum and is used throughout the Safety Factor Assessment Report. The Williamsburg Formation Clay was found to be between 30-ft to 90-ft thick in the vicinity of WGS based on a review of the regional geology.

Coal Combustion Residuals

CCRs in the form of fly ash, boiler slag, and bottom ash have been stored within Ash Pond A. Four soil borings and twelve CPT soundings were performed in the CCRs contained within Ash Pond A during Geosyntec's subsurface investigations. Ash Pond A contains predominantly fly ash, which was described as follows:

- Fly Ash: Fly ash was found to be soft, black, sandy silt with SPT blow counts between 0 (i.e., weight of hammer) and 2 blows per foot. The measured CPT tip resistance of ponded fly ash ranged between 5 tsf and 75 tsf, with most values below 15 tsf. It is noted that most of the higher tip resistance values were observed within the upper 5 ft below ground surface (ft bgs).

PHREATIC SURFACE INTERPRETATION AND CURRENT WATER LEVELS

During each site investigation by Geosyntec, water levels from rotary wash borings located on the dike centerline were measured 24 hours after borehole termination and daily until borehole abandonment. CPT soundings were advanced with a porewater pressure transducer, which recorded porewater pressure measurements during advancement, located behind the cone. The measured porewater pressures were interpreted to locate the phreatic surface at the time of each sounding. Dissipation tests were conducted during the CPT soundings at several locations. Excess porewater pressures were allowed to dissipate to equilibrium or hydrostatic conditions over 5 to 30 minutes depending on the rate of porewater pressure dissipation. Once hydrostatic conditions were observed, the measured pressure was utilized to compute the elevation of the phreatic surface during CPT sounding. The measured phreatic surface level at each location is summarized in Table 2.

Dike Phreatic Surface

During the recent site investigations, six soil borings and ten CPT soundings were advanced through the perimeter dike centerline. Dissipation tests were performed during several of the CPT soundings through the Ash Pond A perimeter dikes. The measurements of the phreatic surface at these locations indicate that the phreatic surface elevation through the perimeter dikes ranges from 22 to 29 ft National Geodetic Vertical Datum of 1929 (NGVD29).

Free Field (Dike Toe) Phreatic Surface

During the 2013 site investigation, six CPT soundings were advanced and two piezometers (PPZW-8D and PPZW-9D) were installed along the downstream toe of the Ash Pond A perimeter dikes near the Intake and Discharge Canals. Groundwater elevation measurements from two monitoring wells were collected monthly and measurements in May 2015 were included in this evaluation. Porewater pressure (u_0) signatures, piezometer, and monitoring well measurements indicate that the phreatic surface elevation along the toe of this area ranges from 21 to 24 ft NGVD29.

Water Levels Since 2013 Investigation

As described within Attachment 1 of the Safety Factor Assessment Report, the free water level within Ash Pond B, located to the south of Ash Pond A, is maintained at an elevation of 34.9 ft NGVD29 by a 4-ft by 4-ft concrete riser structure. Ash Pond A does not typically contain free water, but the surface impoundment does convey stormwater and process water through a series of rim ditches to Ash Pond B. A 30-in diameter corrugated metal pipe (CMP), a 48-in diameter smooth steel pipe, and a 42-in diameter smooth steel pipe convey free water from the rim ditches through the northeast corner of the divider dike into Ash Pond B. The concrete riser structure in Ash Pond B maintains free water at an operating water level of 34.9 ft NGVD29 and discharges free water westward into the Discharge Canal.

Temporary piezometer (PPZ-AS-1) installed within the Ash Pond A interior indicates that the phreatic elevation within the CCR at the center of Ash Pond A has ranged between 36.0 and 37.2 ft NGVD29 since installation. Thus, the phreatic surface elevation within the center of the surface impoundment was selected as 37.2 ft NGVD29 and assumed to transition to 34.9 ft NGVD29 adjacent to the perimeter dikes. The free water elevation of the Cooling Pond was selected as 19.1 ft NGVD29 based on the operating level of the Cooling Pond required to manage runoff from the 25-yr, 24-hr rainfall event. The maximum free water elevation during the IDF within Ash Pond A was computed as 38.2 ft

NGVD29, which was used to represent the "Maximum Surcharge Pool" level within the Safety Factor Assessment Report.

In 2015, WGS installed supplementary groundwater monitoring wells (WAP-12, WAP-17, WAP-18, and WAP-19) at the downstream dike toe and perimeter dike crest of Ash Pond A. Groundwater elevations have been measured on a quarterly basis since installation and the most recent available measurements are provided in Table 2. On 21 June 2016, the phreatic surface elevation was measured as 23.8 ft NGVD29 at the dike toe and between 25.9 ft and 26.7 ft NGVD29 through the dike crest.

SPT AND CPT INTERPRETATION

Results of SPT and CPT sounding data were processed and interpreted by the methods described below.

Standard Penetration Test Interpretation

During a SPT, the number of "blows" or impacts from a standard, 140-lb hammer falling 30 inches needed to advance the split spoon sampler 6-inches is recorded over 3 intervals for a total of 18 inches. The blows for the last two 6-inch intervals are summed and referred to as an "N-value". Due to variations in drill rigs, hammer efficiency, and sampling methods, the field or measured value must be corrected to a standard value for use in engineering correlations and computations. This standard value is based on a hammer system that is 60 percent efficient (i.e., applies 60 percent of the theoretical maximum potential energy). The corrected N-value (N_{60}) is computed as follows:

$$N_{60} = N_{meas} C_E C_B C_S C_R \tag{1}$$

where:

N_{60} = N-value corrected to 60 percent efficiency (blows/ft);
N_{meas} = measured N-value in the field (blows/ft);
C_E = correction factor for the applied energy of the hammer;
C_B = correction factor for the borehole diameter;
C_S = correction factor for the sampling method; and
C_R = correction factor for the rod length.

The correction factor for the applied energy (C_E) of the hammer is often variable between drilling rigs and hammer type. This correction factor can be computed as follows:

$$C_E = \frac{ER}{60} \tag{2}$$

where:

ER = energy ratio of the SPT hammer.

SCI provided calibration records for the hammer system of the CME-550X drilling rig used to advance the soil borings based on calibration tests performed offsite on 3 April 2013. An Energy Ratio (ER) of 88 percent was computed for the CME-550X drilling rig (calibration records are provided in Table 3). N-values measured during the installation of two piezometers (PPZW-8D and PPZW-9D) in November 2013 were also utilized during this evaluation. These piezometers were installed by South Atlantic Environmental Drilling and Construction Co. Inc. (SAEDACCO). The drilling rig utilized by SAEDACCO was calibrated by GRL Engineers, Inc. (GRLE) on 30 July 2013 prior to mobilizing to the site and measured to have an ER of 87 percent (GRLE, 2013). The CME-45C drilling rig utilized by Carolina Drilling, Inc. in the 2016 site investigation was computed to have an ER of 79.3 percent as shown in Table 4.

Values for the other correction factors were selected based on industry standards (Idriss and Boulanger, 2008) and are provided in Table 5. N_{60} was computed for the soil borings based on a 4-inch diameter borehole (101.6 mm) and a standard split spoon sampler. Rod length for the C_R conversion factor was selected based on the depth of the measured SPT blow counts and a 5-ft stickup from the length of the drilling rod and anvil above the top of the borehole.

In many correlations, N_{60} is normalized based on the in-situ stress state at the time of boring. The normalized and corrected blow count is referred to as $(N_1)_{60}$ and is computed as follows:

$$(N_1)_{60} = C_N N_{60} \tag{3}$$

where:

C_N = stress normalization parameter.

C_N is calculated as:

$$C_N = (P_a/\sigma'_{vo})^n \tag{4}$$

where:

P_a = atmospheric pressure (psf);
σ_{vo}' = effective vertical stress (psf); and
n = exponent based on soil type.

The exponent, n, is typically 1.0 for clays and ranges from 0.5 to 0.6 for sands. Soil specific correlations for the exponent have been developed for various geomaterials, but are not locally available. A value of 0.5 was selected for sands encountered at WGS. N-values can be either corrected to N_{60} or $(N_1)_{60}$ depending on the correlation or analysis being performed.

Cone Penetration Test Interpretation

CPT soundings performed onsite measured the cone tip resistance (q_c), the sleeve friction (f_s), and the pore pressure (u_2) in 0.05-m (≈ 2-in.) intervals. The cone tip resistance (q_c) must be corrected for the influence of pore pressure acting on the cone tip (Robertson and Cabal, 2012). The corrected cone tip resistance is computed as follows:

$$q_t = q_c + (1 - a_n)u_2 \quad (5)$$

where:

q_t = corrected cone tip resistance (tsf);
a_n = net area ratio; and
u_2 = measured pore pressure (tsf).

The cone used by MAD had a net area ratio of 0.80, which was applied by Geosyntec to each CPT sounding.

CPT sounding data are commonly interpreted into a Soil Behavior Type Index (I_c) (Robertson and Cabal, 2012), which is computed using the normalized cone tip resistance and normalized sleeve friction ratio. The normalized cone tip resistance (Q) is computed as follows:

$$Q = \left(\frac{q_t - \sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \quad (6)$$

where:

Q = normalized cone resistance;
q_t = corrected cone tip resistance (tsf);
σ_{vo} = total vertical stress (tsf);
σ'_{vo} = effective vertical stress (tsf);
P_a = atmospheric pressure (tsf); and
n = coefficient dependent on soil type and stress level.

A coefficient, n, of 1 was selected when interpreting each CPT sounding.

The normalized sleeve friction ratio (F) is calculated as follows:

$$F = \left(\frac{f_s}{q_t - \sigma_{vo}}\right) \times 100\% \quad (7)$$

where:

F = normalized sleeve friction ratio;
f_s = sleeve friction (tsf);
q_t = corrected tip resistance (tsf); and
σ_{vo} = total vertical stress (tsf).

Finally, the I_c is calculated as follows:

$$I_c = ((3.47 - \log Q)^2 + (\log F + 1.22)^2)^{0.5} \tag{8}$$

The normalized cone tip resistance and normalized friction ratio can be plotted on the Normalized Soil Behavior Type (SBT_N) Chart presented in Figure 2 to estimate the I_c. Figure 2 also presents the range of I_c corresponding to a given soil type. I_c was plotted with depth or elevation for each CPT sounding performed at WGS. An example of Geosyntec's interpretation is presented in Figure 3 for CPT-137.

LABORATORY TESTING PROGRAM AND INTERPRETATION

Geosyntec subcontracted Excel Geotechnical Testing, Inc. (EGT) of Roswell, Georgia to conduct geotechnical laboratory testing of select split spoon and thin-walled Shelby tube samples collected within the dike fill soils, foundation soils, and CCRs. The geotechnical laboratory testing program included index (grain size distribution, Atterberg limits, natural water content), unit weight, shear strength, and one dimensional (1-D) consolidation testing. Appendix 1 summarizes the index testing, unit weight, and shear strength testing results from Geosyntec's investigation. The raw laboratory test results are provided as Attachment 4 to the Safety Factor Assessment Report. Results from this laboratory testing program are discussed further below.

Index Testing

Dike Fill and Foundation Soils

The index testing program on dike fill and foundation soils included fourteen grain size distribution tests, four of which included hydrometer tests to evaluate the distribution of grain sizes of the soil which passes the No. 200 sieve (i.e., particle diameters less than 0.0029 in.). Grain size distribution analyses indicated that dike fill soils typically consisted of 72 percent to 87 percent sand-sized particles (diameters smaller than 0.187 in. but greater than 0.0029 in.) and 6 percent to 28 percent silt and clay-sized particles (i.e., "fines" with diameters smaller than 0.0029 in.) with most of the samples containing 10 percent to 15 percent fines. One sample of the dike fill soil at SPT-119 was classified as silt and contained 91 percent fines. This data point was considered an outlier and not representative of the Ash Pond A perimeter dikes.

Foundation soils were observed to be variable across the Ash Pond A perimeter dike footprint and were found to be composed predominantly of poorly graded to silty sands with pockets of clayey sand or high plasticity clay and clayey shell hash. The poorly graded and silty or clayey sands were observed to be composed typically of 60 percent to 90 percent sand-sized particles with typically 10 percent to 25 percent fines in the grain size distribution tests. Some samples were described to resemble shell hash and contained shells and fine gravel that constituted between 11 percent and 33 percent of the sample by weight. The grain size distribution analysis results are shown graphically in Figure 4.

Fines content tests were performed on fifteen samples to supplement grain size distribution tests. Fines content data, including results from grain size distribution tests, are provided in Figure 5. Results indicated that the dike fill soils typically contained less than 15 percent fines (except in the upper 5 to 7 ft bgs), while foundation materials typically had up to 20 percent fines, except where clay is encountered. For most samples, the foundation soils were relatively poorly graded sands (less than 10 percent fines).

Tests for moisture content and Atterberg Limits were performed on select soil samples. Geosyntec conducted nine Atterberg limits tests on clays or clayey sands within the foundation soils. Generally, Geosyntec did not perform Atterberg limits tests on soils that were observed in the field to be non-plastic. Clayey soils were computed to have plasticity indices (PI) ranging between 55 and 91, with three of the four samples ranging between 55 and 58. Atterberg limits for tests performed on sandy soils were calculated to be non-plastic. Moisture content of the sandy foundation soils were calculated to range between 15 percent and 30 percent. Clay foundation soils were calculated to have moisture contents of approximately 32 percent to 59 percent. A plot of the calculated moisture contents with elevation is provided in Figure 6.

One specific gravity test was performed on a Shelby tube sample of foundation soil collected for shear strength testing. The foundation soil was calculated to have a specific gravity of 2.72, which is within the typical range for soils.

Fly Ash

Index testing was performed on thin-walled Shelby tube samples collected from SPT-123 and from split spoon samples collected from SPT-304 and SPT-305 advanced within the interior of Ash Pond A. Grain size distribution tests on four samples of fly ash indicated that the samples were composed of 4 percent to 80 percent and 20 percent to 96 percent sand-sized and fine-sized (silts and clays) particles, respectively. The samples of fly ash were calculated to be non-plastic. One specific gravity test was performed on one of the Shelby tube samples, and a specific gravity of 2.31 was calculated. A pH test (ASTM D 4792) and carbonate content test (ASTM D 4373) were performed on one fly ash sample from Ash Pond A and the pH and carbonate contents were calculated to be 5.7 percent and 0.0 percent, respectively.

Williamsburg Formation Clay

Several grain size distribution tests were performed on the Williamsburg Formation Clay and these samples were generally calculated to consist of 19 percent to 58 percent sand-sized particles and 52 percent to 81 percent fine-sized particles. A layer of clayey sand was encountered below the Williamsburg Formation Clay stratum and calculated to consist of 43 percent to 71 percent sand-sized particles and 21 percent to 52 percent fine-sized particles. The Williamsburg Formation Clay was calculated to have plasticity indices ranging between 22 and 41 while the underlying clayey sand was calculated to be non-plastic or to have low plasticity indices (between 3 and 12). The calculated moisture content of the Williamsburg Formation clay ranged from 43 percent to 65 percent while the underlying clayey sand ranged from 27 percent to 37 percent.

Total Unit Weight

Dike Fill and Foundation Soils

The dry unit weight and initial moisture content were measured during triaxial shear strength and 1-D consolidation testing on thin-walled Shelby tube samples of the foundation soils. Since the dike fill soils were observed to consist of dense, silty to poorly graded sands, Shelby tube samples of these materials could not be collected. However, the unit weight for the dike fill soils was estimated using V_S measurements discussed later within this calculation package. Thin-walled Shelby tube samples collected from Ash Pond A soil borings SPT-116, SPT-117, and SPT-118 and Ash Pond B (SPT-308 – included herein for completeness) indicated that the sandy foundation soils have a total unit weight of 119 pcf (SPT-116), while clayey foundation soils were estimated to have a total unit weight ranging between 90 and 103 pcf. A plot of the total unit weight measurements is provided in Figure 7 (including results from one CU test from an Ash Pond B sample).

Fly Ash

The dry unit weight and initial moisture content were measured as part of the shear strength and consolidation tests for two fly ash samples collected within the interior of Ash Pond A. The total unit weight was calculated using the measured dry unit weight and initial moisture content. The results indicated that the total unit weight of the residual fly ash ranges from 100 to 111 pcf.

Williamsburg Formation Clay

Total unit weight of the Williamsburg Formation Clay was computed from the moisture content and dry unit weight measured during a hydraulic conductivity test. The total unit weight for the Williamsburg Formation Clay sample was computed as 111 pcf.

Undrained Shear Strength

Consolidated undrained (CU) triaxial strength tests were performed on extruded thin-walled Shelby tube samples of foundation soil and fly ash materials. CU tests were performed on four samples of foundation soils and two samples of ponded fly ash material. A description of the CU test and its interpretation is presented herein.

Methodology

For CU triaxial tests, a soil sample is usually trimmed into two or three specimens (depending on the Shelby tube recovery), and each specimen is tested under a different initial effective confining stress. The initial effective confining stress applied in each test should generally be applied at the effective overburden stress state or greater. The larger overburden stress states compensate for the effect of sample disturbance. The undrained shear strength (S_u) measured in each CU test corresponds to the initial effective confining stress applied to the specimens rather than the in-situ effective overburden stress to which the specimens were subjected. Therefore, the measured S_u from each CU test cannot directly be used in subsequent analyses. However, a relationship between the S_u in the field and the calculated S_u from the CU test results can be used to calculate the "in-situ" S_u.

The undrained shear strength ratio, defined as S_u/σ_c', can be calculated from CU test results, where S_u is the undrained shear strength measured in the laboratory and is equal to one half of the peak deviator stress (the peak deviator stress is assumed to indicate the failure point of the specimen in this calculation package), and σ_c' is the initial effective confining stress applied in the CU test. If the sample is overconsolidated, the calculated S_u/σ_c' is then corrected for the overconsolidation effect by multiplying by a factor of $OCR^{0.8}$ (Kulhawy and Mayne, 1990). The S_u/σ_c', or corrected S_u/σ_c' if the soil is overconsolidated, can be applied directly to a slope stability analysis program. The slope stability analysis program calculates the effective stress for each slice and then assigns the appropriate S_u value based on the undrained shear strength ratio.

Foundation Soils

Three sets of 1-point and 2-point CU tests (i.e., the testing of one or two specimens) were conducted on samples collected from the clay foundation soils. The undrained shear strength ratio (S_u/σ_c') was calculated based on the calculated peak deviator stress from each specimen confined at different stresses The triaxial test results indicate that undrained shear strength ratios range from 0.25 to 0.65 for foundation soils classified as sandy clay and fat clay. A plot of the S_u/σ_c' is shown in Figure 8. An OCR of 1.0 (i.e., normally consolidated soil) was selected for data interpretation and thus, the correction factor discussed above was not required.

Fly Ash

Two sets of 3-point CU tests were conducted on thin-walled Shelby tube samples of fly ash. The undrained shear strength ratio was calculated for each test specimen based on the calculated peak deviator stress for each point. The test results indicated that undrained shear strength ratios range from 0.98 to 6.93 for the residual fly ash. A plot of the S_u/σ_c' assuming an OCR of 1.0 is shown in Figure 8.

Drained Shear Strength

Effective stress friction angles (ϕ') and cohesion intercepts (c') were also computed for the different materials based on the CU triaxial test results. The stress states (i.e., shear stress vs. effective normal stress) of the tested samples are represented using Mohr's circles. The Mohr's circles for all tests of the same material are then fit with a line that is approximately tangent to all of the Mohr's circles, which represents the failure envelope. The effective normal stress and shear stress (i.e., x- and y-coordinates) at the point of tangency are defined as the normal stress at failure (σ_{NF}') and shear stress at failure (τ_{NF}'), respectively. The slope of the "best-fit" line corresponds to ϕ' and the shear stress intercept (i.e., y-intercept) corresponds to c'. The ϕ′ and c′ estimated by the "best-fit" line are then used to represent the drained shear strengths of the soils and fly ash.

Foundation Soils

The Mohr's circles from the CU triaxial tests on the clay foundation soils are plotted in Figure 9. The σ_{NF}' and τ_{NF}' for the Mohr circles shown in Figure 9 are plotted in Figure 10. Based on the CU test results, the ϕ′ and c′ for the fat clay were estimated to be 18° and 250 psf, respectively.

Fly Ash

The Mohr's circles from the CU tests on the residual fly ash are plotted in Figure 11. The σ_{NF}' and τ_{NF}' obtained from these Mohr's circles are shown in Figure 12. Based on the CU test results, the ϕ′ and c′ for the residual fly ash were estimated to be 34° and 0 psf, respectively.

Consolidation Test Interpretation

Foundation Soils

One-dimensional (1-D) consolidation tests were conducted on three thin-walled Shelby tube samples of clays within the foundation soils (from SPT-117, SPT-118, and SPT-308). The preconsolidation pressures (σ_p') estimated from these tests were between 1,700 and 3,100 pounds per square foot (psf). The strain is plotted against the applied vertical load in Figure 13. The modified compression index ($C_{c\varepsilon}$) and modified recompression index ($C_{r\varepsilon}$) were calculated for each 1-D consolidation test. The range of $C_{c\varepsilon}$ and $C_{r\varepsilon}$ were computed to be between 0.29 and 0.34 and 0.05 and 0.07, respectively. The coefficient of consolidation (C_v) and modified coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) were calculated for each load increment and plotted as a function of a stress ratio (σ_v'/σ_p'). Figures 14 and 15 present the C_v and $C_{\alpha\varepsilon}$ results for the clay foundation soils. It is noted that the 1-D test from boring SPT-308 appeared to be heavily disturbed in the laboratory and was only used to approximate $C_{c\varepsilon}$ and $C_{r\varepsilon}$ during this evaluation.

Fly Ash

A 1-D consolidation test was conducted on a thin-walled Shelby tube sample collected from the interior of Ash Pond A. The σ_p' estimated during this test was 11,000 psf. The strain is plotted against the applied vertical load in Figure 13. $C_{c\varepsilon}$ and $C_{r\varepsilon}$ were calculated as 0.12 and 0.004, respectively. Additionally, C_v and $C_{\alpha\varepsilon}$ were calculated from each load increment and plotted as a function of σ_v'/σ_p'. Figures 16 and 17 present the C_v and $C_{\alpha\varepsilon}$ results for the fly ash.

Hydraulic Conductivity

One hydraulic conductivity test was performed on a Shelby Tube sample collected in the Williamsburg Formation Clay. The test results computed a hydraulic conductivity (k) of 1.4×10^{-8} cm/s for the

Williamsburg Formation Clay.

IN-SITU TESTING INTERPRETATION

Correlations were applied to in-situ testing data (q_t, f_s, etc.) to compute index and strength properties of the materials. The computed values from the correlations were then compared to the laboratory test results. Additionally, correlations were used for the in-situ measurements of the V_s and porewater pressure dissipation performed at several locations along the perimeter dike centerline and dike toe. The following section describes the methodology and correlations applied to interpret index and strength properties from the in-situ testing performed at the Site.

Shear Wave Velocity

Shear wave velocity measurements were taken in 5-ft depth intervals at several locations along the dike centerline and dike toe using a seismic CPT. Raw V_s data is provided in Attachment 3 of the Safety Factor Assessment Report. The field V_s testing data was supplemented with correlated values developed from CPT sounding sleeve friction data from adjacent soundings. Robertson and Cabal (2012) provides a correlation between V_s and CPT sounding data for saturated sands, clays, and silts, as follows:

$$V_s = [\alpha_{vs} \times (\frac{q_t \times \sigma_{vo}}{P_a})]^{0.5} \tag{9}$$

where:

V_s = shear wave velocity (m/s);
q_t = corrected tip resistance (tsf);
σ_{vo} = total vertical stress (tsf);
P_a = atmospheric pressure (tsf); and
α_{vs} = $10^{(0.55 \times Ic + 1.68)}$.

Drained Friction Angle

SPT N-values were utilized to estimate the drained peak effective stress friction angle of sandy soils. The Hatanaka and Uchida (1996) correlation was applied to estimate the peak friction angle of sand layers that are relatively clean as follows:

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20° \tag{10}$$

ϕ' = effective stress friction angle (degrees); and
$(N_1)_{60}$ = stress normalized and energy corrected N-value (blows/ft).

Undrained Shear Strength Ratio

Undrained shear strength ratio, as computed by the following correlation, was compared with laboratory test data. The undrained shear strength ratio was estimated from CPT soundings based on the correlation presented by Robertson and Cabal (2012) as follows:

$$\frac{S_u}{\sigma'_v} = \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}}\left(\frac{1}{N_{kt}}\right) \tag{11}$$

where:

S_u/σ_v' = undrained shear strength (tsf);
q_t = corrected tip resistance (tsf);
σ_{vo} = total vertical stress (tsf);
σ'_{vo} = effective vertical stress (tsf); and
N_{kt} = coefficient based on shear mode.

N_{kt} varies regionally and by material type, with a typical range between 10 and 20; a value of 15 was selected in this calculation package (FHWA, 2002).

Effective Friction Angle by CPT Sounding Correlation

The effective friction angle was computed for each CPT sounding by the following correlation suggested by Robertson and Campanella (1983) for un-cemented, un-aged, moderately compressible quartz sands based on calibration chamber testing, as follows:

$$\tan\phi' = \frac{1}{2.68}\left[\log\frac{q_c}{\sigma'_{vo}} + 0.29\right] \tag{12}$$

where:

ϕ' = effective friction angle (degrees);
q_c = tip resistance (tsf); and
σ'_{vo} = effective vertical stress (tsf).

Total Unit Weight

The total unit weight (γ_t) of saturated subsurface materials can be approximated based on V_s measurements (Mayne, 2001) as shown below:

$$\gamma_t = 8.32\log(V_s) - 1.81\log(z) \tag{13}$$

where:

γ_t = total unit weight (kN/m^3)
V_s = shear wave velocity (m/s); and

z = depth (m).

The total unit weight of subsurface layers computed by Equation 13 is included in Figure 7.

RECOMMENDED MATERIAL PROPERTIES

The following paragraphs describe the recommended values of the material properties for analysis of the perimeter dikes surrounding Ash Pond A. The index and shear strength properties calculated from the laboratory tests and in-situ testing correlations were evaluated to establish the recommended values. Table 6 summarizes the recommended values for analysis.

Total Unit Weight

Figure 7 presents total unit weight (γ_t) values measured as part of CU testing and consolidation testing as well as γ_t interpreted from V_s measurements (Equation 13). A total unit weight of 100 pcf is selected for the clayey foundation soils, generally encountered below 5 ft NGVD29. A total unit weight between 110 pcf and 115 pcf is recommended for the sandy foundation materials. A total unit weight of 125 pcf is conservatively selected for the dike fill. Total unit weight values for the fly ash measured as part of CU testing and consolidation testing indicate that the total unit weight of fly ash ranges from 100 to 111 pcf. A unit weight of 100 pcf is recommended for fly ash within Ash Pond A.

Undrained Shear Strength

Based on undrained shear strength ratios estimated from CU testing, a typical S_u/σ_c' value of 0.3 is recommended for the clay foundation soils, as shown in Figure 8. A representative S_u/σ_c' value of 1.0 is selected for the residual fly ash. However, CPT data indicates that the undrained shear strength ratio ranges vary throughout Ash Pond A as shown in Figure 18. For the safety factor assessment (Attachment 8), CPT and laboratory data closest to each evaluated cross section were used to select the undrained shear strength.

Drained Shear Strength

In general, estimated drained strength parameters exhibited significant variability across the Ash Pond A dike fill and foundation soils, as shown in Figures 19 and 20. The effective friction angle was calculated to range between 38 degrees and 55 degrees within the dike fill soils and 28 degrees to 36 degrees within the sandy foundation soils. Drained shear strength parameters were selected on a cross section-by-cross section basis for the safety factor assessment (Attachment 8). Isolated pockets or lenses of clay were observed within the foundation soils. An effective friction angle of 18 degrees with an effective cohesion of 250 psf is selected for these clay zones (Figures 9 and 10).

Consolidation Parameters

The clay foundation soils and fly ash are assumed to be normally consolidated (OCR = 1.0) since the material has recently been deposited within the impoundment. An average $C_{c\varepsilon}$ of 0.31 and 0.12 are selected for the clay foundation soils and fly ash, respectively. An average $C_{r\varepsilon}$ of 0.06 and 0.004 are selected for the clay foundation soils and fly ash, respectively. For the clay foundation soils, representative values for C_V of 0.09 and 0.009 ft^2/day are selected for stress ratios (σ_V'/σ_P') less than 1.0 and stress ratios greater than 1.0, respectively. A representative C_V value of 7.4 ft^2/day is selected for the fly ash. For the clay foundation soils, representative $C_{\alpha\varepsilon}$ values of 0.2 percent and 2.0 percent are selected for stress ratios (σ_V'/σ_P') less than 1.0 and stress ratios greater than 1.0, respectively. For the fly ash, representative $C_{\alpha\varepsilon}$ values of 0.05 percent and 0.2 percent are selected for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Selected C_V and $C_{\alpha\varepsilon}$ values for the clay foundation soils are presented in Figures 14 and 15. Selected C_V and $C_{\alpha\varepsilon}$ values for the fly ash are shown in Figures 16 and 17.

Representative Subsurface Profiles for Site Response Analysis

Shear wave velocity profiles, soil plasticity, and total unit weight are required as input for site response analyses presented in Attachment 6 of the Safety Factor Assessment Report. Therefore, two representative subsurface profiles were developed for sections of the perimeter dike structures based on the height of the perimeter dikes and the properties of the underlying soils. Representative profile 1 represents the 15-ft tall perimeter dike structures adjacent to the Intake and Discharge Canals. Representative profile 2 represents the 20 to 24-ft tall perimeter dikes adjacent to the Cooling Pond. Shear wave velocity profiles were developed from seismic CPT tests performed in 5-ft depth intervals during several CPT soundings and the correlated V_S (by Equation 9) from CPT sounding sleeve friction (f_S). The raw V_S data and interpretation of these tests are provided in Attachment 3 of the Safety Factor Assessment Report. The developed V_S profiles (by elevation) are summarized within Table 7 and provided in Figures 21 and 22 for representative profiles 1 and 2, respectively. Selection of the V_S of the Chicora and Williamsburg Formation Clay strata is discussed in Attachment 6 of the Safety Factor Assessment Report.

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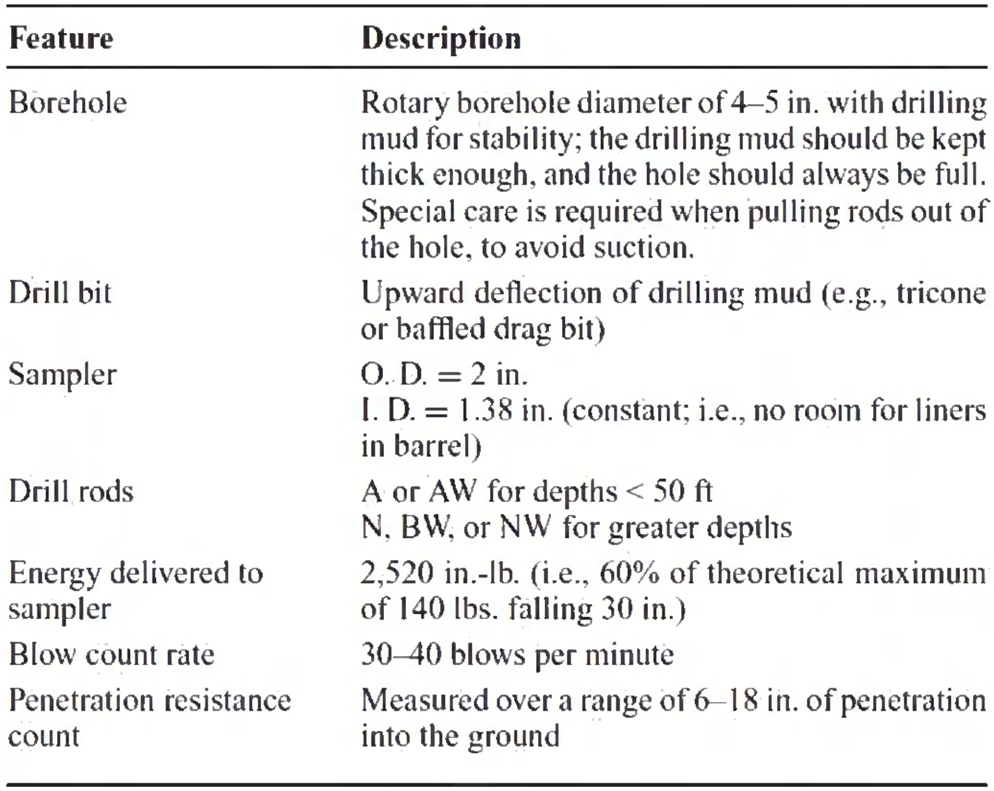
Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

TABLES

Table 1. Recommended SPT Procedure for Liquefaction Evaluations (Idriss and Boulanger, 2008)



Feature	Description
Borehole	Rotary borehole diameter of 4–5 in. with drilling mud for stability; the drilling mud should be kept thick enough, and the hole should always be full. Special care is required when pulling rods out of the hole, to avoid suction.
Drill bit	Upward deflection of drilling mud (e.g., tricone or baffled drag bit)
Sampler	O. D. = 2 in. I. D. = 1.38 in. (constant; i.e., no room for liners in barrel)
Drill rods	A or AW for depths < 50 ft N, BW, or NW for greater depths
Energy delivered to sampler	2,520 in.-lb. (i.e., 60% of theoretical maximum of 140 lbs. falling 30 in.)
Blow count rate	30–40 blows per minute
Penetration resistance count	Measured over a range of 6–18 in. of penetration into the ground

Table 2. Summary of Water Level Measurements

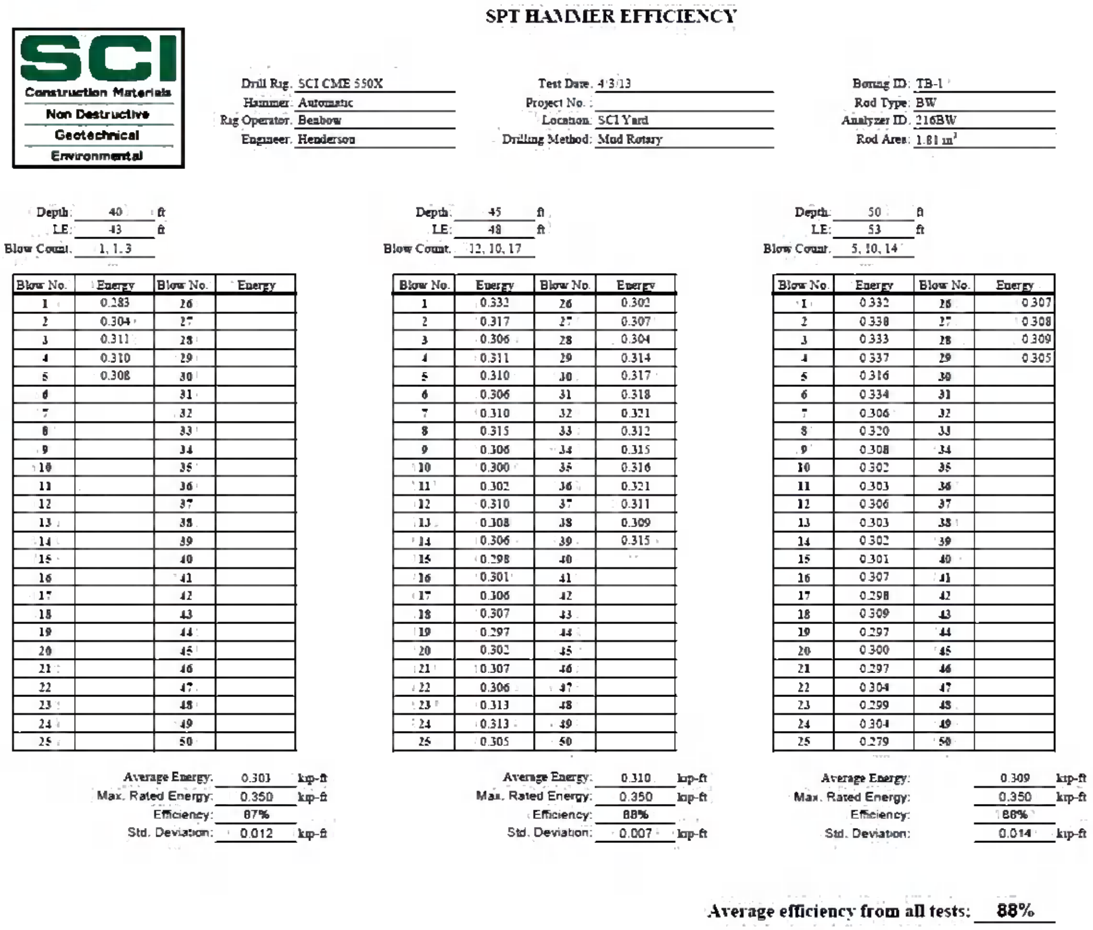
Boring ID	Method	Location	Ground Surface El.	Depth to Water (24-hr)	Depth of Dissipation Test	Measured Hydrostatic Pressure	Phreatic Surface Elevation
-	-	-	ft NGVD29	ft bgs	ft bgs	ft	ft NGVD29
CPT-137	u_2 Signature	Dike Center	41.5	18.0	-	-	23.5
CPT-138	u_2 Signature	Dike Center	41.2	16.8	-	-	24.4
CPT-138	Diss. Test	Dike Center	41.2	16.8	35.0	18.2	24.4
CPT-139	u_2 Signature	Dike Toe	26.8	5.0	-	-	21.8
CPT-140	u_2 Signature	Dike Center	40.9	14.0	-	-	26.9
CPT-141	u_2 Signature	Dike Toe	27.9	5.0	-	-	22.9
CPT-142	u_2 Signature	Dike Center	41.4	15.0	-	-	26.4
CPT-143	u_2 Signature	Dike Center	41.2	18.0	-	-	23.2
CPT-144	u_2 Signature	Dike Center	40.6	18.0	-	-	22.6
CPT-145	u_2 Signature	Dike Toe	24.7	3.5	-	-	21.2
CPT-146	u_2 Signature	Dike Center	40.2	18.1	-	-	22.1
CPT-146	Diss. Test	Dike Center	40.2	18.1	30.1	12.0	22.1
CPT-147	u_2 Signature	Dike Center	39.9	18.0	-	-	21.9
CPT-148	u_2 Signature	Dike Toe	27.5	5.5	-	-	22.0
CPT-155	Diss. Test	Inside Pond	47.5	13.0	14.9	2.0	34.5
CPT-155	Diss. Test	Inside Pond	47.5	13.1	24.9	11.9	34.4
CPT-156	u_2 Signature	Inside Pond	43.3	8.5	-	-	34.8
CPT-157	Diss. Test	Inside Pond	45.6	12.4	15.9	3.5	33.2
CPT-157	Diss. Test	Inside Pond	45.6	13.6	24.9	11.4	32.0
CPT-228	u_2 Signature	Dike Center	24.3	1.5	-	-	22.8
CPT-229	u_2 Signature	Dike Toe	21.1	0.5	-	-	20.6
CPT-229A	u_2 Signature	Dike Center	38.7	18.0	-	-	20.7
SPT-116	Borehole	Dike Center	41.4	15.0	-	-	26.4
SPT-117	Borehole	Dike Center	39.7	18.2	-	-	21.5
SPT-118	Borehole	Dike Center	39.7	11.6	-	-	28.1
SPT-119	Borehole	Dike Center	42.7	18.4	-	-	24.3
SPT-120	Borehole	Dike Center	41.1	12.2	-	-	28.9
SPT-121	Borehole	Dike Center	40.8	11.5	-	-	29.3
PPZ-AS-1	Piezometer	Inside Pond	45.1	12.5	-	-	32.6
WAP-8	MW	Dike Toe	30.4	-	-	-	21.3
PPZW-8D	Piezometer	Dike Toe	28.1	-	-	-	21.7
WAP-9	MW	Dike Toe	26.2	-	-	-	19.3
PPZW-9D	Piezometer	Dike Toe	24.5	-	-	-	23.9
WAP-12	MW	Dike Toe	-	-	-	-	23.8
WAP-17	MW	Dike Toe	-	-	-	-	23.8
WAP-18	MW	Dike Crest	-	-	-	-	26.7
WAP-19	MW	Dike Crest	-	-	-	-	25.9

Notes:

1. Depth to water levels in mud rotary boreholes may not be representative of existing conditions due to borehole collapse or the influence of drilling mud on measured depth to water levels.

2. In dissipation tests at CPT-138 and CPT-143, equilibrium conditions were not reached due to the presence of clay. Those data are excluded from the table above.
3. Piezometer and WAP-8 water levels were measured on 22 May 2015.
4. Monitoring well water levels (excluding WAP-8) were measured on 21 June 2016 and provided by Santee Cooper. Surface elevations for WAP-12 and WAP-17 through WAP-19 were not furnished to Geosyntec.

Table 3. CME-550X Energy Ratio Calibration (provided by Soil Consultants, Inc.) from Fall 2013 Investigation



SPT HAMMER EFFICIENCY

SCI
Construction Materials
Non Destructive
Geotechnical
Environmental

Drill Rig: SCI CME 550X
Hammer: Automatic
Rig Operator: Benbow
Engineer: Henderson

Test Date: 4/3/13
Project No.:
Location: SCI Yard
Drilling Method: Mud Rotary

Boring ID: TB-1
Rod Type: BW
Analyzer ID: 216BW
Rod Area: 1.81 in²

Depth: 40 ft
LE: 43 ft
Blow Count: 1, 1. 3

Blow No.	Energy	Blow No.	Energy
1	0.283	26	
2	0.304	27	
3	0.311	28	
4	0.310	29	
5	0.308	30	
6		31	
7		32	
8		33	
9		34	
10		35	
11		36	
12		37	
13		38	
14		39	
15		40	
16		41	
17		42	
18		43	
19		44	
20		45	
21		46	
22		47	
23		48	
24		49	
25		50	

Average Energy: 0.303 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 87%
Std. Deviation: 0.012 kip-ft

Depth: 45 ft
LE: 48 ft
Blow Count: 12, 10, 17

Blow No.	Energy	Blow No.	Energy
1	0.332	26	0.302
2	0.317	27	0.307
3	0.306	28	0.304
4	0.311	29	0.314
5	0.310	30	0.317
6	0.306	31	0.318
7	0.310	32	0.321
8	0.315	33	0.312
9	0.306	34	0.315
10	0.300	35	0.316
11	0.302	36	0.321
12	0.310	37	0.311
13	0.308	38	0.309
14	0.306	39	0.315
15	0.298	40	
16	0.301	41	
17	0.306	42	
18	0.307	43	
19	0.297	44	
20	0.302	45	
21	0.307	46	
22	0.306	47	
23	0.313	48	
24	0.313	49	
25	0.305	50	

Average Energy: 0.310 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 88%
Std. Deviation: 0.007 kip-ft

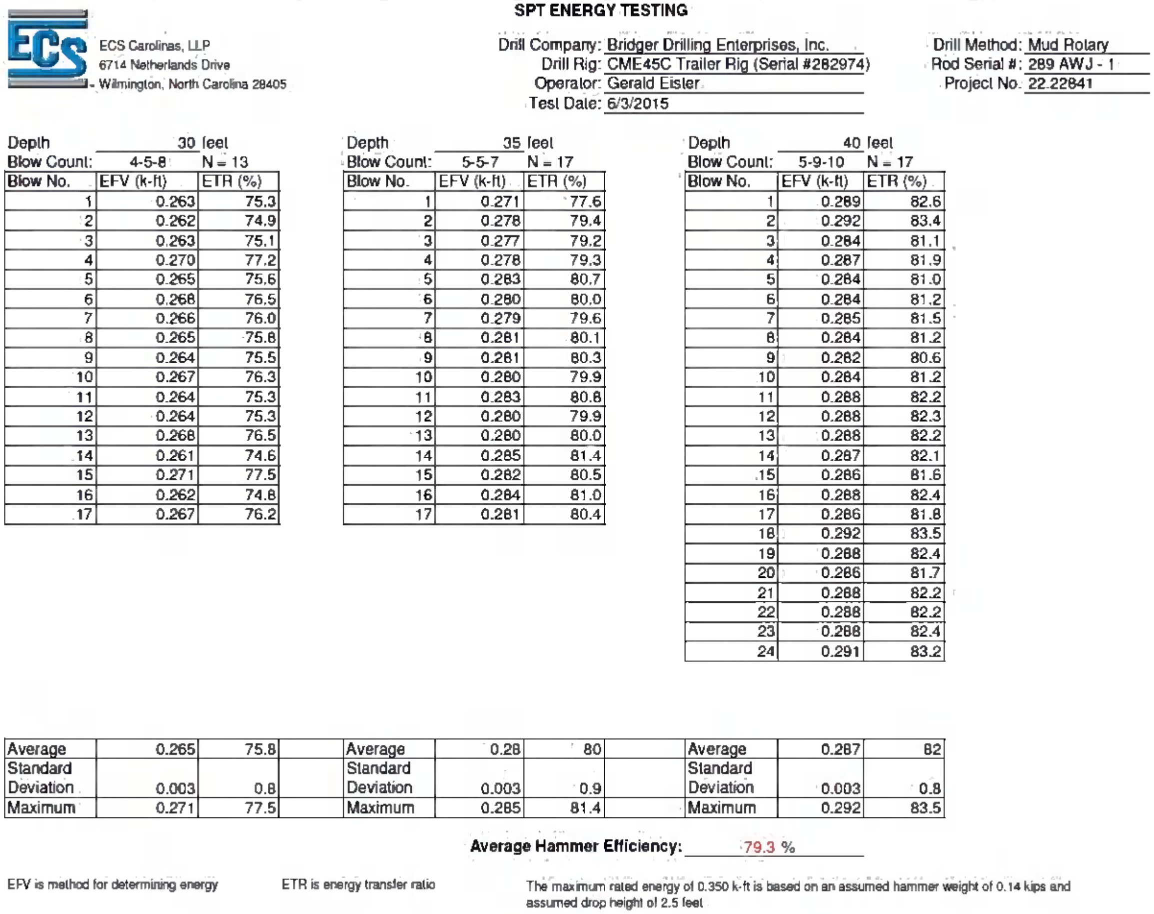
Depth: 50 ft
LE: 53 ft
Blow Count: 5, 10, 14

Blow No.	Energy	Blow No.	Energy
1	0.332	26	0.307
2	0.338	27	0.308
3	0.333	28	0.309
4	0.337	29	0.305
5	0.316	30	
6	0.334	31	
7	0.306	32	
8	0.320	33	
9	0.308	34	
10	0.302	35	
11	0.303	36	
12	0.306	37	
13	0.303	38	
14	0.302	39	
15	0.301	40	
16	0.307	41	
17	0.298	42	
18	0.309	43	
19	0.297	44	
20	0.300	45	
21	0.297	46	
22	0.304	47	
23	0.299	48	
24	0.304	49	
25	0.279	50	

Average Energy: 0.309 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 88%
Std. Deviation: 0.014 kip-ft

Average efficiency from all tests: 88%

Table 4. CME-45C Energy Ratio Calibration for 2016 Investigation (Provided by Bridger Drilling Enterprises, Inc. (Carolina Drilling))



SPT ENERGY TESTING

ECS Carolinas, LLP
6714 Netherlands Drive
Wilmington, North Carolina 28405

Drill Company: Bridger Drilling Enterprises, Inc.
Drill Rig: CME45C Trailer Rig (Serial #282974)
Operator: Gerald Eisler
Test Date: 6/3/2015

Drill Method: Mud Rotary
Rod Serial #: 289 AWJ - 1
Project No. 22.22841

Depth 30 feet
Blow Count: 4-5-8 N = 13

Blow No.	EFV (k-ft)	ETR (%)
1	0.263	75.3
2	0.262	74.9
3	0.263	75.1
4	0.270	77.2
5	0.265	75.6
6	0.268	76.5
7	0.266	76.0
8	0.265	75.8
9	0.264	75.5
10	0.267	76.3
11	0.264	75.3
12	0.264	75.3
13	0.268	76.5
14	0.261	74.6
15	0.271	77.5
16	0.262	74.8
17	0.267	76.2
Average	0.265	75.8
Standard Deviation	0.003	0.8
Maximum	0.271	77.5

Depth 35 feet
Blow Count: 5-5-7 N = 17

Blow No.	EFV (k-ft)	ETR (%)
1	0.271	77.6
2	0.278	79.4
3	0.277	79.2
4	0.278	79.3
5	0.283	80.7
6	0.280	80.0
7	0.279	79.6
8	0.281	80.1
9	0.281	80.3
10	0.280	79.9
11	0.283	80.8
12	0.280	79.9
13	0.280	80.0
14	0.285	81.4
15	0.282	80.5
16	0.284	81.0
17	0.281	80.4
Average	0.28	80
Standard Deviation	0.003	0.9
Maximum	0.285	81.4

Depth 40 feet
Blow Count: 5-9-10 N = 17

Blow No.	EFV (k-ft)	ETR (%)
1	0.289	82.6
2	0.292	83.4
3	0.284	81.1
4	0.287	81.9
5	0.284	81.0
6	0.284	81.2
7	0.285	81.5
8	0.284	81.2
9	0.282	80.6
10	0.284	81.2
11	0.288	82.2
12	0.288	82.3
13	0.288	82.2
14	0.287	82.1
15	0.286	81.6
16	0.288	82.4
17	0.286	81.8
18	0.292	83.5
19	0.288	82.4
20	0.286	81.7
21	0.288	82.2
22	0.288	82.2
23	0.288	82.4
24	0.291	83.2
Average	0.287	82
Standard Deviation	0.003	0.8
Maximum	0.292	83.5

Average Hammer Efficiency: 79.3 %

EFV is method for determining energy

ETR is energy transfer ratio

The maximum rated energy of 0.350 k-ft is based on an assumed hammer weight of 0.14 kips and assumed drop height of 2.5 feet

Table 5. Standard N-value Correction Factors for Drilling Methods

Factor	Description
Energy ratio	Energy measurements are required to determine the delivered energy ratios or to calibrate the specific equipment being used. The correction factor is then computed as $C_E = \frac{ER_m}{60}$ where ER_m is the measured energy ratio as a percentage of the theoretical maximum. Empirical estimates of C_E (for rod lengths of 10 m or more) involve considerable uncertainty, as reflected by the following ranges: Doughnut hammer $C_E = 0.5\text{–}1.0$ Safety hammer $C_E = 0.7\text{–}1.2$ Automatic triphammer $C_E = 0.8\text{–}1.3$ (Seed et al. 1984, Skempton 1986, NCEER 1997)
Borehole diameter	Borehole diameter of 65–115 mm $C_B = 1.0$ Borehole diameter of 150 mm $C_B = 1.05$ Borehole diameter of 200 mm $C_B = 1.15$ (Skempton 1986)
Rod length	Where the ER_m is based on rod lengths of 10 m or more, the ER delivered with shorter rod lengths may be smaller. Recommended values from Youd et al. (2001) are as follows: Rod length < 3 m $C_R = 0.75$ Rod length 3–4 m $C_R = 0.80$ Rod length 4–6 m $C_R = 0.85$ Rod length 6–10 m $C_R = 0.95$ Rod length 10–30 m $C_R = 1.00$
Sampler	Standard split spoon without room for liners (the inside diameter is a constant $1^3/_8$ in.), $C_S = 1.0$. Split-spoon sampler with room for liners but with the liners absent (this increases the inside diameter to $1^1/_2$ in. behind the driving shoe): $C_S = 1.1 \quad \text{for} \quad (N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100} \quad \text{for} \quad 10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3 \quad \text{for} \quad (N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Table 6. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters		Undrained Parameters		Consolidation Parameters [1]				
		ϕ' (°)	c' (psf)	S_u/σ_v'	$S_{u,\min}$ (psf)	$C_{c\varepsilon}$	$C_{r\varepsilon}$	$C_{\alpha\varepsilon}$ (%)	C_v (ft^2/day)	OCR
Dike Fill Soils	125	Varies[2]	0	-	-	-	-	-	-	-
Foundation Soils (Clayey)	100	18	250	Varies[2]	100	0.31	0.06	2.0	0.009	1.0
Foundation Soils (Sandy)	115	Varies[2]	0	-	-	-	-	-	-	-
Chicora	130	50	0	-	-	-	-	-	-	-
Williamsburg Formation Clay [3]	105	50	0	-	-	-	-	-	-	-
Fly Ash	100	34	0	1.0	-	0.12	0.004	0.2	7.4	1.0

Notes:

1. C_v and $C_{\alpha\varepsilon}$ values are provided assuming soils are normally consolidated in-situ and additional loading would yield a stress ratio greater than 1.0 (i.e., $\sigma_v' / \sigma_p' > 1.0$).
2. Strength parameters for dike fill and foundation soils were selected on a cross section by cross section basis during the safety factor assessment (Attachment 8).
3. Strength parameters for the Williamsburg Formation Clay are based on direct shear testing performed from cored samples provided by S&ME (2001). The Williamsburg Formation Clay is typically 50 ft bgs, and critical slip surfaces during slope stability analyses are not anticipated to pass through this zone based on the perimeter dike heights. Measured blow counts (N-values) within this material ranged from 30 to 100 blows per foot and were typically in excess of 50 blows per foot.

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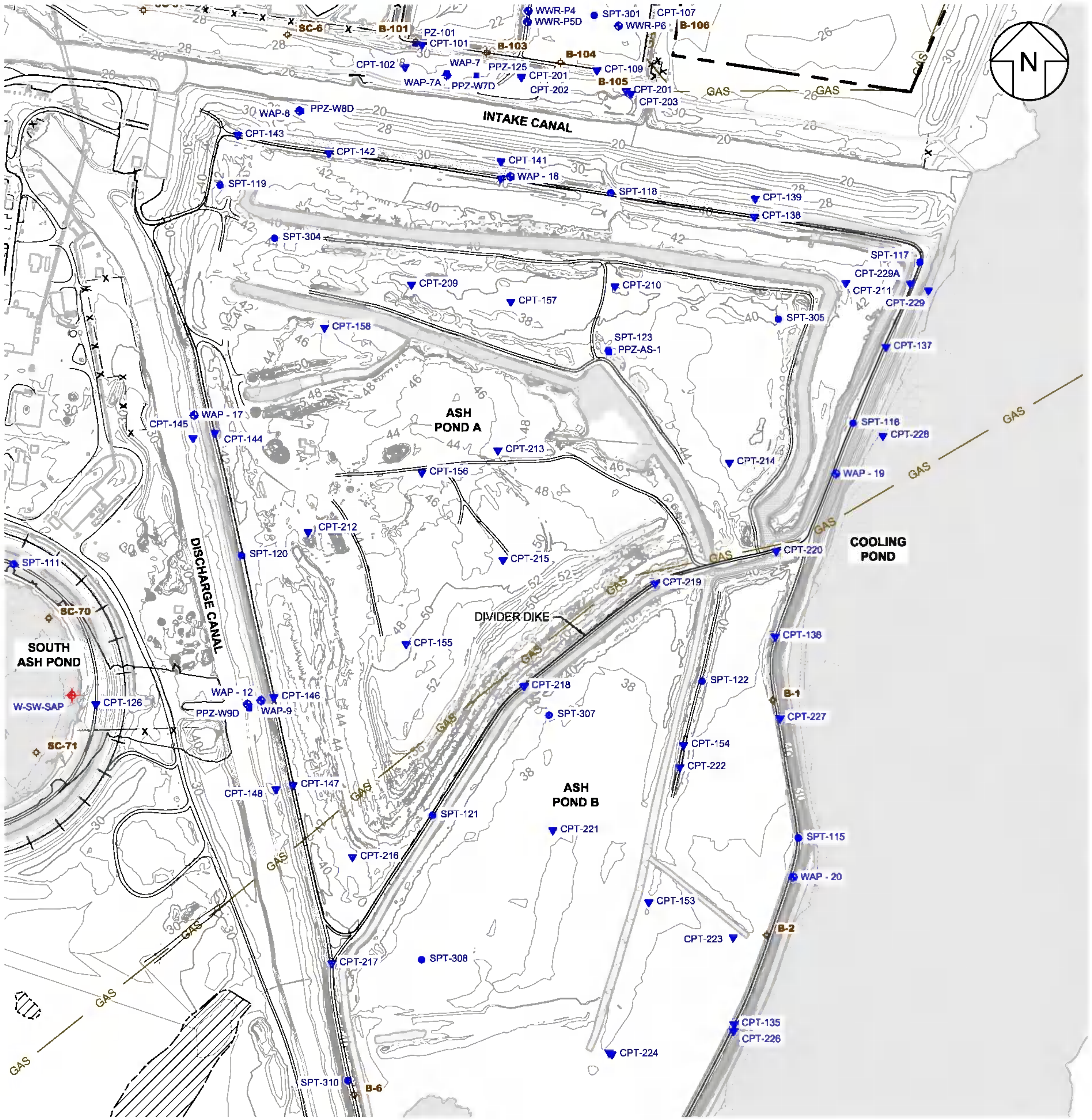
Table 7. Summary of Representative Shear Wave Velocity Profiles

Profile 1 (Dike Centerline)		Profile 2 (Dike Centerline)	
Elev. (ft)	V_s (ft/s)	Elev. (ft)	V_s (ft/s)
-60 to -10	1500+	-60 to -10	1500+
-10 to 0	600	-10 to 10	700
0 to 25	700	10 to 20	800
25 to 35	900	20 to 37	900
35 to 40	550	37 to 42	550

Notes:

1. Elevations are provided in ft NGVD29.
2. Shear wave velocities at elevations below -10 ft NGVD29 are discussed in Attachment 6.

FIGURES



LEGEND

— GAS — GAS —	EXISTING GAS LINE
— 20 —	EXISTING MAJOR GRADE CONTOUR
+—+—+—+	EXISTING RAILROAD
▨	EXISTING PONDED WATER
⌖ W-SW-SAP	EXISTING STAFF GAUGE
▼ CPT-101	GEOSYNTEC CONE PENETRATION TEST
● SPT-111	GEOSYNTEC SOIL BORING
⌖ B-1	HISTORICAL BORING
◆ WAP-7, WWR-P4	MONITORING WELL
■ PPZ-125, PPZ-AS-1, PZ-101	PIEZOMETER

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
3. THE POSITION OF UNDERGROUND UTILITIES SHOWN ON THIS DRAWING IS BASED UPON THE LOCATION OF SURFACE APPURTENANCES AND/OR SURFACE MARKINGS AND SHOULD BE CONSIDERED APPROXIMATE.

0 400' 800'

SCALE IN FEET

ASH POND A
BORING LOCATION MAP

Geosyntec consultants

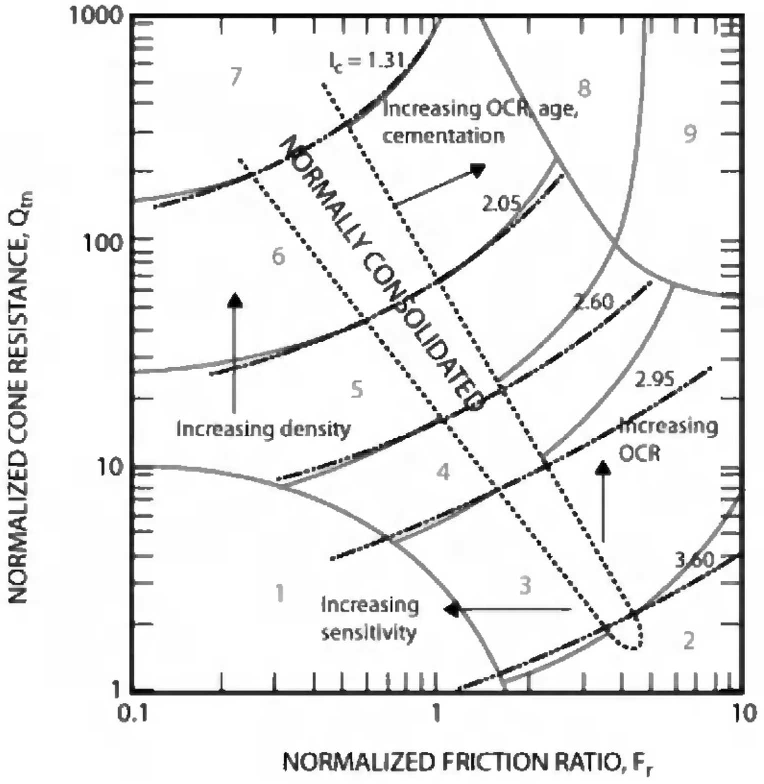
FIGURE

1

PROJECT NO: GSC5242

OCTOBER 2016

M:\S\SANTEE COOPER\SANTEE COOPER-WINYAH\0028-WINYAH H&H ANALYSES\FIGURES\F-0-SC-585-00-F0028-040



Zone	*Soil Behavior Type*	*I_c*
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

* *Heavily overconsolidated or cemented*

Figure 2. SBT_N Chart with typical I_c Ranges used in CPT Interpretation (Robertson and Cabal, 2012)

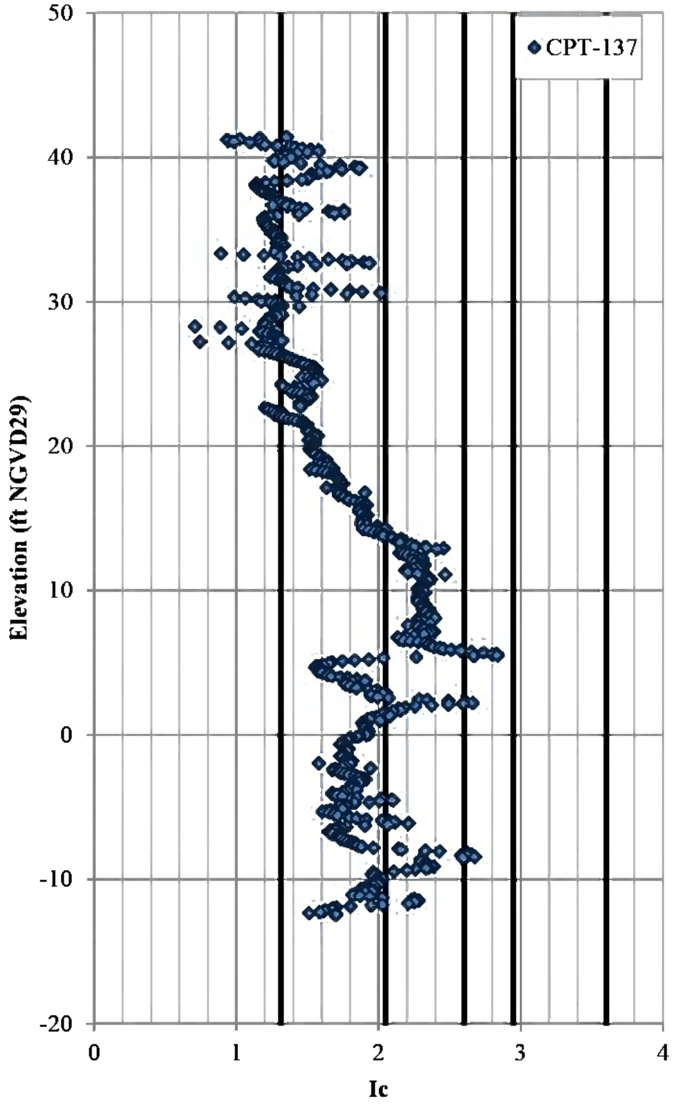


Figure 3. Example I_c classification Profile for CPT-137

Notes:

1. I_c – Soil Behavior Index by Robertson and Cabal (2012).
2. Thick black lines represent transitions between soil types shown in Figure 2.

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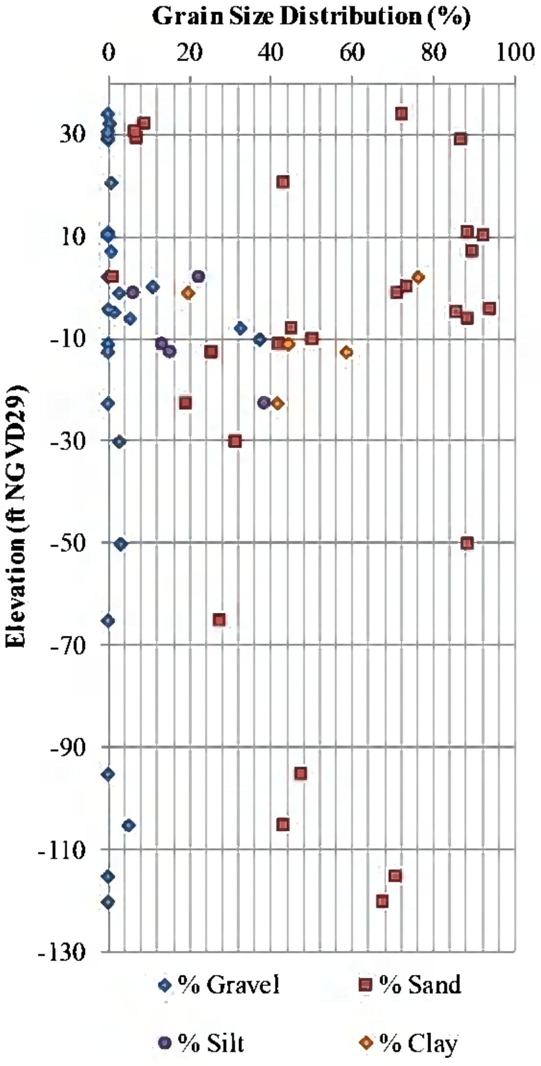

Figure 4. Grain Size Distribution Test Results

Note:

1. Samples of Williamsburg Formation Clay were observed below Elevation -20.0 ft NGVD29.

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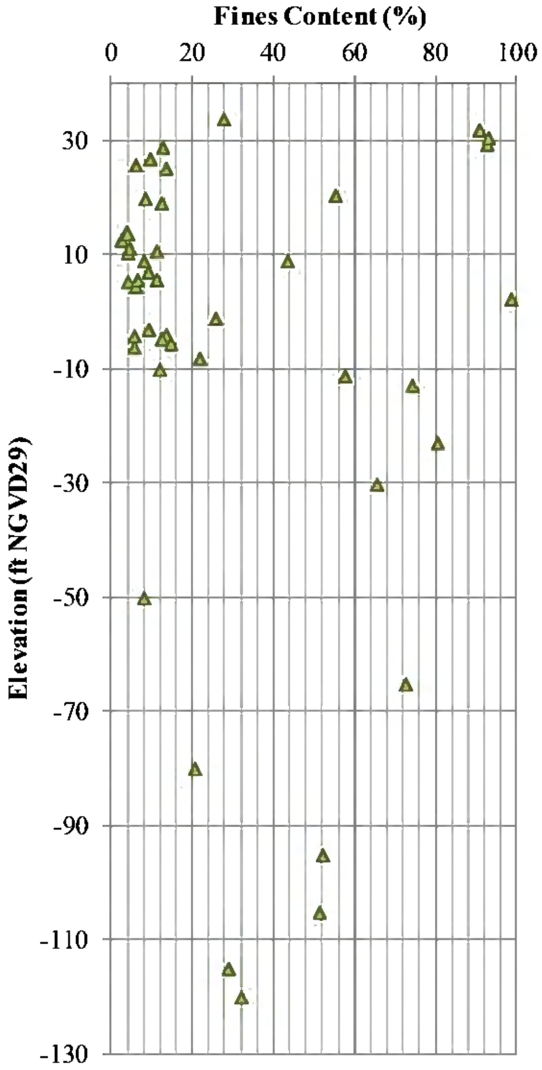

Figure 5. Fines Content Test Results

Note:

1. Samples of Williamsburg Formation Clay were observed below Elevation -20.0 ft NGVD29.

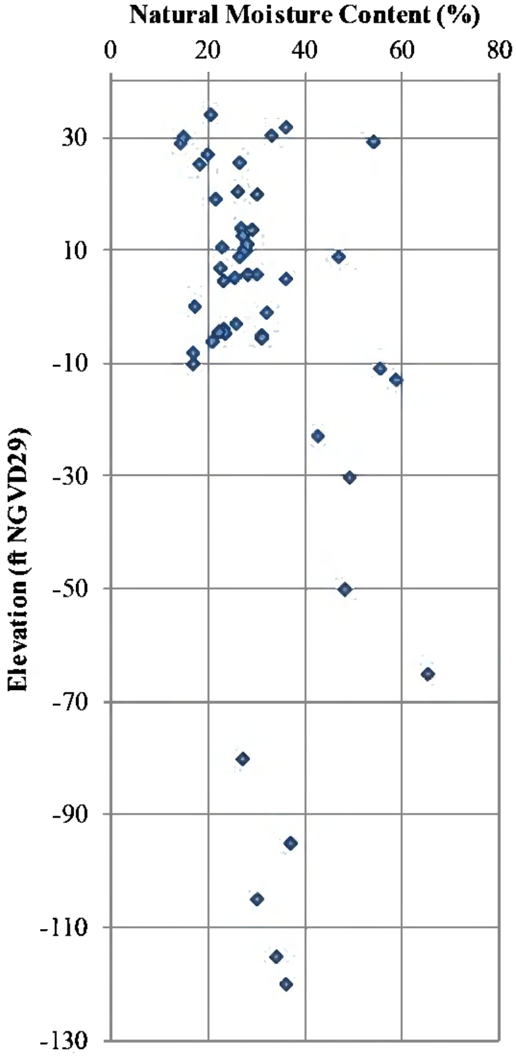

Figure 6. Moisture Content Test Results

Note:

1. Samples of Williamsburg Formation Clay were observed below Elevation -20.0 ft NGVD29.

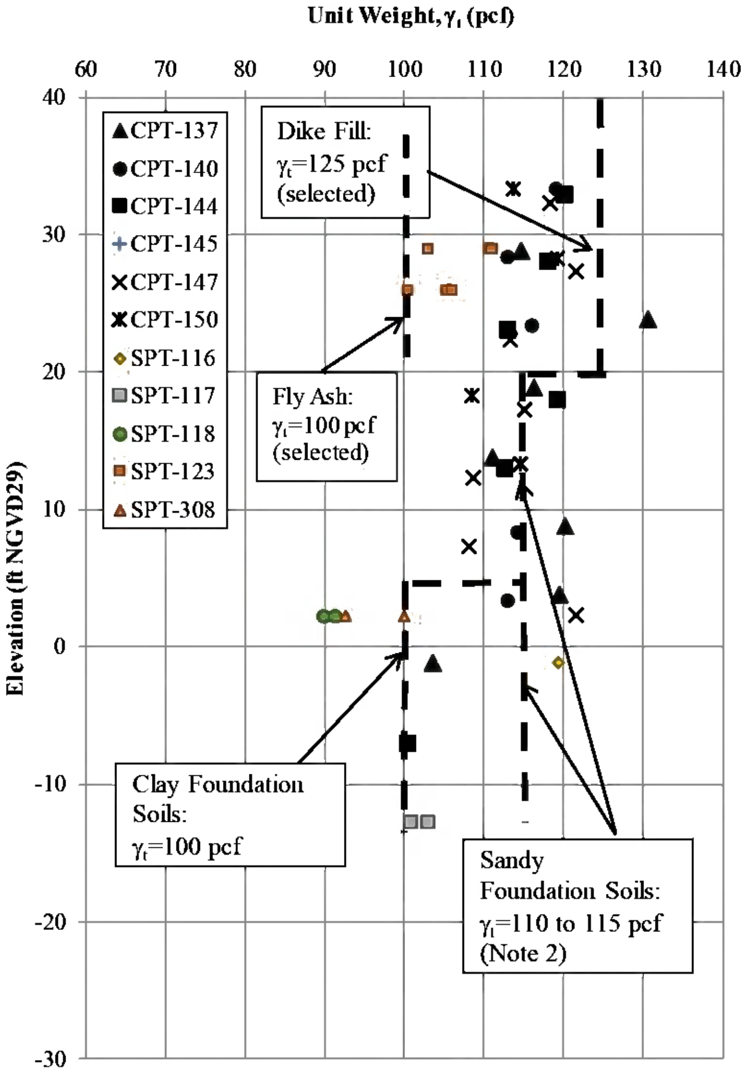

Figure 7. Total Unit Weight from CU and 1-D Consolidation Tests

Note:

1. The total unit weight measurements from triaxial tests are plotted for tests performed on samples from SPT-116, SPT-117, and SPT-118. Shear wave velocity measurements (V_s) were used to supplement laboratory data using the correlation provided by Equation 13.
2. "Loose Sandy Foundation Soils" and "Sandy Foundation Soils" were applied γ_t = 110 pcf and γ_t = 115 pcf, respectively, during the Safety Factor Assessment (Attachment 8).

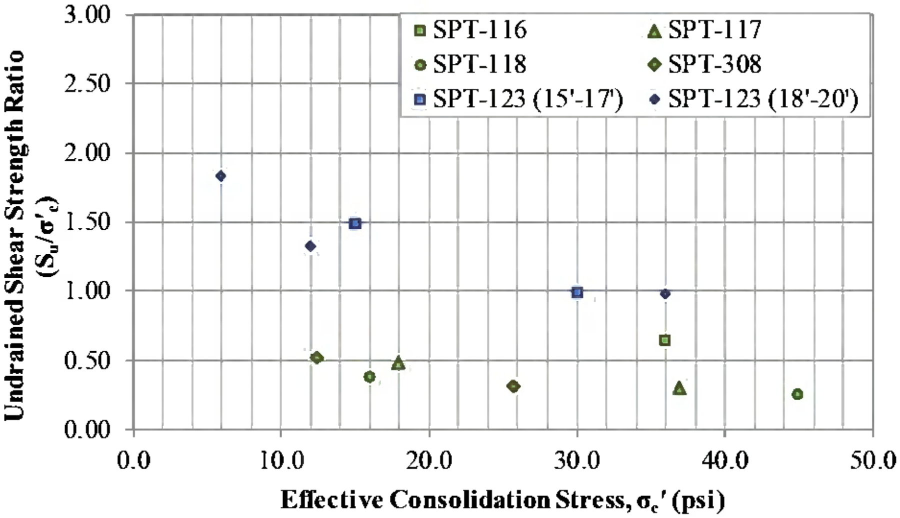

Figure 8. Undrained Shear Strength Ratio from CU Triaxial Tests

Notes:

1. Foundation Soil samples collected from SPT-116, SPT-117, SPT-118, and SPT-308 consisted of clay to sandy clay. Samples collected from SPT-123 consisted of fly ash.
2. Specimen #1 from the CU triaxial test on the sample collected from SPT-123 (15'-17') resulted in a $S_u/\sigma'_c = 6.93$ at $\sigma'_c = 4.0$ psi.
3. Tested samples from SPT-116, SPT-117, SPT-118, and SPT-308 were collected from 41.5 to 43.5 ft bgs, 51.5 to 53.5 ft bgs, 36.5 to 38.5 ft bgs, and 31 to 33 ft bgs, respectively.
4. Tested samples from SPT-123 were collected from 15 to 17 ft bgs and 18 to 20 ft bgs.

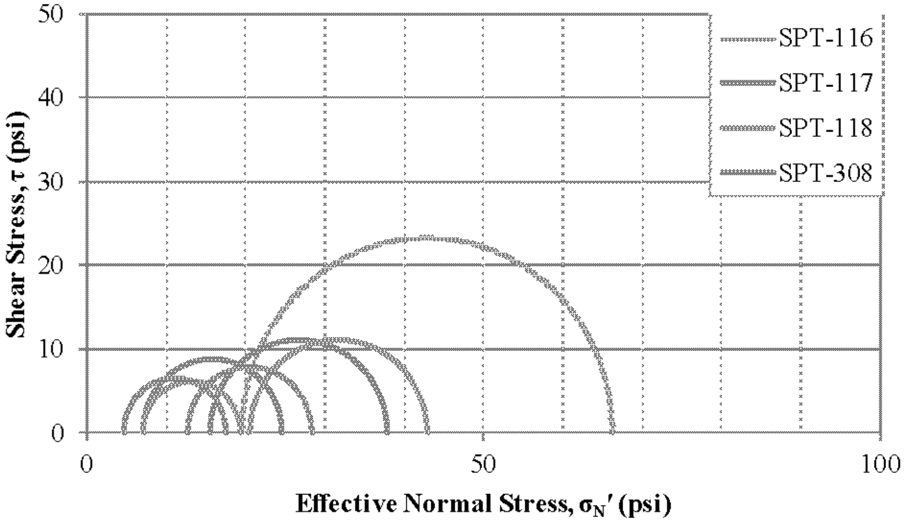

Figure 9. Mohr's Circles for Foundation Soils

Note:

1. Tested samples from SPT-116, SPT-117, SPT-118, and SPT-308 were collected from 41.5 to 43.5 ft bgs, 51.5 to 53.5 ft bgs, 36.5 to 38.5 ft bgs, and 31 to 33 ft bgs, respectively.

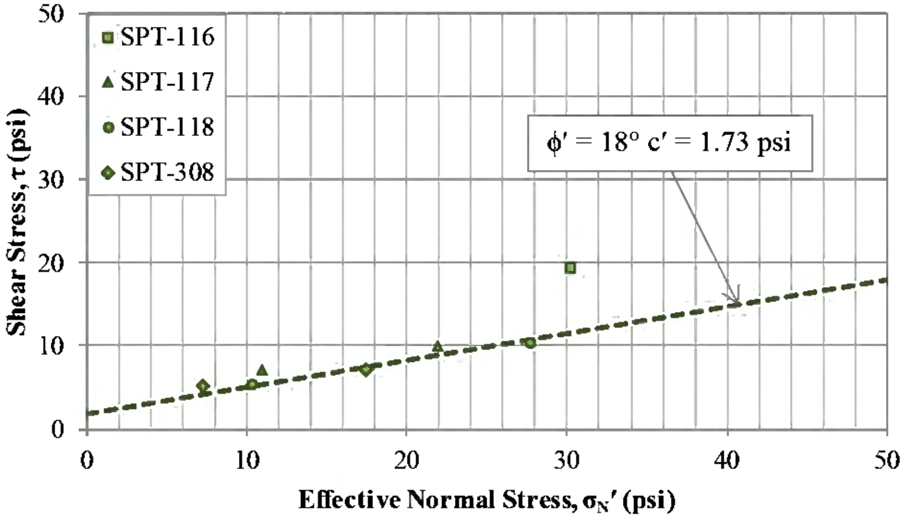

Figure 10. Failure Envelope from CU Triaxial Testing of Clay Foundation Soil

Note:

1. Tested samples from SPT-116, SPT-117, SPT-118, and SPT-308 were collected from 41.5 to 43.5 ft bgs, 51.5 to 53.5 ft bgs, 36.5 to 38.5 ft bgs, and 31 to 33 ft bgs, respectively.

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

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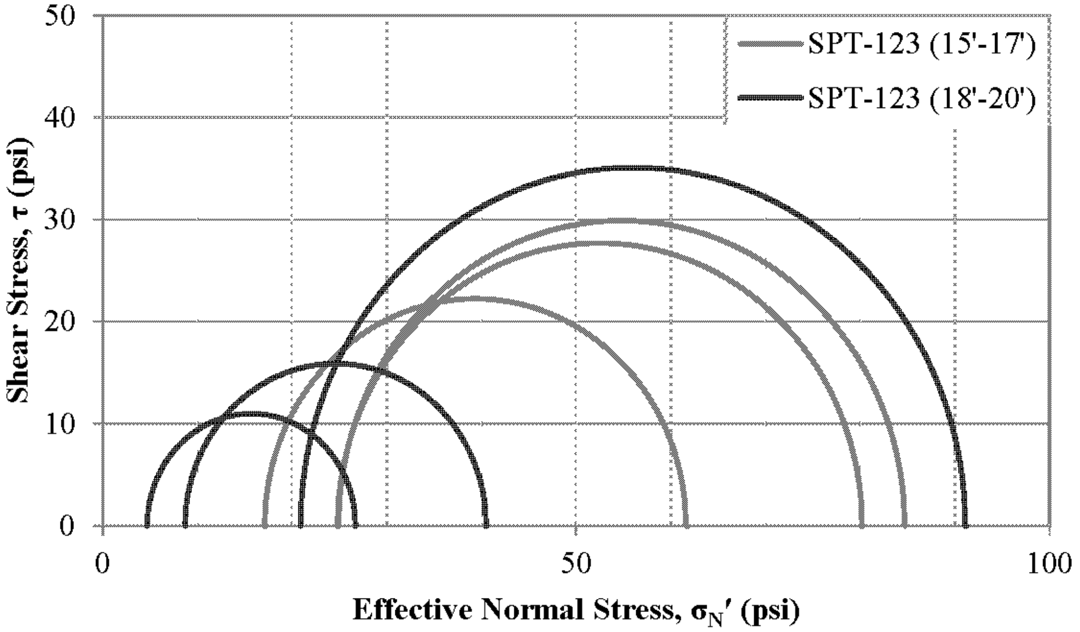

Figure 11. Mohr's Circles for Fly Ash

Note:

1. Tested samples from SPT-123 were collected from 15 to 17 ft bgs and 18 to 20 ft bgs.

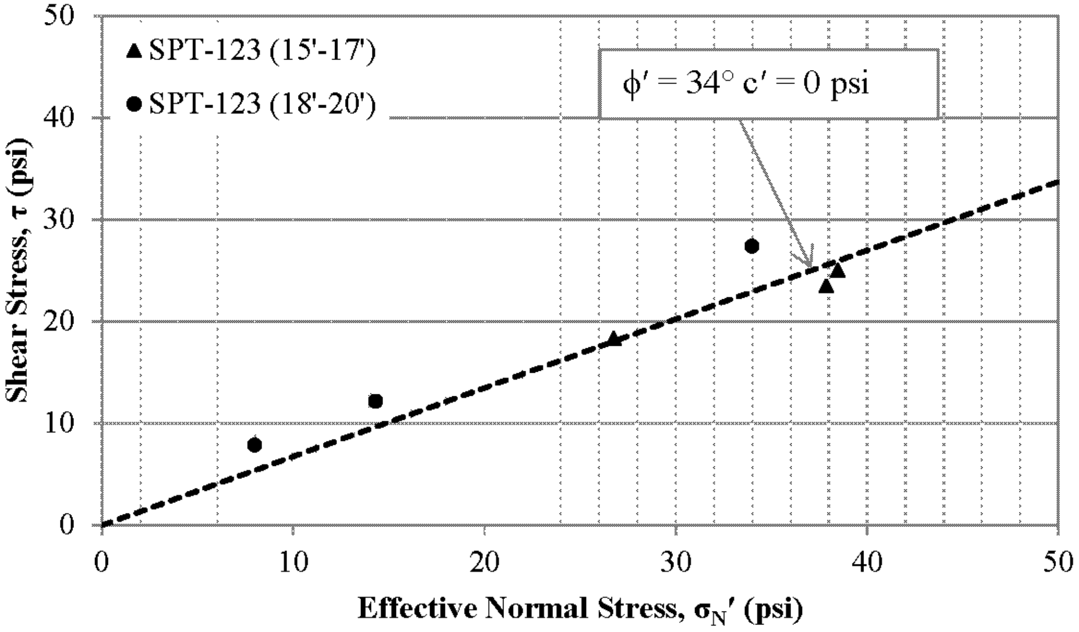

Figure 12. Failure Envelope from CU Triaxial Testing of Fly Ash

Note:

1. Tested samples from SPT-123 were collected from 15 to 17 ft bgs and 18 to 20 ft bgs.

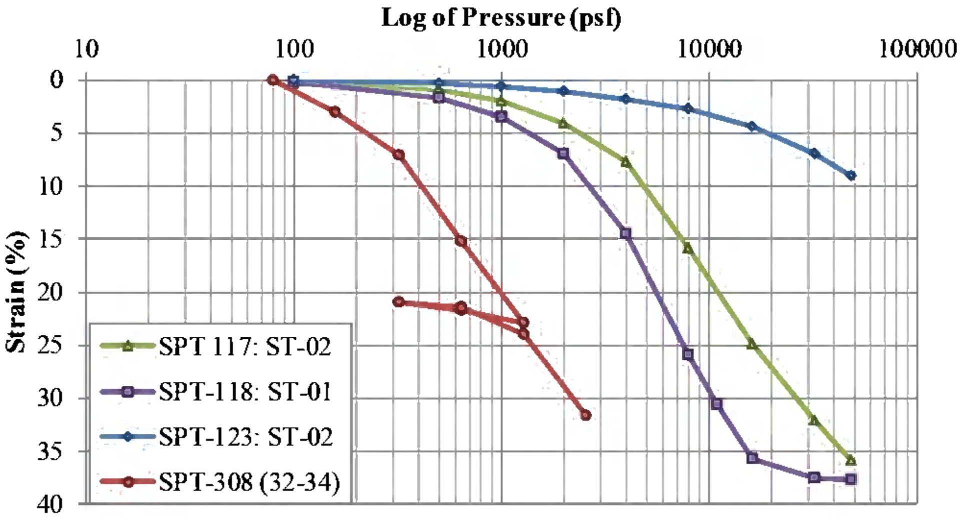

Figure 13. Load-Strain Curves for Clay Foundation Soils and Residual Fly Ash

Notes:

1. The sample from SPT-123 correspond to fly ash and was collected from 18 to 20 ft bgs.
2. The samples from SPT-117 and SPT-118 correspond to clay foundation soils and was collected from 51 to 53.5 ft bgs and 36.5 and 38.5 ft bgs, respectively.
3. The sample from SPT-308, collected from 31 to 33 ft bgs, corresponds to clay soils immediately south of the divider dike between Ash Pond A and Ash Pond B and is included for completeness.

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

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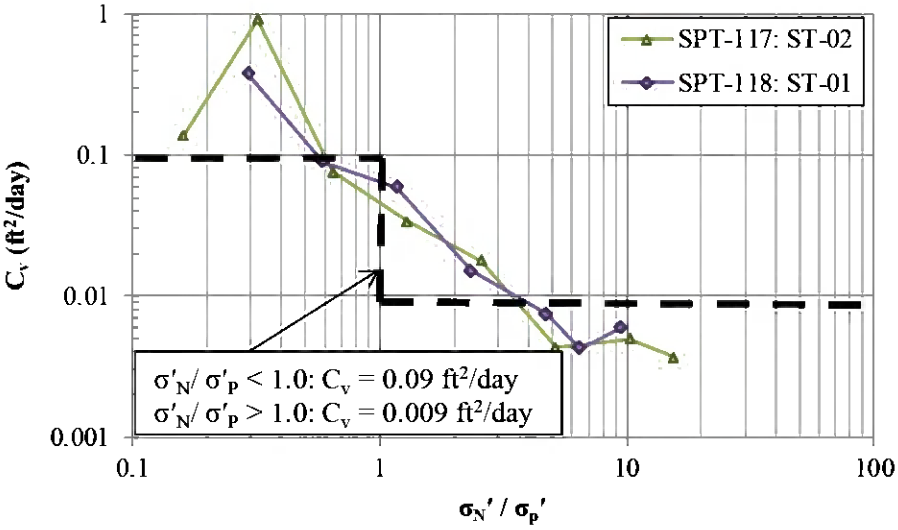

Figure 14. Evaluation of the Coefficient of Consolidation (C_V) for Clayey Foundation Soils

Note:

1. SPT-308 sample appeared to be disturbed and was not included within this plot.

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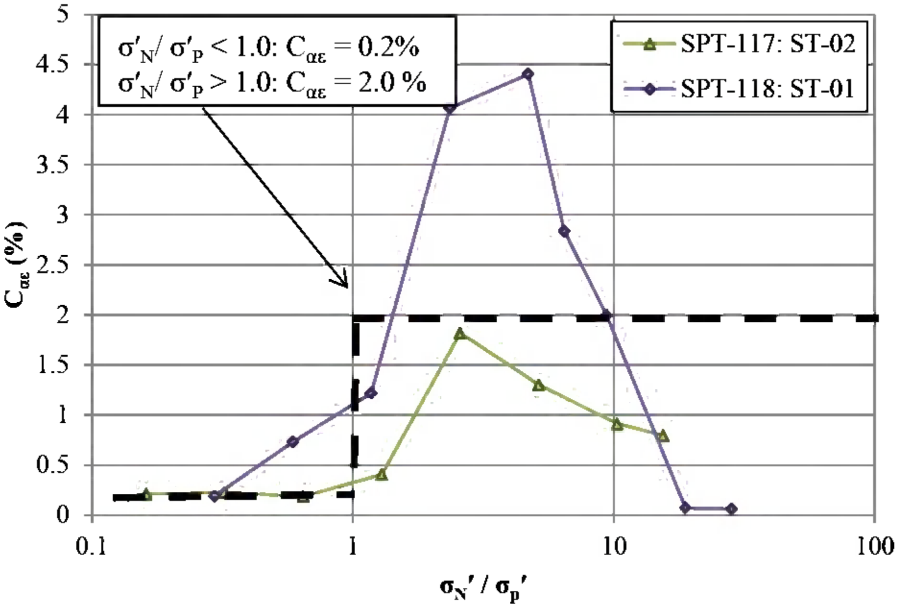

Figure 15. Evaluation of the Coefficient of Secondary Compression ($C_{\alpha\varepsilon}$) for Clay Foundation Soils

Note:

1. SPT-308 sample appeared to be disturbed and was not included in this plot.

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Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GSC5242**	Task No.:	**01BT**

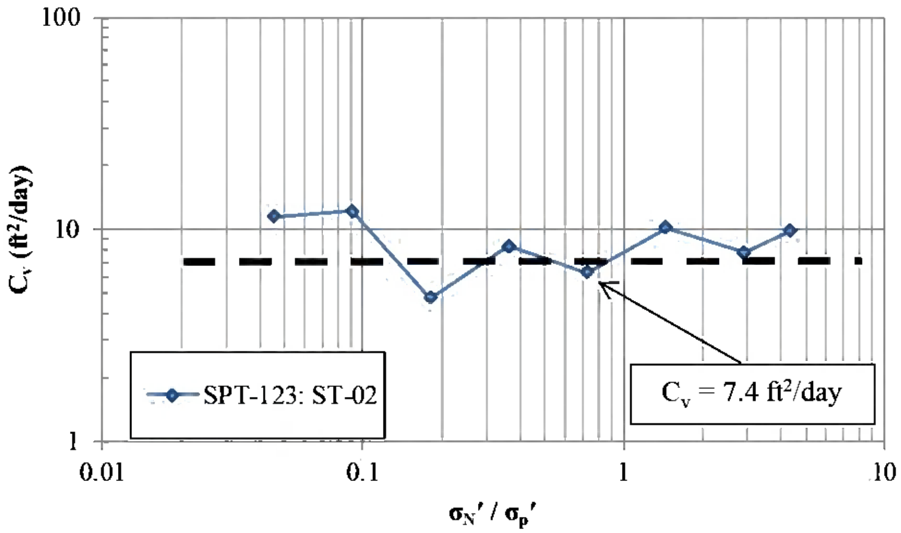

Figure 16. Evaluation of the Coefficient of Consolidation (C_V) for Fly Ash

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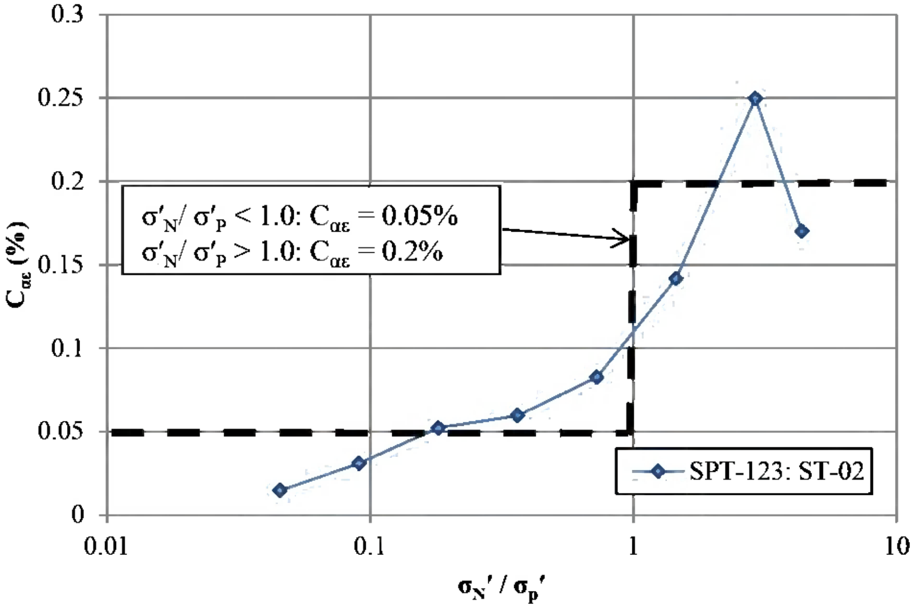

Figure 17. Evaluation of the Coefficient of Secondary Compression ($C_{\alpha\varepsilon}$) for Fly Ash

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Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

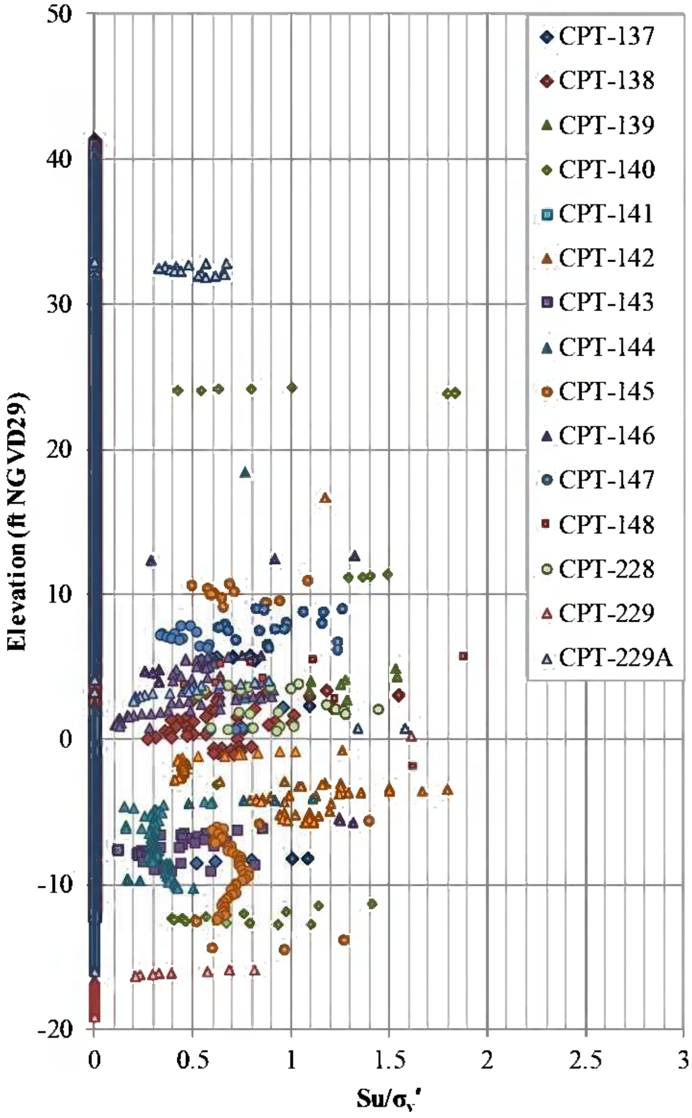

Figure 18. Undrained Shear Strength Ratio from CPT Sounding Correlation

Notes:

1. Undrained shear strength is computed by Robertson and Cabal (2012) for "clay-like" (I_c >2.60) soils. Soils classifying as "sand-like" were plotted with a zero value.
2. CPT-209 through CPT-220 were advanced within the Ash Pond A interior and not considered for the selection of engineering parameters for the perimeter dike structures. These CPTs are not included in this plot.

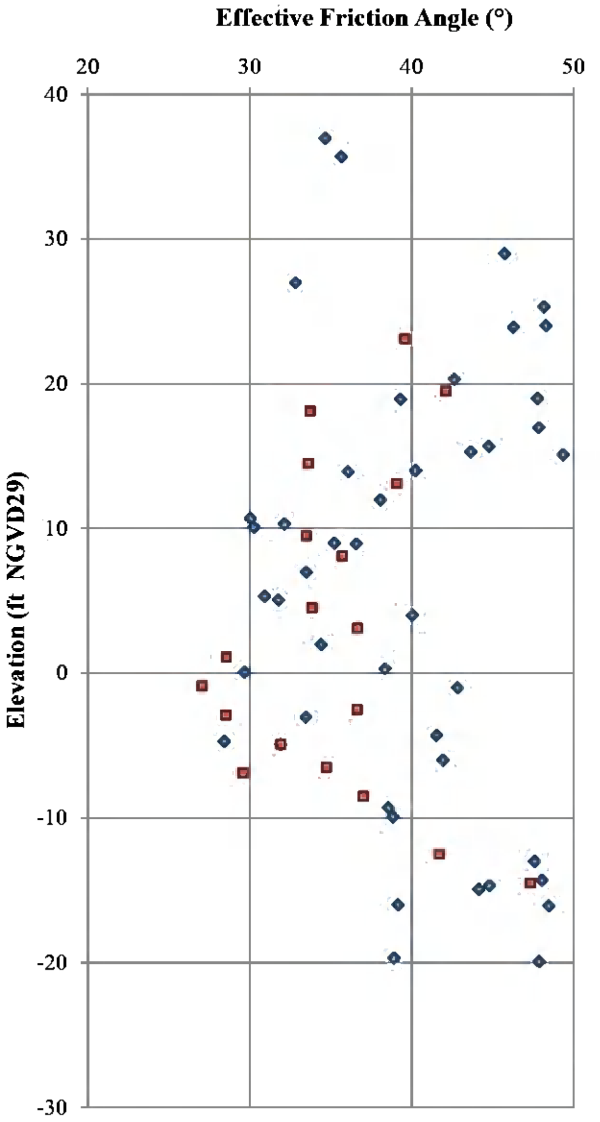

Figure 19. Effective Stress Friction Angle from SPT Correlation

Notes:

1. Effective stress friction angle is computed by Hatanaka and Uchida (1996) for sands.
2. Soil borings performed as a part of the geotechnical site investigation are plotted as blue diamonds; data collected during piezometer installation (i.e., using hollow stem auger / 6-in diameter borehole) is depicted as red squares.
3. Note that SPT-304, SPT-305, and SPT-306 were advanced within the Ash Pond A interior and not considered for the selection of engineering parameters for the perimeter dike structures. These SPTs are not included in this plot.

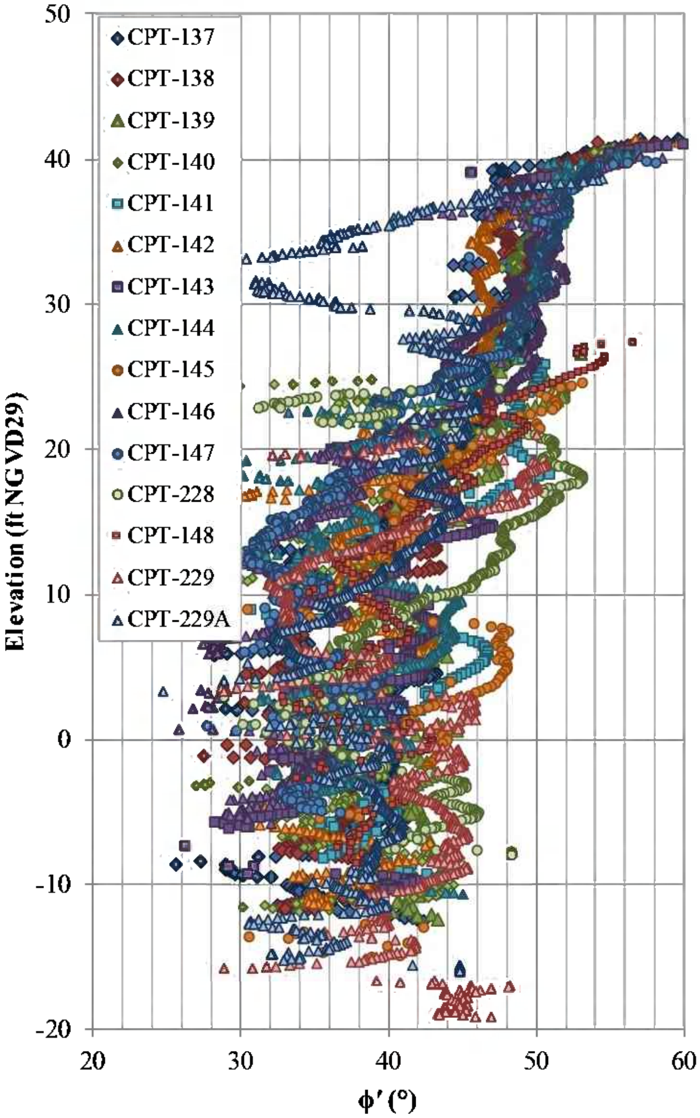

Figure 20. Effective Stress Friction Angle from CPT Correlation

Notes:

1. Effective stress friction angle is computed by Robertson and Campanella (1983) for sands (I_c <2.60).
2. Note that CPT-209 through CPT-220 were advanced within the Ash Pond A interior and not considered for the selection of engineering parameters for the perimeter dike structures. These CPTs are not included in this plot.

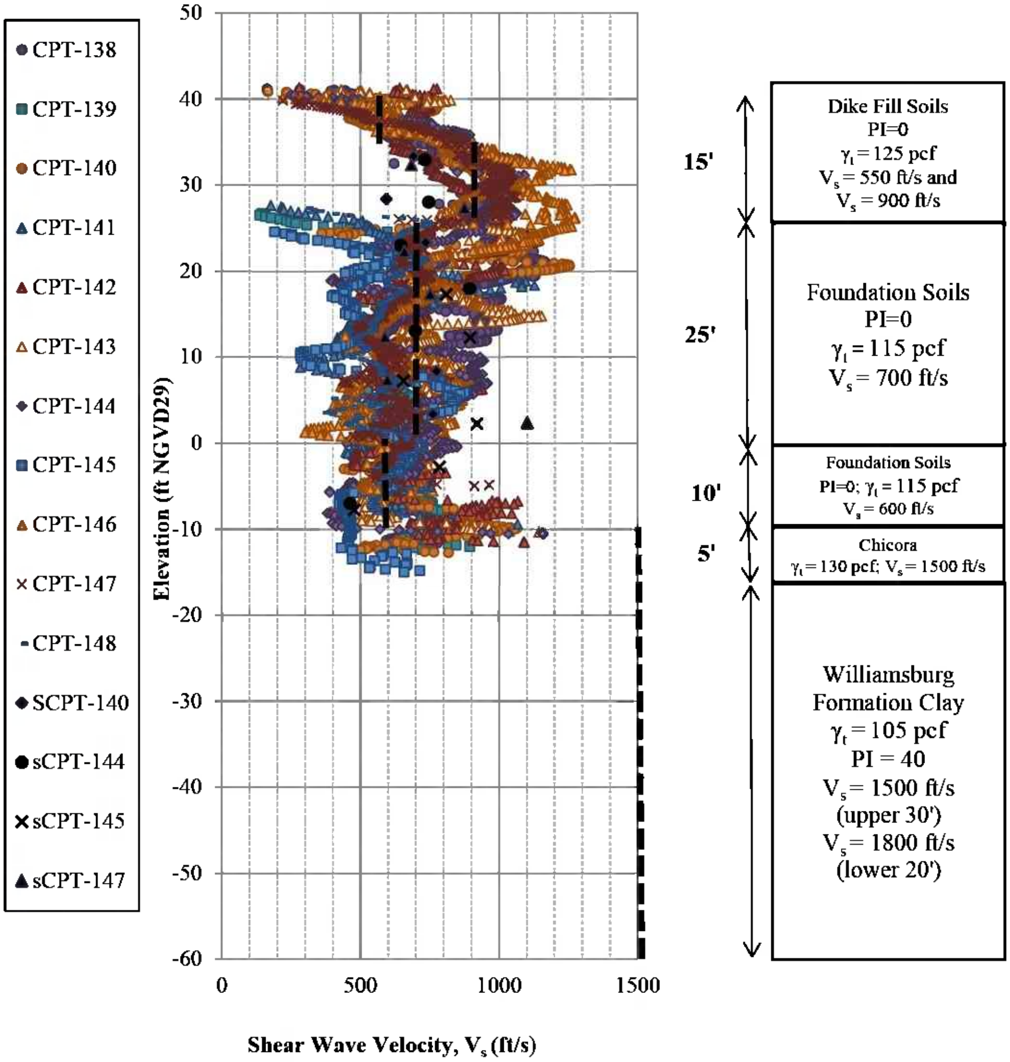

Figure 21. Representative Shear Wave Velocity Profile 1 (for dikes adjacent to Intake and Discharge Canals)

Notes:

1. sCPT refers to a seismic CPT where V_s measurements are collected in 5 ft depth intervals.
2. Representative profile is provided for the subsurface in the upper 100 ft bgs.
3. The upper 5 ft of dike fill soils were modeled with a V_s = 550 ft/s.
4. Note that the development of shear wave velocity profiles and site response analyses were performed prior to the 2016 site investigation and are not included within this figure.

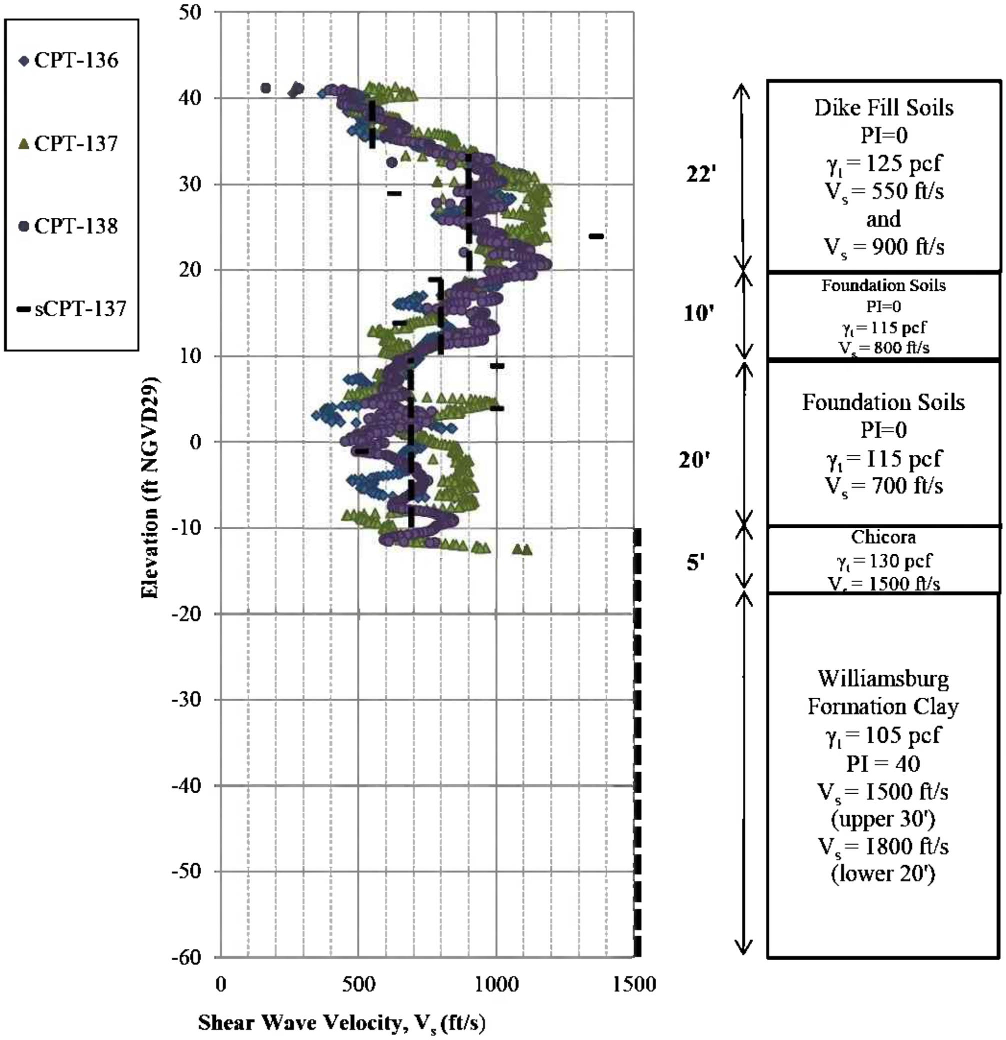

Figure 22. Representative Shear Wave Velocity Profile 2 (for dikes adjacent to the Cooling Pond)

Notes:

1. sCPT refers to a seismic CPT where V_s measurements are collected in 5 ft depth intervals.
2. Representative profile is provided for the subsurface in the upper 100 ft bgs.
3. The upper 5 ft of dike fill soils were modeled with a V_s = 550 ft/s.
4. Note that the development of shear wave velocity profiles and site response analyses were performed prior to the 2016 site investigation and are not included within this figure.

Appendix 1
Summary of Laboratory Testing Results

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

Table 1-1. Summary of Index Testing

Boring ID	Sample ID	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	Specific Gravity	Hydraulic Conductivity	pH
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%	-	cm/s	-
SPT-116	SS-3	15.75	25.69	26.2	-	-	-	-	-	-	-	6.0	-	-	-
SPT-116	SS-6	30.75	10.69	22.9	-	-	-	0.2	88.5	-	-	11.3	-	-	-
SPT-116	SS-7	35.75	5.69	29.9	-	-	-	-	-	-	-	11.2	-	-	-
SPT-116	ST-1	42.50	-1.06	32.0	78	22	56	2.7	71.5	6.2	19.6	25.8	2.724		-
SPT-116	SS-9	45.30	-3.86	23.0	-	-	-	-	-	-	-	13.7	-	-	-
SPT-117	SS-1	5.75	33.99	20.3	-	-	-	0.0	72.3	-	-	27.7	-	-	-
SPT-117	SS-4	20.75	18.99	21.5	-	-	-	-	-	-	-	12.5	-	-	-
SPT-117	SS-6	30.75	8.99	46.6	-	-	-	-	-	-	-	43.5	-	-	-
SPT-117	SS-10	50.75	-11.01	55.1	90	35	55	0.0	42.3	13.3	44.4	57.7	-	-	-
SPT-117	ST-2	52.50	-12.76	58.5	82	24	58	0.2	25.5	15.4	58.9	74.3	-	-	-
SPT-117	ST-3	62.50	-22.76	42.5	65	24	41	0.0	19.5	38.6	41.9	80.5	-	1.40 x 10^{-8}	-
SPT-118	SS-2	10.75	28.92	14.3	-	-	-	0.0	87.0	-	-	13.0	-	-	-
SPT-118	SS-5	25.75	13.92	26.7	-	-	-	-	-	-	-	3.6	-	-	-
SPT-118	SS-6	30.75	8.92	26.5	-	-	-	-	-	-	-	8.2	-	-	-
SPT-118	ST-1	37.50	2.17	-	140	49	91	0.0	1.2	22.4	76.4	98.8	-	-	-
SPT-118	SS-9	45.75	-6.08	20.9	-	-	-	5.4	88.7	-	-	5.9	-	-	-
SPT-119	SS-2	10.75	31.97	36.0	-	-	-	0.4	9.0	-	-	90.6	-	-	-
SPT-119	SS-3	15.75	26.97	19.9	-	-	-	-	-	-	-	9.8	-	-	-
SPT-119	SS-7	35.75	6.97	22.3	-	-	-	1.0	89.8	-	-	9.2	-	-	-
SPT-119	SS-9	45.75	-3.03	25.8	-	-	-	-	-	-	-	9.4	-	-	-
SPT-119	SS-10	50.75	-8.03	16.8	-	-	-	32.7	45.5	-	-	21.8	-	-	-
SPT-120	SS-3	15.75	25.31	18.1	-	-	-	-	-	-	-	13.5	-	-	-
SPT-120	SS-6	30.75	10.31	27.4	-	-	-	-	-	-	-	4.1	-	-	-
SPT-120	SS-7	35.75	5.31	25.5	-	-	-	-	-	-	-	4.3	-	-	-
SPT-120	SS-9	45.75	-4.69	23.3	-	-	-	1.7	86.0	-	-	12.3	-	-	-
SPT-121	SS-2	10.75	30.07	14.8	-	-	-	-	-	-	-	10.7	-	-	-
SPT-121	SS-6	30.75	10.07	27.8	-	-	-	0.0	92.6	-	-	7.4	-	-	-

Table 1-1. Summary of Index Testing (Continued)

Boring ID	Sample ID	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	Specific Gravity	Hydraulic Conductivity	pH
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%	-	cm/s	-
SPT-119	SS-2	10.75	31.97	36.0	-	-	-	0.4	9.0	-	-	90.6	-	-	-
SPT-119	SS-3	15.75	26.97	19.9	-	-	-	-	-	-	-	9.8	-	-	-
SPT-119	SS-7	35.75	6.97	22.3	-	-	-	1.0	89.8	-	-	9.2	-	-	-
SPT-119	SS-9	45.75	-3.03	25.8	-	-	-	-	-	-	-	9.4	-	-	-
SPT-119	SS-10	50.75	-8.03	16.8	-	-	-	32.7	45.5	-	-	21.8	-	-	-
SPT-120	SS-3	15.75	25.31	18.1	-	-	-	-	-	-	-	13.5	-	-	-
SPT-120	SS-6	30.75	10.31	27.4	-	-	-	-	-	-	-	4.1	-	-	-
SPT-120	SS-7	35.75	5.31	25.5	-	-	-	-	-	-	-	4.3	-	-	-
SPT-120	SS-9	45.75	-4.69	23.3	-	-	-	1.7	86.0	-	-	12.3	-	-	-
SPT-121	SS-2	10.75	30.07	14.8	-	-	-	-	-	-	-	10.7	-	-	-
SPT-121	SS-6	30.75	10.07	27.8	-	-	-	0.0	92.6	-	-	7.4	-	-	-
SPT-121	SS-7	35.75	5.07	36.0	-	-	-	-	-	-	-	14.3	-	-	-
SPT-121	SS-8	40.75	0.07	17.3	-	-	-	11.0	73.7	-	-	15.3	-	-	-
SPT-121	SS-9	45.75	-4.93	31.1	-	-	-	-	-	-	-	14.5	-	-	-
SPT-123[1]	ST-2	16.00	28.96	-	NP	NP	NP	0.0	3.8	70.4	25.8	96.2	2.308	-	5.7
SPT-123	ST-3	19.00	25.96	-	NP	NP	NP	0.0	80.2	10.8	9.0	19.8	-	-	-
SPT-304	13.5-15	14.25	29.34	54.0	NP	NP	NP	0.1	7.1	-	-	92.7	-	-	-
SPT-304	30-32	31.00	12.59	27.0	-	-	-	-	-	-	-	2.5	-	-	-
SPT-304	38-40	39.00	4.59	23.0	-	-	-	-	-	-	-	6.3	-	-	-
SPT-304	48.5-50	49.25	-5.66	31.0	-	-	-	-	-	-	-	15	-	-	-
SPT-305	13.5-15	14.25	30.47	33.0	NP	NP	NP	0.0	6.9	-	-	93.1	-	-	-
SPT-305	23.5-25	24.25	20.47	26.0	NP	NP	NP	1.0	43.5	-	-	55.5	-	-	-
SPT-305	30-32	31.00	13.72	29.0	-	-	-	-	-	-	-	4.2	-	-	-
SPT-305	38-40	39.00	5.72	28.0	-	-	-	-	-	-	-	6.6	-	-	-
SPT-305	48-50	49.00	-4.28	22.0	NP	NP	NP	0.1	94.2	-	-	5.6	-	-	-
SPT-306	23.5-25	24.25	19.94	30.0	-	-	-	-	-	-	-	8.6	-	-	-
SPT-306	32-34	33.00	11.19	28.0	NP	NP	NP	-	-	-	-	4.7	-	-	-

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **C. Carlson/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

Table 1-1. Summary of Index Testing (Continued)

Boring ID	Sample ID	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	Specific Gravity	Hydraulic Conductivity	pH
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%	-	cm/s	-
SPT-306	53.5-55	54.25	-10.06	17.0	NP	NP	NP	37.3	50.5	-	-	12.2	-	-	-
SPT-306	73.5-75	74.25	-30.06	49.0	59	26	33	2.7	31.7	-	-	65.7	-	-	-
SPT-306	93.5-95	94.25	-50.06	48.0	56	34	22	3.2	88.8	-	-	8.0	-	-	-
SPT-306	108.5-110	109.25	-65.06	65.0	77	38	39	0.0	27.5	-	-	72.5	-	-	-
SPT-306	123.5-125	124.25	-80.06	27.0	NP	NP	NP	-	-	-	-	20.8	-	-	-
SPT-306	138.5-140	139.25	-95.06	37.0	36	24	12	0.0	47.6	-	-	52.4	-	-	-
SPT-306	148.5-150	149.25	-105.06	30.0	24	21	3	5.1	43.2	-	-	51.6	-	-	-
SPT-306	158.5-160	159.25	-115.06	34.0	NP	NP	NP	0.0	71.1	-	-	28.9	-	-	-
SPT-306	163.5-165	164.25	-120.06	36.0	NP	NP	NP	0.0	67.8	-	-	32.2	-	-	-

Notes:

1. Carbonate content was measured in accordance with ASTM D4373. The carbonate content was measured as 0% in the samples tested.
2. Elevations are in ft NGVD29.

Table 1-2. Summary of Triaxial Testing Results (from EGT or Terracon)

Boring ID	Depth	Elevation	Moisture Content	Dry Unit Weight	Wet Unit Weight	σ_{consol}'	$\sigma_{1,f}'$	$\sigma_{3,f}'$	S_u	S_u / σ_c'
Units	ft bgs	ft NGVD29	%	pcf	pcf	psi	psi	psi	psi	-
SPT-116	42.5	-1.1	32.0	90.3	119.2	36.0	66.2	19.6	23.3	0.65
SPT-117	52.5	-12.8	50.6	68.4	103.0	18.0	24.6	7.1	8.8	0.49
SPT-117	52.5	-12.8	51.5	66.6	100.9	37.0	37.9	15.5	11.2	0.30
SPT-118	37.5	2.2	100.6	44.8	89.9	16.0	19.3	7.1	6.1	0.38
SPT-118	37.5	2.2	103.0	45.0	91.4	45.0	42.9	20.5	11.2	0.25
SPT-123	16.0	29.0	44.0	76.8	110.6	4.0	80.2	24.8	27.7	6.93
SPT-123	16.0	29.0	44.3	71.4	103.0	15.0	61.6	17.1	22.3	1.49
SPT-123	16.0	29.0	45.1	76.5	111.0	30.0	84.7	24.9	29.9	1.00
SPT-123	19.0	26.0	49.9	67.0	100.4	6.0	26.7	4.7	11.0	1.83
SPT-123	19.0	26.0	28.3	82.0	105.2	12.0	40.5	8.7	15.9	1.33
SPT-123	19.0	26.0	22.9	86.3	106.1	36.0	91.1	20.9	35.1	0.98
SPT-308	33.0	2.3	115.4	46.4	99.9	12.5	17.58	4.6	6.5	0.52
SPT-308	33.0	2.3	145.0	37.8	92.6	25.7	28.43	12.6	7.9	0.31

ATTACHMENT 6

Seismic Hazard Evaluation and Site Response Analysis

Geosyntec
consultants

CALCULATION PACKAGE COVER SHEET

Client: Santee Cooper **Project:** Winyah Generating Station **Project No.** GSC5242

TITLE OF PACKAGE: SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS: ASH POND A

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Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**

SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS: ASH POND A

PURPOSE

The purpose of this calculation package is to present the results of the seismic hazard evaluation and site response analyses performed for Ash Pond A at the Winyah Generating Station (WGS or "Site"). This calculation package is provided as Attachment 6 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report* (Safety Factor Assessment Report). Seismic hazard analysis for the Site includes the selection of an appropriate hazard level and associated hazard parameters. Based on the selected hazard level and associated hazard parameters, site response analyses were performed to evaluate the local site effects on the selected time history records propagated from the hypothetical firm ground outcrop to the ground surface of the Site. The objective of this site response analysis is to calculate accelerations and shear stresses within the representative soil profiles of the Ash Pond A perimeter dikes. Cyclic shear stresses will be examined to evaluate liquefaction potential for dike fill and foundation soils and to calculate the seismic coefficient for seismic slope stability analyses presented in Attachments 7 and 8 of the Safety Factor Assessment Report, respectively.

SEISMIC HAZARD EVALUATION

Seismic hazard analysis for the Site includes the selection of: (i) appropriate hazard level; and (ii) associated hazard parameters. The appropriate hazard level is often expressed in probabilistic terms as a specific hazard level that has a certain probability of exceedance in a given time period. Selecting the hazard parameters includes developing an understanding of the seismic sources, ground motion attenuation, and site response. The goals of this section are to: (i) develop the target response spectrum, including the peak ground acceleration (PGA), at a hypothetical firm ground outcrop at WGS corresponding to the appropriate seismic hazard level; (ii) select the earthquake magnitude that contributes predominantly to the seismic hazard at WGS; and (iii) select a set of ground motion time histories that envelope the target spectrum, and are generally consistent with the source and path characteristics of ground motions at WGS.

Seismic Hazard Level

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published the CCR Rule (40 Code of Federal Regulations [CFR] Parts 257 and 261). §257.63(a) of the CCR Rule states that:

"New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones, unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site."

§257.53 of the CCR Rule defines the maximum horizontal acceleration in lithified earth material as:

"... the maximum expected horizontal acceleration at the ground surface as depicted on a seismic hazard map, with a 98 percent or greater probability that the acceleration will not be exceeded in 50 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment."

As the purpose of the Safety Factor Assessment Report is to demonstrate compliance of the existing CCR surface impoundments at WGS with the structural integrity criteria provided in §257.73, the seismic factor of safety must also exceed 1.0 considering *"the peak ground acceleration for a seismic event with a 2% probability of exceedance in 50 years, equivalent to a return period of approximately 2,500 years, based on the U.S. Geological Survey (USGS) seismic hazard maps"*.

Therefore, the analysis performed herein is based on design parameters consistent with a 98 percent probability that the PGA will not be exceeded in 50 years. This hazard level results in seismic design parameters consistent with a 2 percent probability that the PGA will be exceeded in 50 years. This selected hazard level has a return period of 2,475 years, which is commonly referred to as a 2,500-year event.

Peak Ground Acceleration (PGA)

PGA values corresponding to different hazard levels and different site conditions, including firm ground outcrops, are published as seismic hazard maps. While USGS national seismic hazard maps are the most commonly used resources for the selection of PGA, regional seismic hazard maps developed by local experts consider regional geologic setting and seismicity and are often the preferred alternatives.

USGS national seismic hazard maps for a 2 percent probability of exceedance in 50 year ground motion (i.e., 2,475-year return period event) provide the PGA and spectral accelerations for a hypothetical firm ground outcrop at the Site. The software available at the USGS website (USGS, 2008) uses pre-calculated hazard values at nearby grid locations and interpolates the hazard value for a given site location. As presented in Appendix 1, the USGS interpolated PGA is 0.469g for the Site.

The South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (SCDOT, 2010) presents seismic hazard maps with PGAs for "geologically realistic" site conditions, as well as for the hypothetical "hard-rock" basement outcrop conditions for locations throughout SC. The SCDOT seismic hazard maps were developed by Chapman and Talwani (2006) to incorporate their local experience and research over several decades for the Charleston Seismic Zone. The "geologically realistic" site condition is a hypothetical site condition that was included via a depth-dependent transfer (i.e., site amplification) function for Coastal Plain and non-Coastal Plain regions of SC. The Coastal Plain "geologically realistic" site condition was modeled with two layers: (i) the shallowest layer consisting of Coastal Plain sedimentary soils (γ = 125 pcf, shear wave velocity, V_s = 2,300 ft/s); and (ii) weathered rock (γ = 155 pcf, V_s = 8,200 ft/s) over a half-space of unweathered Mesozoic and Paleozoic sedimentary and Metamorphic/Igneous rock (γ = 165 pcf, V_s = 11,200 ft/s). Conversely, the USGS national seismic hazard maps were developed using a generic site amplification function that does not account for the soil conditions in the Coastal Plain of SC as well as the SCDOT maps.

The SCDOT (2010) seismic hazard maps for a probability of exceedance of 2 percent in 50 years for the "geologically realistic" and "hard rock" conditions are presented in Appendix 1. The PGA seismic hazard map for the "geologically realistic" condition is also presented in Figure 1. The Site PGA is about 0.16g and 0.21g for "geologically realistic" and "hard rock" conditions, respectively. A site response analysis can be performed either by: (i) using the hard rock acceleration as the hypothetical outcrop acceleration and modeling the soil/rock column extending to the rock layer with a V_s =11,200 ft/s; or (ii) using the "geologically realistic" acceleration as the hypothetical outcrop acceleration and modeling the soil column extending to the firm ground layer with V_s = 2,300 ft/s. The latter approach will be used for this project because it is less practical to extend the site response model to reach a hard rock outcrop with V_s = 11,000 ft/s at reasonable depths in the SC Coastal Plain region.

SCDOT hazard maps for "geologically realistic" conditions were used to select the PGA for the hypothetical firm ground outcrop at WGS when evaluating the seismic response of existing CCR surface impoundments. While the approach used for developing the SCDOT maps and USGS maps is the same (i.e., a probabilistic seismic hazard analysis), the following key features are noted by Chapman and Talwani (2006) with regards to their study: (i) inclusion of alternative source configurations for earthquakes in the magnitude range from 5.0 to 7.0; (ii) use of alternative source models for larger, characteristic-type earthquakes with magnitudes 7.0 to 7.5 in the coastal areas of SC; (iii) use of a maximum magnitude for characteristic earthquakes in the coastal areas; and (iv) more accurate representation of actual geologic conditions in SC. A PGA value of 0.16g is selected at the Site using the hazard maps for "geologically realistic" hypothetical firm ground outcrop conditions.

Earthquake Magnitude

In a probabilistic seismic hazard analysis, the PGA cannot be associated with a single earthquake event due to the hazard contribution from multiple possible events. An earthquake moment magnitude (M_W) value is required to conduct liquefaction potential analyses and to select earthquake time histories. A process called deaggregation can be performed for sites that have multiple hazard sources using the USGS (2002) deaggregation tool.

Figure 2 presents the deaggregation for the PGA near Georgetown, South Carolina. A 7.3 moment magnitude earthquake event at a source-to-site distance of approximately 70 km appears to be the main event contributing to the hazard at the site. Thus, a 7.3 moment magnitude was selected for liquefaction potential analyses and time history selection for WGS.

Target Acceleration Response Spectra

The target spectrum for a "geologically realistic" site was selected using the SCDOT seismic hazard maps for different spectral periods (or frequencies) as presented in Appendix 2. This spectrum is presented in Figure 3. The "geologically realistic" target acceleration response spectrum has a PGA (represented by a spectral period of 0.01 seconds) of 0.16g and a peak spectral acceleration of 0.48g at a spectral period of 0.2 seconds. As stated previously, the "geologically realistic" condition target acceleration response spectrum was selected for WGS.

Time Histories

Time histories of ground motions are used as input for site response analysis and are selected such that their response spectra match or envelope the target spectrum. While use of recorded ground motion time histories from earthquakes with similar source characteristics is preferred, synthetic motions may be used if recordings are not available for a particular seismic zone. Earthquake events with a moment magnitude, M_W, 7.0 or greater have not occurred in the stable continental tectonic environment of the Central and Eastern United States since the Charleston earthquake in 1886, so ground motion time history records matching the seismic source characteristics for the WGS are generally not available. Two synthetic acceleration time histories were selected from the six synthetic acceleration time histories developed for the Site using the USGS Interactive Deaggregation tool (USGS, 2002). These time histories are referred to herein as Winyah1 and Winyah2, and provide a reasonable match to the short-period portion of the "geologically realistic" target acceleration response spectrum. Three time histories, BOS-T1, DEL090, and YER360, developed by McGuire et al. (2001) as part of a study for the Nuclear Regulatory Commission to provide time histories representative of expected earthquake events in the Central and Eastern United States were selected to provide a reasonable match with the long-period portion of the "geologically realistic" target acceleration response spectrum. One time history, RSN8529-HNE, from the Next Generation Attenuation – East (NGA East) database (Goulet et al., 2014), which provides a database of time histories recorded for earthquake events in the Central and Eastern United States, was selected to also provide a reasonable match with the "geologically realistic" target acceleration response spectrum for longer periods. As shown in Figure 4, this suite of six time histories provides a reasonable envelope of the "geologically realistic" target spectrum for the Site over a broad range of periods. Time histories were scaled in the site response evaluation computer program to match the target PGA of 0.16g. These scaled acceleration time histories are presented in Appendix 3. Additional details of the time histories are presented in Table 1.

SITE RESPONSE ANALYSIS

Site response analyses were performed to evaluate the effect of local site conditions on the expected ground motions at the Site. The objective of the site response analysis is to calculate accelerations and shear stresses within the Site soil profiles. Shear stresses are examined to evaluate the liquefaction potential analysis (Attachment 7 of the Safety Factor Assessment Report) and seismic stability analysis (Attachment 8 of the Safety Factor Assessment Report).

Methodology for Site Response Analysis

Site response analyses presented herein were conducted using DEEPSOIL® (Hashash et al., 2015), a one-dimensional nonlinear site response analysis program. The program assumes that all the soil layers are perfectly horizontal (i.e., "layer cake") and that ground response is mainly caused by vertically-propagating, horizontally polarized shear waves. This assumption is valid for many geotechnical cases including the analyses of the Site. Under these assumptions, the subsurface stratigraphy is modeled as a one-dimensional column of soil layers for the analyses.

DEEPSOIL® employs a viscoelastic material model, described by its shear modulus (G), mass density (ρ) or unit weight (γ), and material damping ratio (D). Preliminary equivalent-linear site response analyses yielded calculated maximum shear strains greater than 5 percent in some layers, which is greater than the shear strains for which equivalent-linear analyses are considered applicable (i.e., 1 to 2 percent). Therefore, nonlinear site response analyses were performed.

Input Parameters for Site Response Analysis

Input Motions

As discussed in the Time Histories subsection, six acceleration time histories were selected and scaled to match the design PGA of 0.16g. These ground motions were applied as outcrop motions in DEEPSOIL® at the top of the half space with V_s = 2,300 ft/s.

Representative Soil Profile

A detailed description of the subsurface stratigraphy is presented in Attachment 5 of the Safety Factor Assessment Report titled "*Subsurface Stratigraphy and Material Properties: Ash Pond A*" (Data Package). Information that is specific to the site response analysis is presented herein. To develop representative soil profiles, the Ash Pond A perimeter dike was divided into two sections depending on the depth of the dike fill and the V_s profile of the subsurface as shown in Figure 5. The top of the dike is roughly at the same elevation by the intake/discharge canals and the cooling pond. However, the dike fill extends to greater depths by the cooling pond than by the intake/discharge canals. Two representative profiles to 100 ft below ground surface (bgs) were developed for the perimeter dike: (i) one by the intake/discharge canals (Profile 1); and (ii) one by the cooling pond (Profile 2). For both profiles, the water table was assumed to be at a depth of 15 ft bgs. The two representative profiles are shown in Figure 6.

Profiles 1 and 2 were extended to a depth of 500 ft bgs using information on deep V_s profiles derived from URS (2001) and S&ME (2001). At that depth, the deep V_s profiles indicate the presence of firm Coastal Plain sediments with V_s of approximately 2,300 ft/s, which is consistent with the definition of "geologically realistic" soil conditions described previously. The site response analysis presented in this package thus considers the full depth of the soil columns (i.e., 500 ft bgs), but results are presented for the soil columns to a depth of approximately 100 ft bgs to emphasize the near-surface response.

Dynamic Soil Properties

Shear Modulus Reduction and Damping Curves

The modified Kondner-Zelasko model implemented in DEEPSOIL® is described in Matasovic (1993). The shear modulus reduction and damping curves are required as input parameters to the constitutive soil model, and were developed for regional soil characteristics based on guidance presented in the SCDOT Geotechnical Design Manual (2010) and previous geotechnical reports of the Site. Adopting relationships proposed by Stokoe et al. (1995 and 1999), Andrus et al. (2003)

developed regression equations for shear modulus reduction and damping curves suitable for South Carolina soils. The regression equations are presented in the SCDOT Geotechnical Design Manual (2010). These region-specific curves are a function of the plasticity index (PI) of the soil, effective mean stress, and geologic age and location of soil deposits. Geologic interpretation of the foundation soil at WGS by Paul C. Rizzo Associates (PCRA) (PCRA, 1999) and the SC Department of Natural Resources (DNR) (2012) indicates the native foundation soils above the Chicora and Williamsburg Formation strata are Pleistocene deposits. The dike fill soils were considered to be a Holocene deposit, since the perimeter dikes were constructed of compacted earthen fill in 1979-1980. The SCDOT (2010) shear modulus reduction and damping curves were calculated for the dike fill and foundation soils located above the Chicora and Williamsburg Formation strata. Soft rock curves (Silva et al., 1997) were selected for the Chicora and Williamsburg Formation strata to be consistent with the V_s-based classification indicating soft rock conditions. Pacific Engineering (S&ME, 2001) also used these soft rock shear modulus reduction and damping curves to perform the site response analysis of an ammonia tank building onsite. Figure 7 presents shear modulus reduction and damping curves used for these analyses. An example of the development of the dynamic curves and the references are provided in Appendix 4.

Representative Shear Wave Velocity Profile

Geosyntec developed representative V_s profiles of the dike fill and foundation soils using both direct measurements from Seismic Cone Penetration Tests (SCPTs) and estimates using Cone Penetration Tests (CPTs) and associated correlations. Upon evaluation of several correlations, the Mayne (2006) correlation was found to agree most closely with results of site-specific V_s measurements. This correlation is as follows:

$$V_s = 118.8 \log (f_s) + 18.5$$

where,

V_s = shear wave velocity (m/sec); and

f_s = sleeve friction from CPT (kPa).

Appendix 5 presents SCPT measurements, estimated values, and selected V_s profiles. Figure 8 shows the shallow (depths less than 100 ft bgs) V_s profiles used for the site response analyses presented herein. As described previously, these profiles were extended to greater depths to layers with V_s of approximately 2,300 ft/s to be consistent with the definition of "geologically realistic" soil conditions.

Unit Weight

Unit weights of the dike fill and foundation soils were selected predominantly based on laboratory measured values as presented in the Data Package. The selected unit weight of the dike fill was 125 pcf. The selected unit weight of the foundation soils was 115 pcf. Unit weights of the Chicora and Williamsburg Formation soils were assumed to be 130 pcf and 105 pcf, respectively, based on Standard Penetration Test (SPT) N-values and material descriptions presented in the PCRA (1999) report. Williamsburg Formation soils at depths greater than approximately 110 feet were assumed to have unit weights of 125 pcf.

Site Response Analysis Results

Figures 9a and 9b show calculated maximum shear strains and shear stresses for Profiles 1 and 2, respectively. The maximum shear strains produced by two of the motions (BOS-T1 and YER360) are relatively large in the foundation soils, supporting the use of nonlinear site response analyses. Calculated accelerations within the soil profiles are presented in Appendix 6. The envelopes of maximum shear strain and shear stress for the six motions for each profile are presented in Figure 10. The calculated envelopes of maximum shear stress (τ_{max}) values for different depths are presented in Table 2. These values were used to calculate cyclic stress ratios for the evaluation of liquefaction potential (Attachment 7 in the Safety Factor Assessment Report) and to calculate the seismic coefficient for seismic stability analyses (Attachment 8 in the Safety Factor Assessment Report).

CONCLUSIONS

- The design PGA was selected to be 0.16g. This firm ground PGA corresponds to an event with a probability of exceedance of 2 percent in 50 years and is representative of a motion expected for the "geologically realistic" site condition presented in the SCDOT Geotechnical Design Manual (2010).

- The design earthquake was assumed to have an M_w of 7.3 based on the deaggregation of the probabilistic seismic hazard analysis. This M_w was used for soil liquefaction analysis and time history selection.

- A target response spectrum for "geologically realistic" site conditions was developed using the SCDOT seismic hazard maps and is presented in Figure 4.

- Six time history recordings were selected. Two synthetic time histories were obtained using the USGS Interactive Deaggregation tool (USGS, 2002), three of the time histories were selected from the McGuire et al. (2001) database, and one of the time histories was selected from the NGA East database (Goulet et al., 2014). The time histories were scaled to match the design PGA of 0.16g for site response analyses.

- Nonlinear site response analyses were conducted using DEEPSOIL® (Hashash et al., 2015). The soil profiles were developed based on results of subsurface exploration and historical site data. The analyses used region-specific shear modulus reduction and damping curves. The shear wave velocity profiles were estimated from measured SCPT values and correlations between V_s and measured CPT sleeve frictions. The inputs used for each profile in DEEPSOIL® are shown in Appendix 7.
- The site response analysis results are presented in Figures 9a and 9b and Figure 10. The calculated maximum shear stresses are presented in Table 2 and are used for evaluation of soil liquefaction potential and calculation of the seismic coefficient for seismic stability analyses.

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Tables

Table 1. Summary of Hazard Parameters of the Time Histories Selected for Site Response Analysis

Name	**Site Class**	**M_w**	**R (km)**	**PGA (g)**	**T_p (s)**
BOS-T1	-	7.40	26.1	0.14	0.36
DEL090	C	6.70	59.3	0.27	0.22
RSN8529-HNE	C	5.74	124.1	0.09	0.26
Winyah1	A	7.04	30.2	0.56	0.08
Winyah2	A	7.04	30.2	0.56	0.10
YER360	C	7.30	24.9	0.22	0.22

Note:

1. All accelerations are scaled within DEEPSOIL® to match the target PGA of 0.16g.

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Table 2. Calculated Maximum Shear Stress Envelopes

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	31	2.5	40
7.5	60	7.5	99
12.5	95	12.5	141
17.5	131	17.0	176
22.5	163	20.5	206
27.5	193	24.5	239
32.5	212	29.5	270
37.5	225	34.5	291
42.5	227	39.5	305
47.5	235	44.5	314
52.5	317	49.5	327
60.0	392	54.5	418
70.0	523	62.0	525
80.0	583	72.0	631
90.0	660	82.0	735
100.0	783	92.0	827
110.0	889	102.0	908

Figures

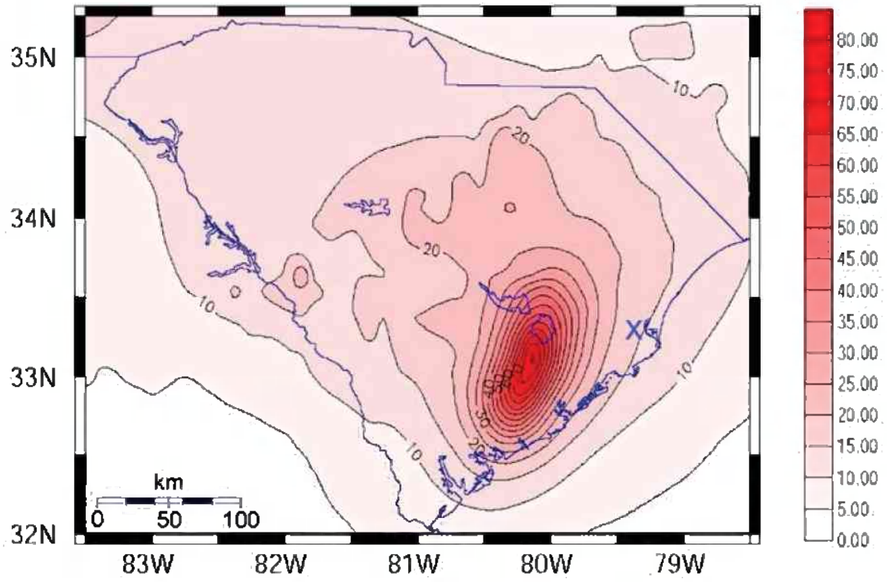

Figure 1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Chapman and Talwani, 2006)

Note:

1. PGA for WGS was selected as 0.16g.

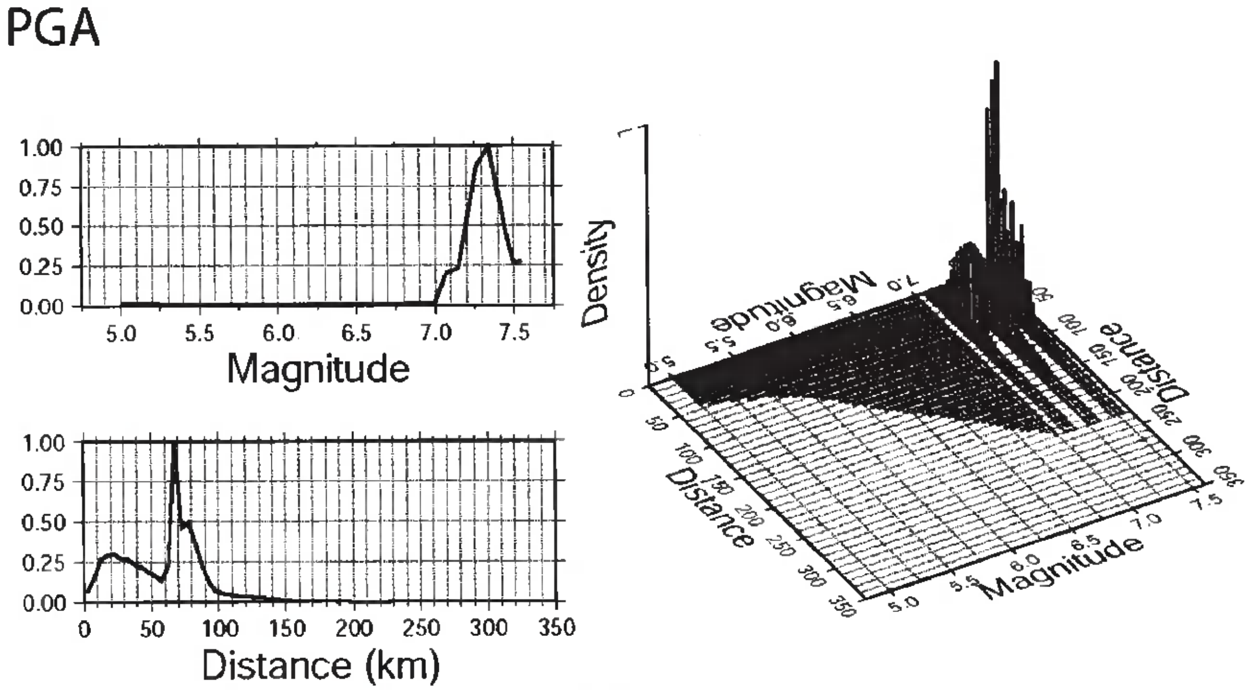

Figure 2. Deaggregation of 2 Percent Probability of Exceedance in 50 Years for PGA near Georgetown, South Carolina

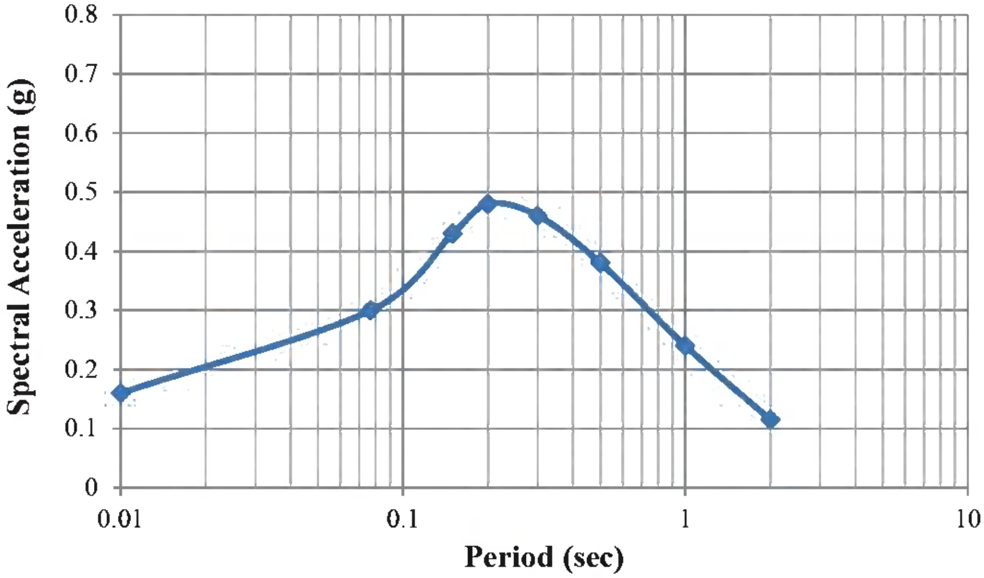

Figure 3. “Geologically Realistic” Target Response Spectrum for WGS

Notes:

1. Target response spectrum shown for “geologically realistic” conditions was developed from SCDOT (2010) seismic hazard maps (see Appendix 2).
2. The target spectrum for “geologically realistic” conditions is selected for seismic evaluations.

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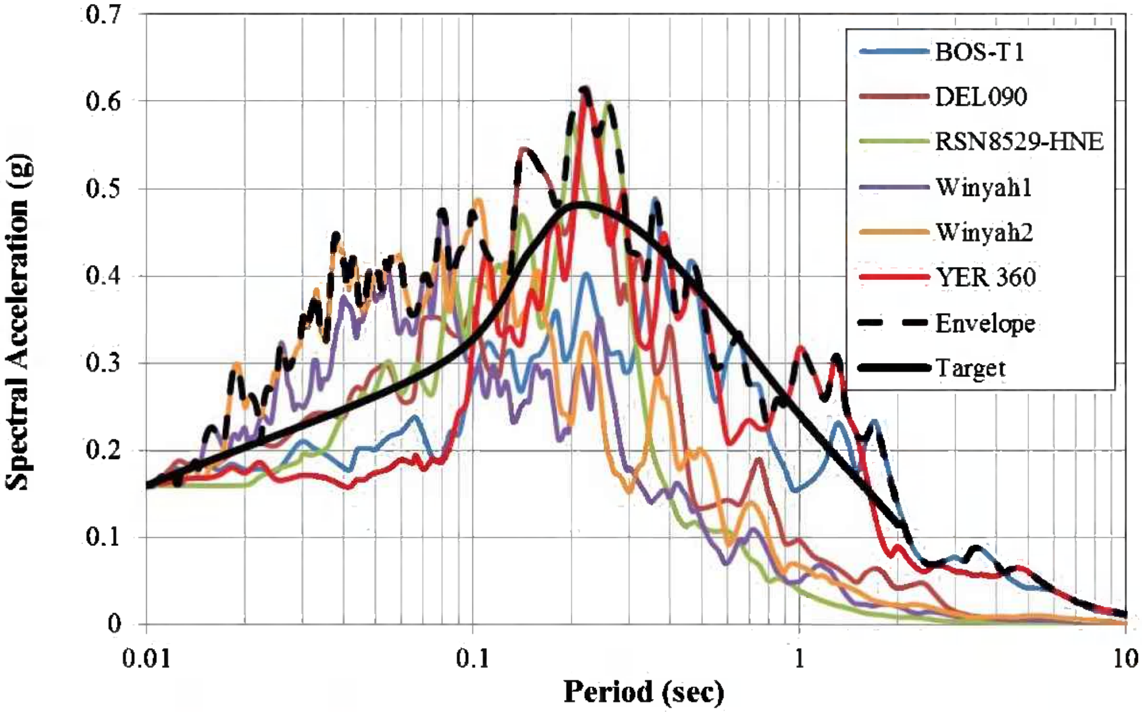

Figure 4. Response Spectra of Scaled Time Histories Selected for Seismic Evaluations

Note:

1. Time histories were scaled to match the target PGA = 0.16g (represented by a period of 0.01 seconds).

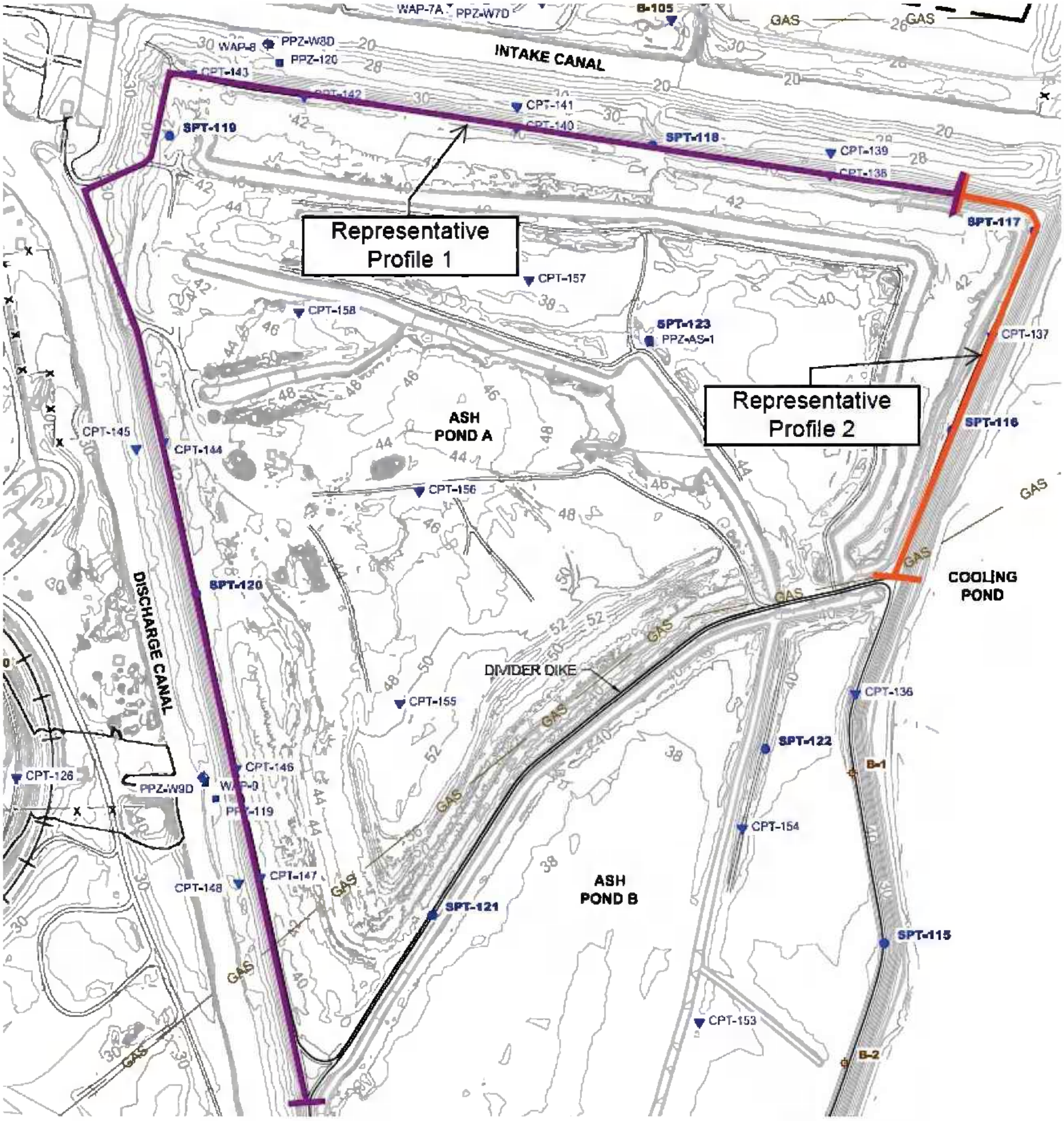

Figure 5. Locations of Representative Soil Profiles

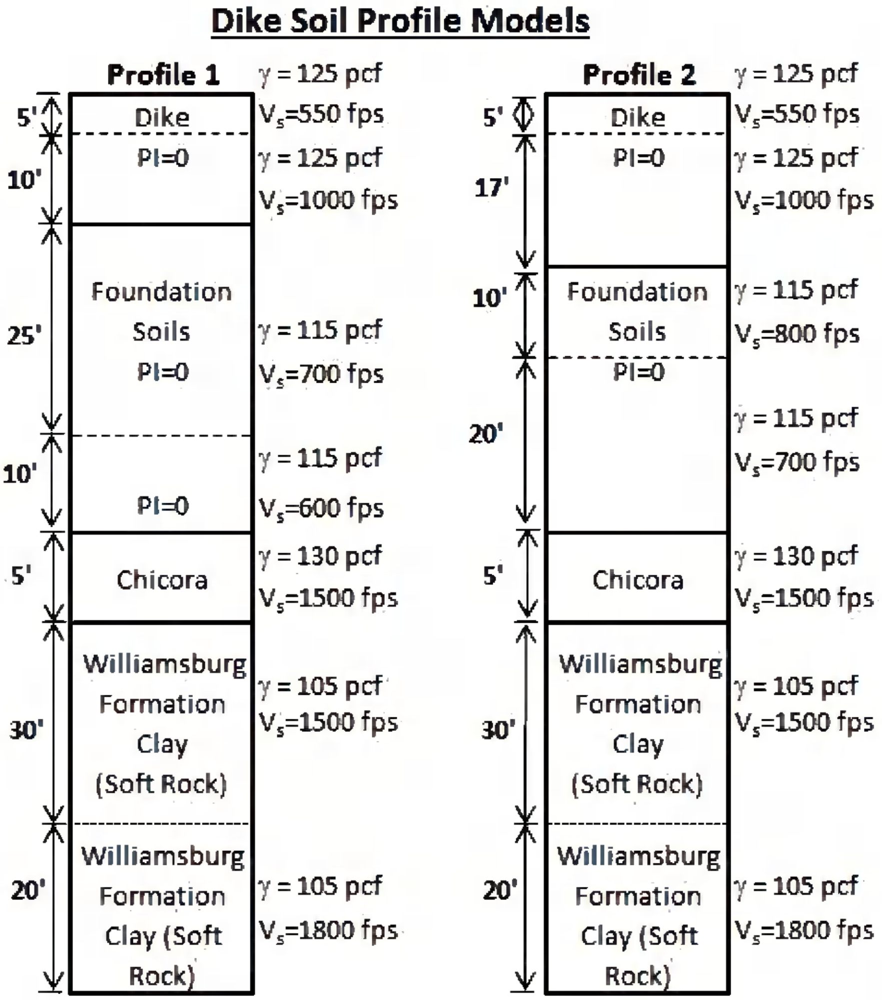

Figure 6. DEEPSOIL® Soil Profile Models for the Two Representative Profiles

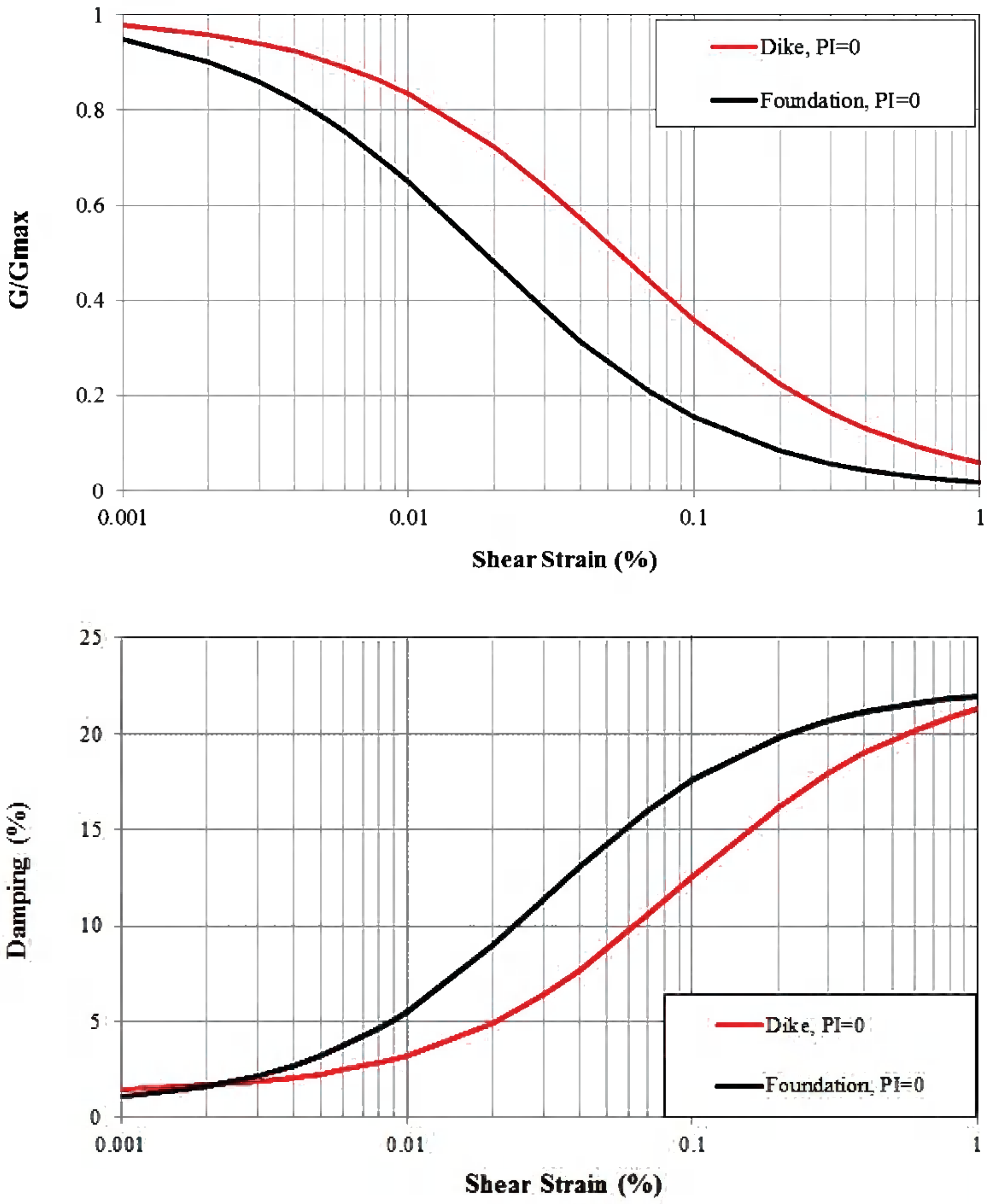

Figure 7. Shear Modulus Reduction (G/G_{max}) and Damping Curves for Soils Used in Site Response Analyses

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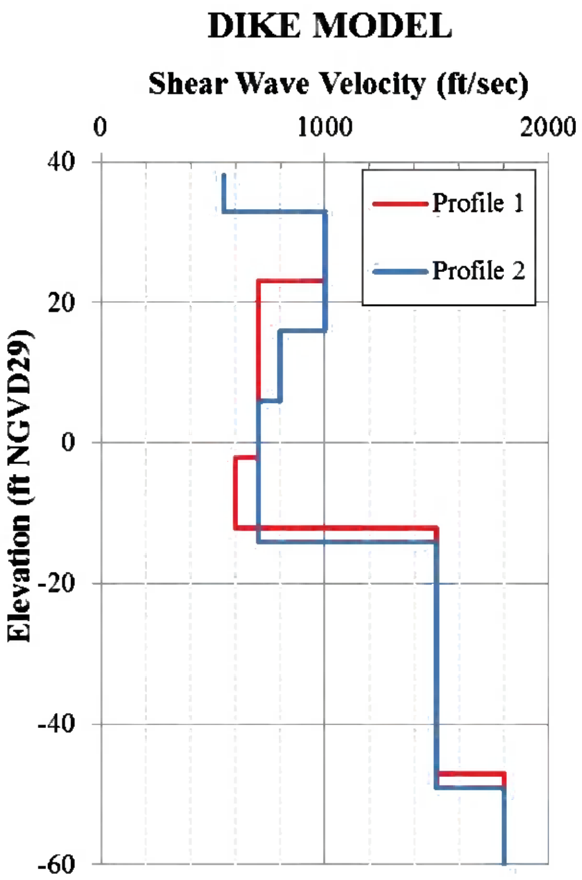

Figure 8. Selected Shear Wave Velocity (V_s) Profiles for Site Response Analyses

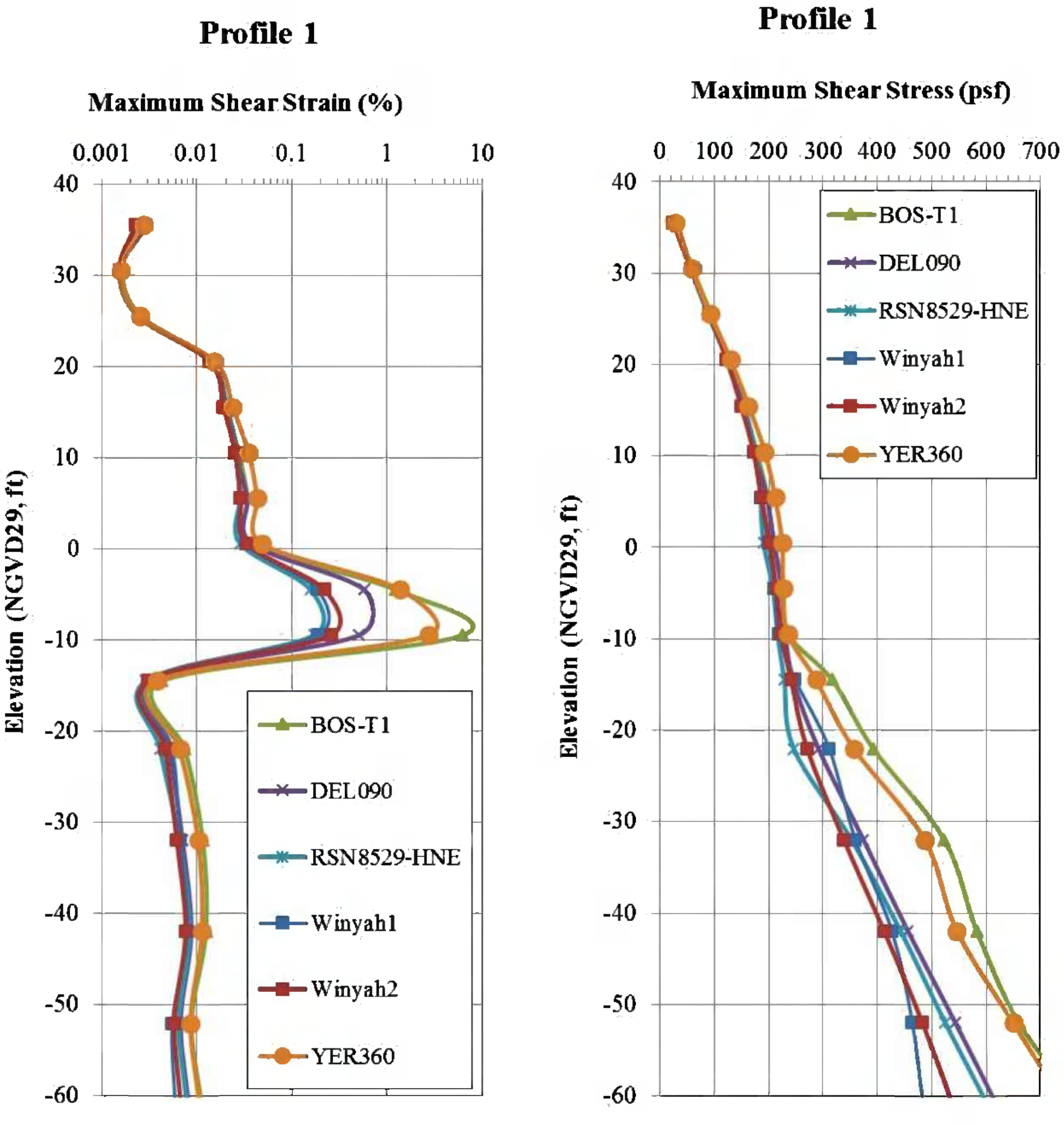

Figure 9a. Site Response Analysis Results for Profile 1

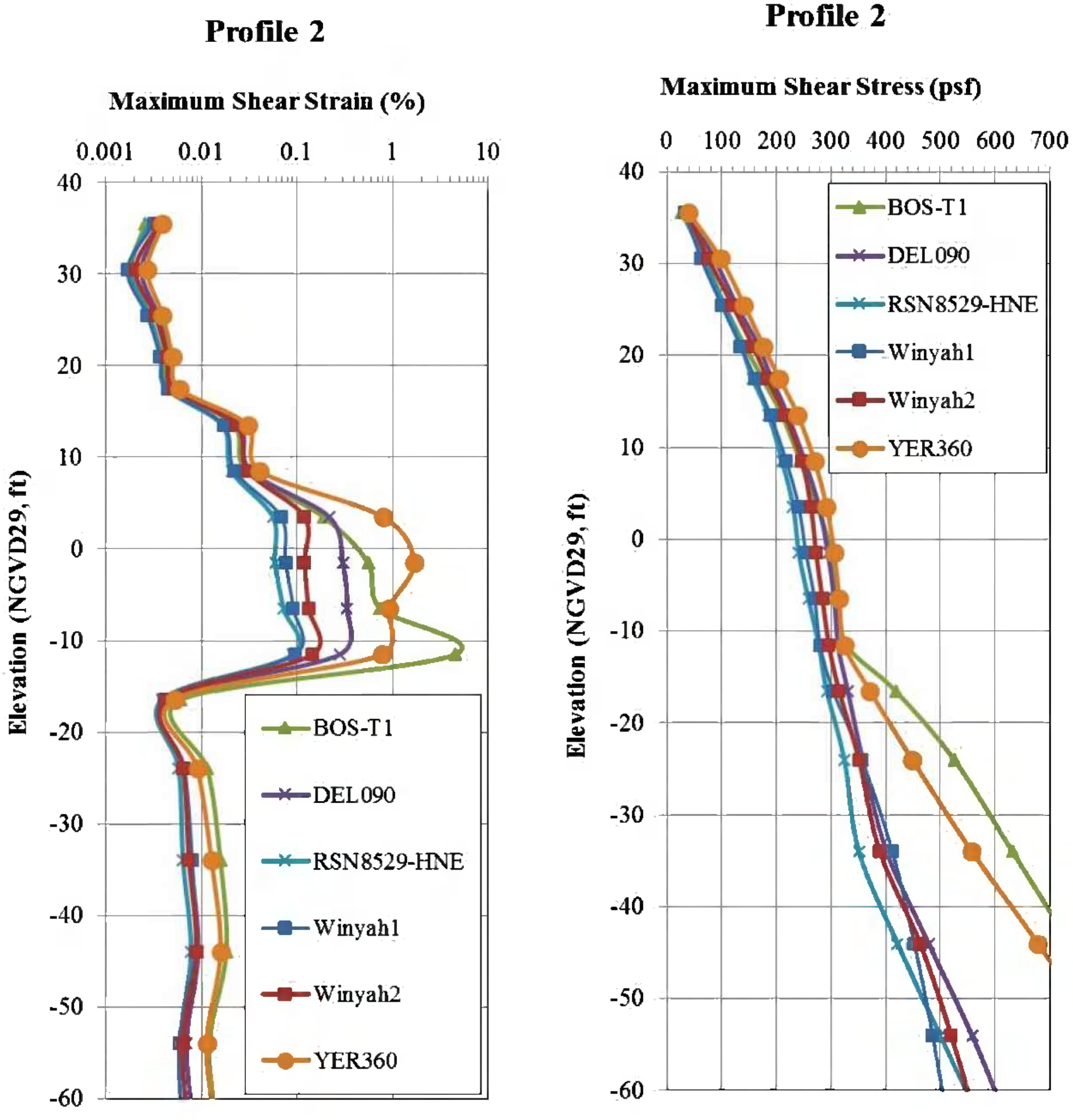

Figure 9b. Site Response Analysis Results for Profile 2

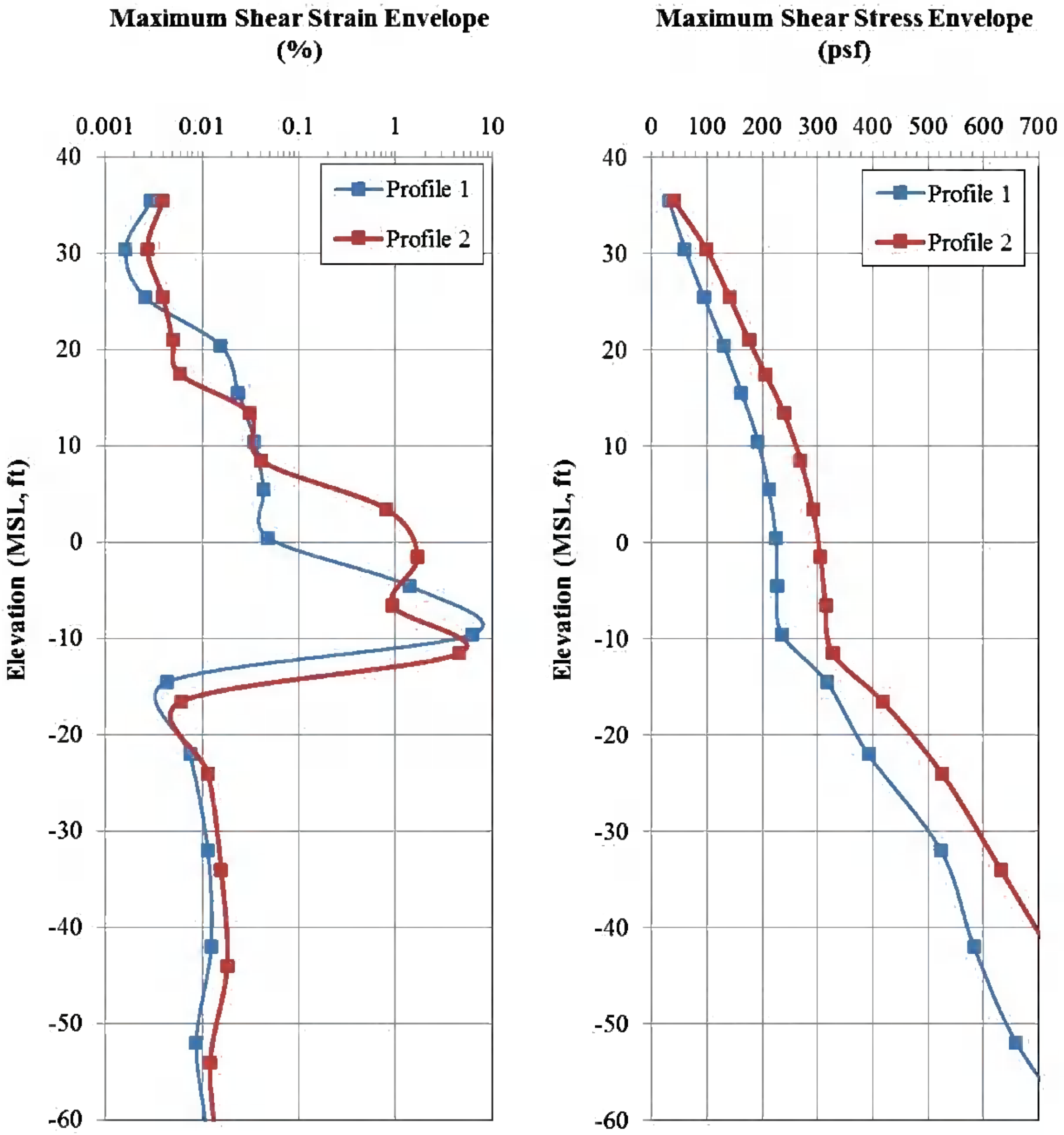

Figure 10. Maximum Shear Strain and Shear Stress Envelopes for Each Profile

Appendix 1

Peak Ground Accelerations from Different Seismic Hazard Maps

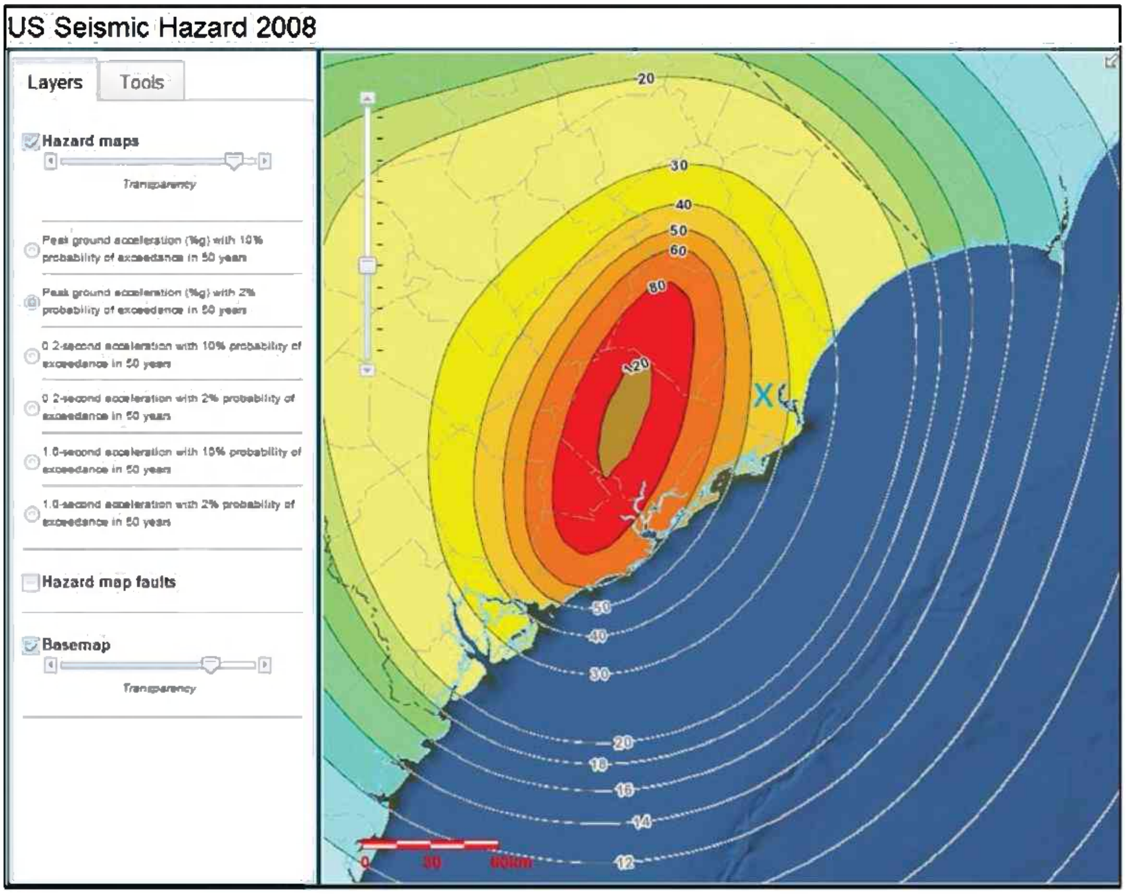

Figure 1-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years (USGS, 2008)

Note:

1. Site PGA based on USGS seismic hazard map (2008) is 0.469g.

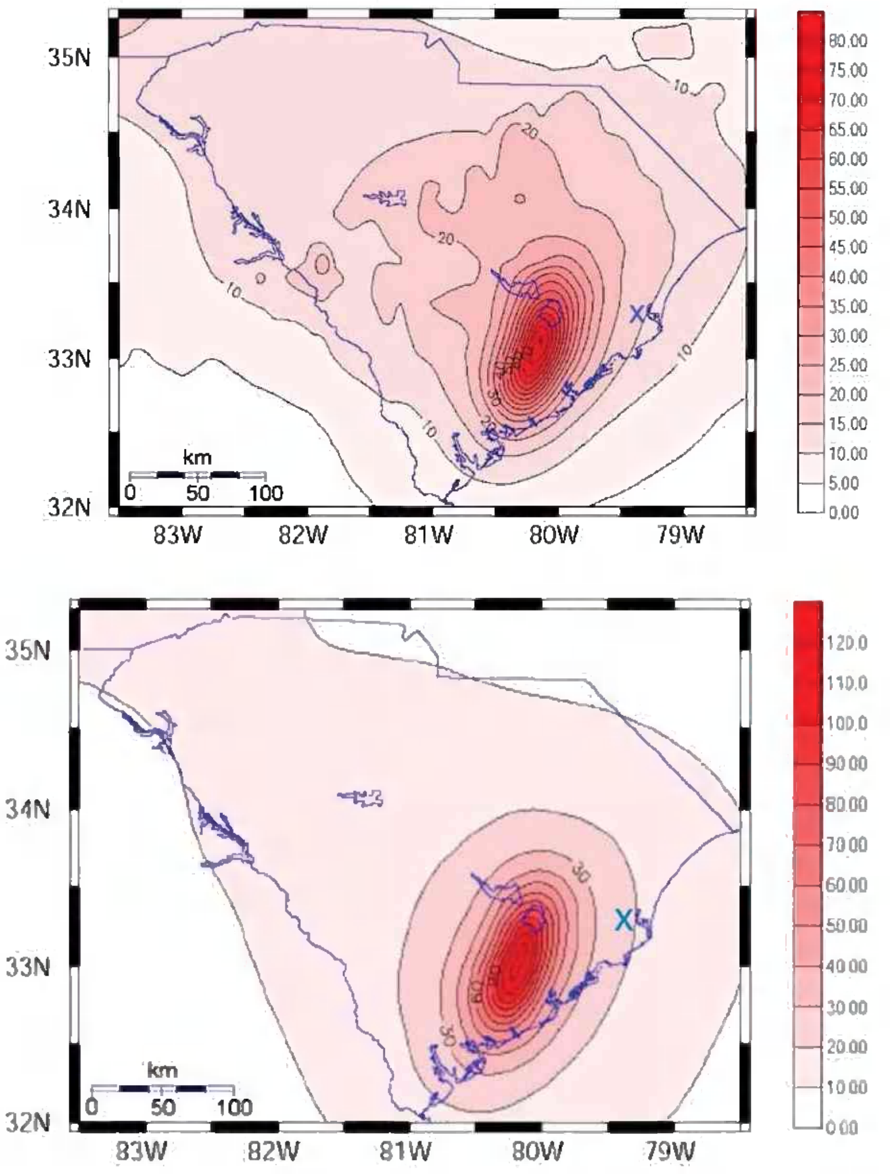

Figure 1-2. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Site PGA based on Chapman and Talwani (2006) is approximately 0.16g for "geologically realistic" conditions and 0.21g for hard rock conditions.

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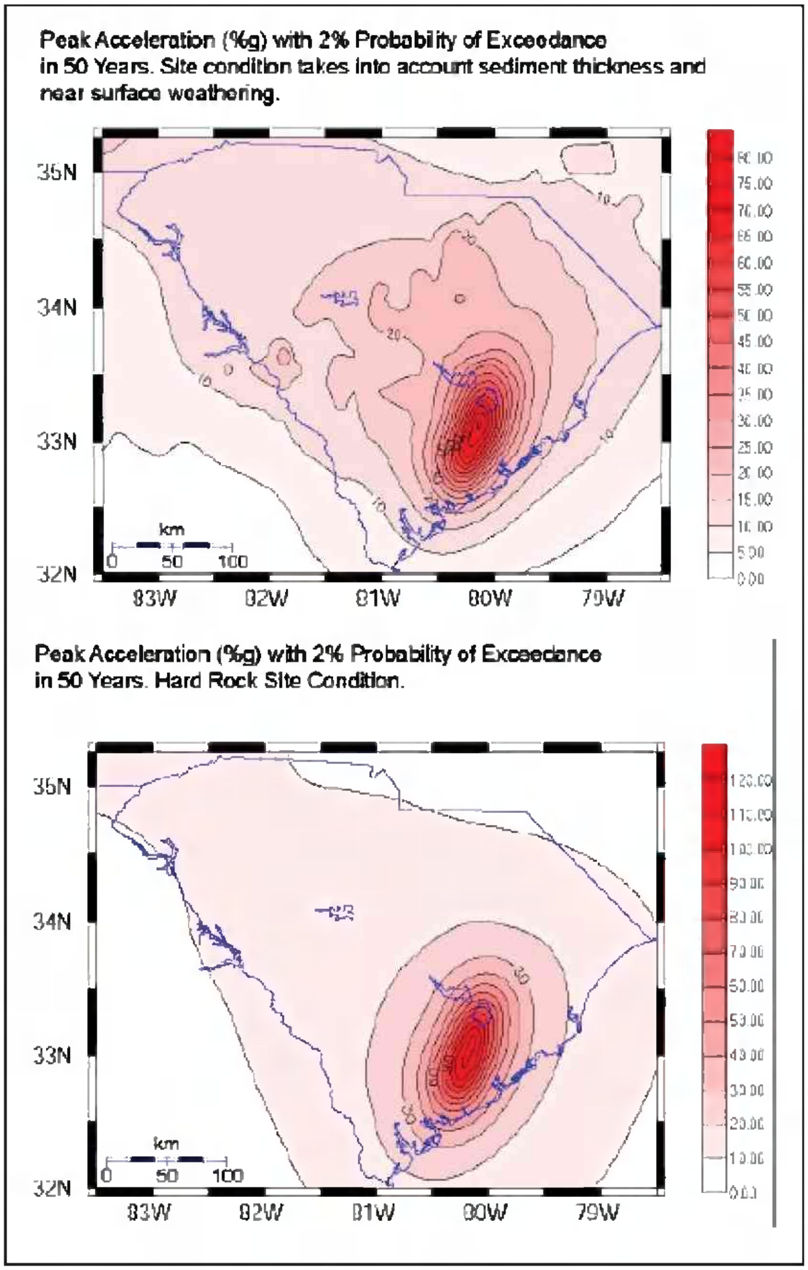
Appendix 2

SCDOT Seismic Hazard Maps Used for Development of Target Design Spectra



Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

Figure 2-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006).

Note:

1. Refer to the figures in Appendix 1 for the site location.

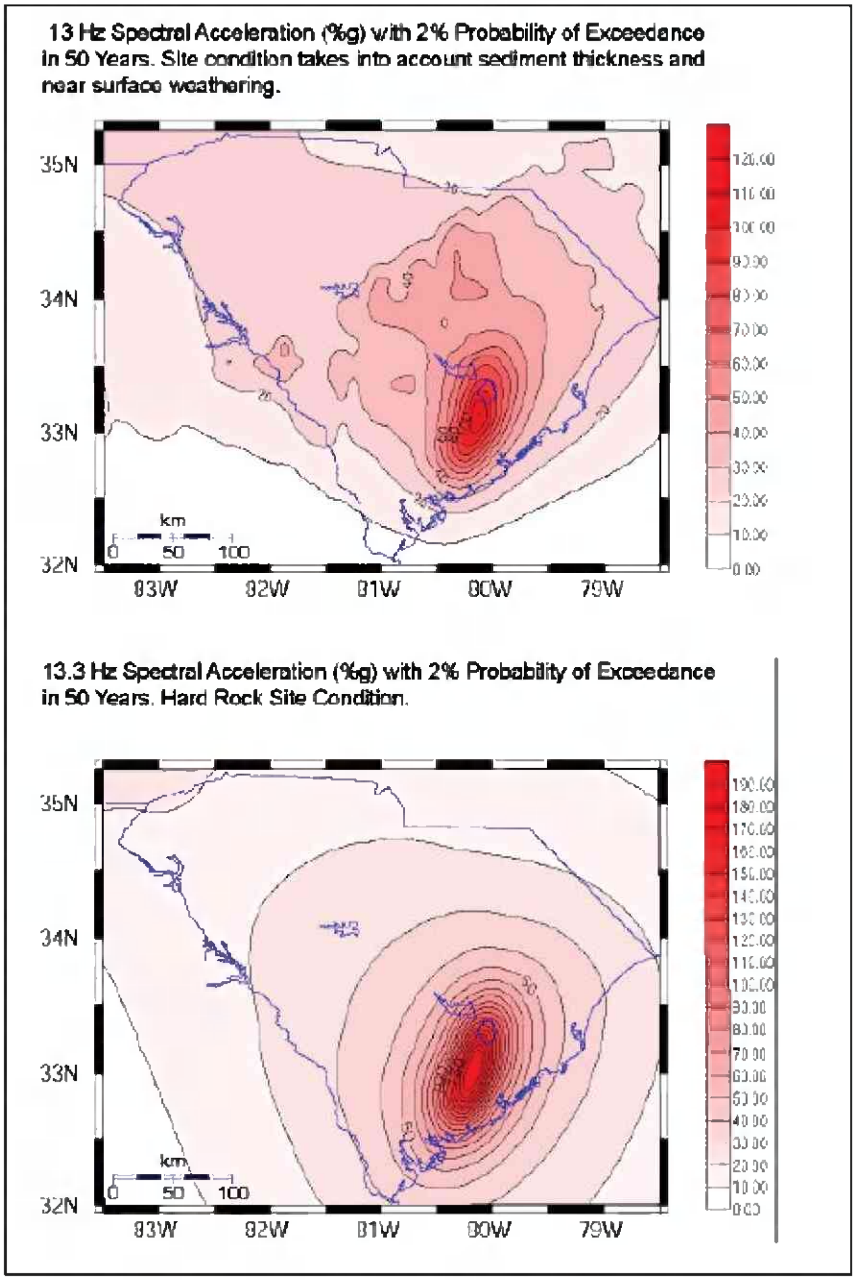
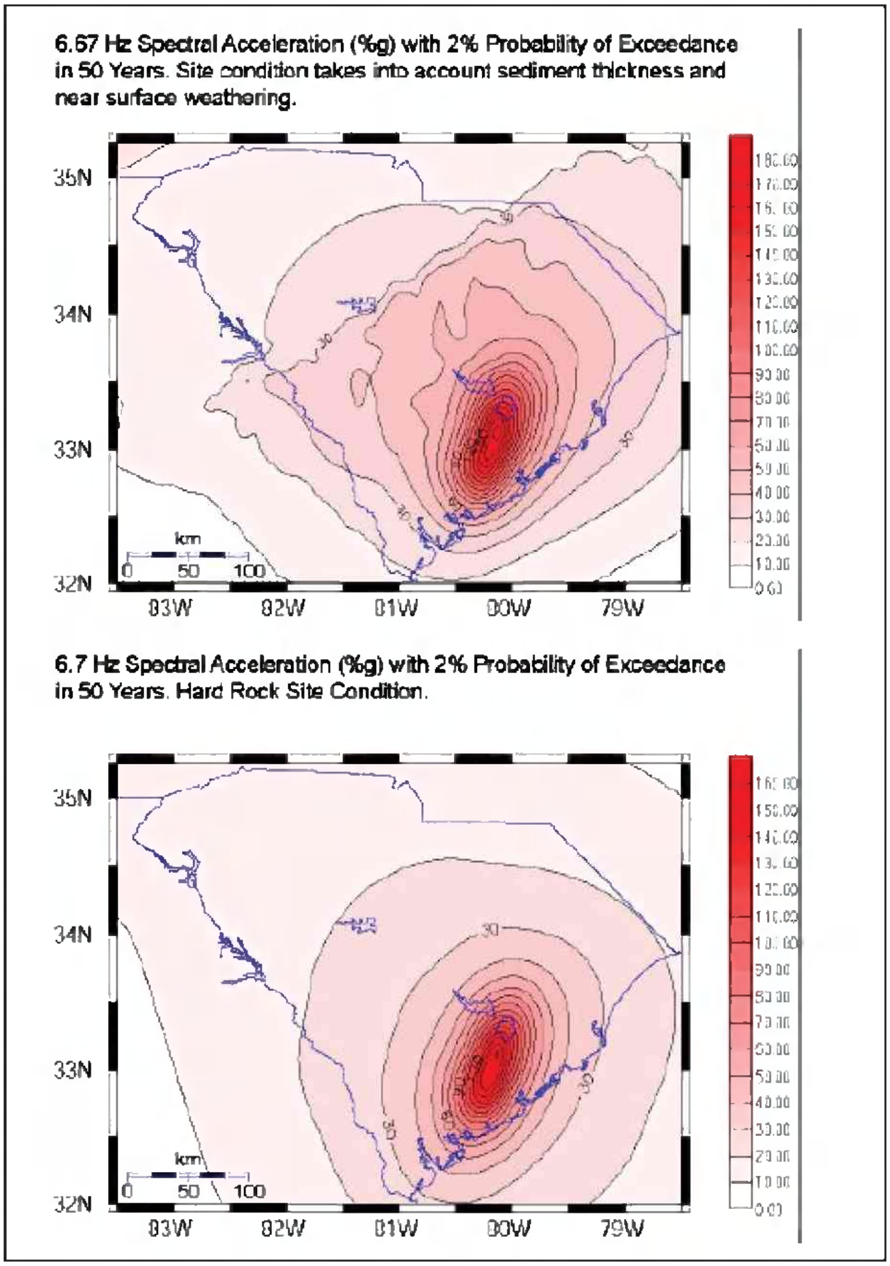


Figure 2-2. Spectral Acceleration (%) for 13 Hz (0.075 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.



6.67 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

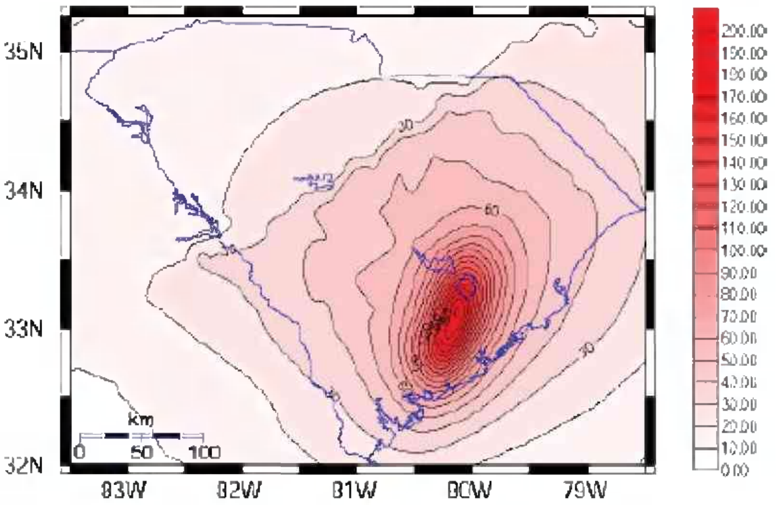
6.7 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

Figure 2-3. Spectral Acceleration (%) for 6.7 Hz (0.15 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

5 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.



5 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

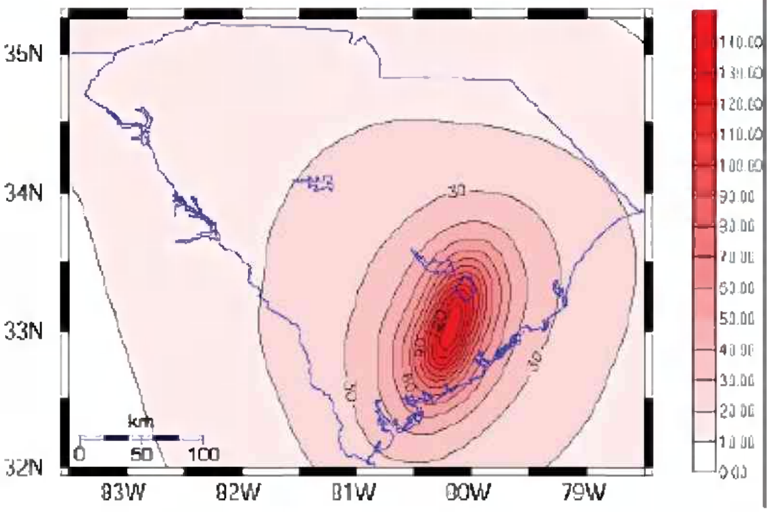
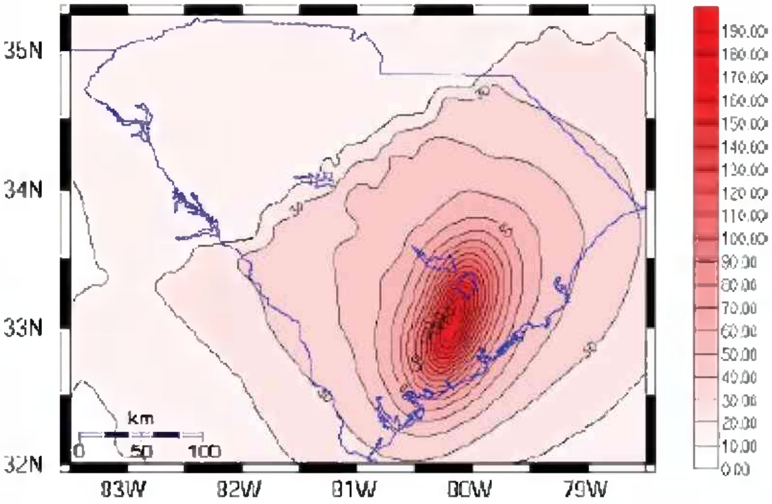


Figure 2-4. Spectral Acceleration (%) for 5 Hz (0.2 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (upper figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

3.33 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.



3.33 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

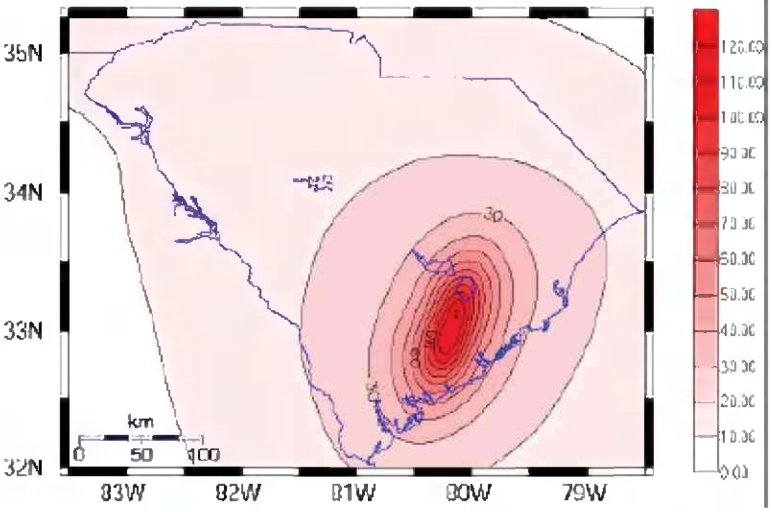
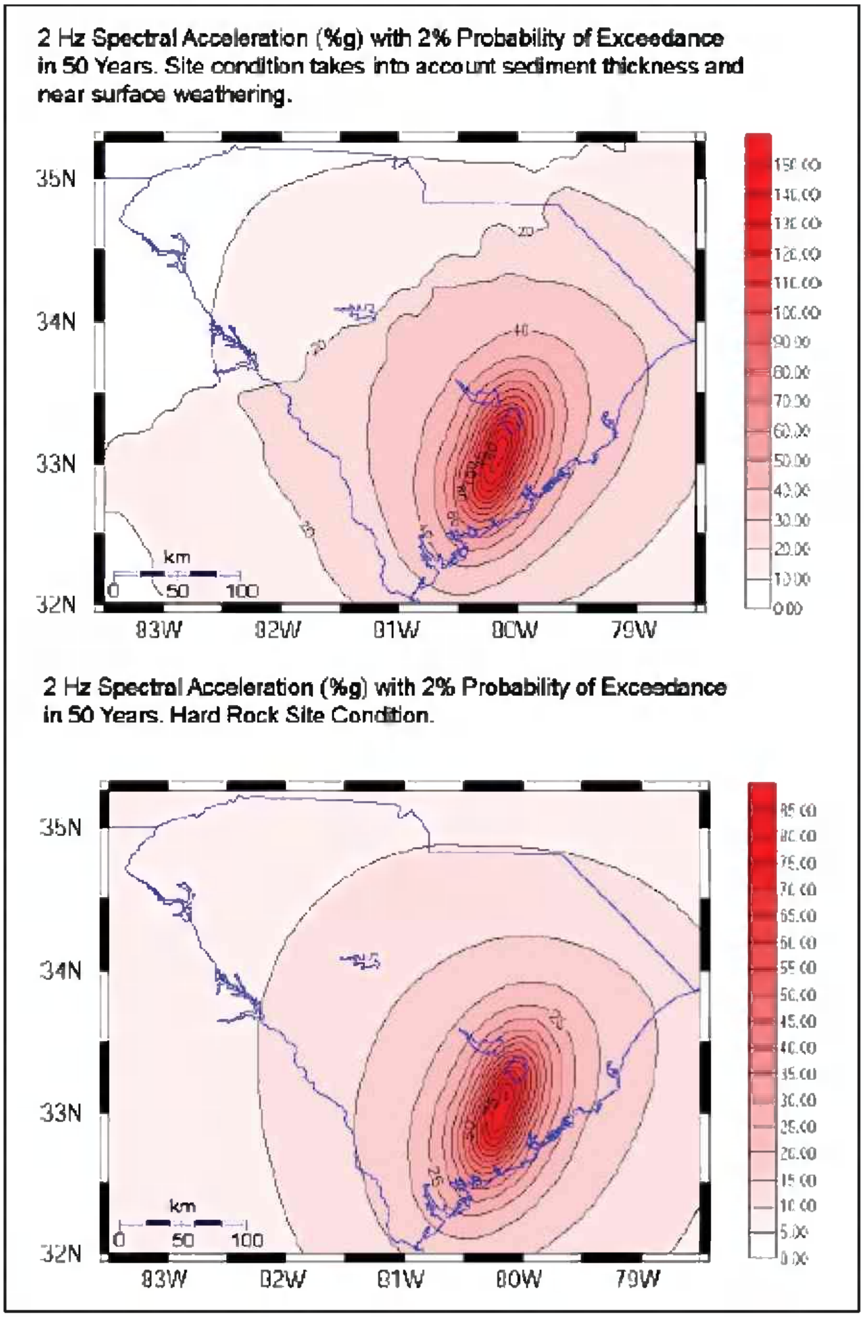


Figure 2-5. Spectral Acceleration (%) for 3.33 Hz (0.3 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.



2 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

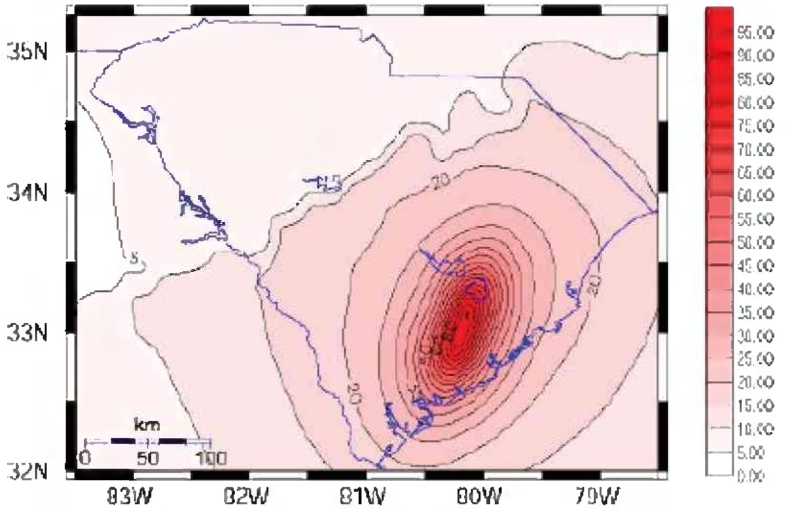
2 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

Figure 2-6. Spectral Acceleration (%) for 2 Hz (0.5 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

1 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.



1 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

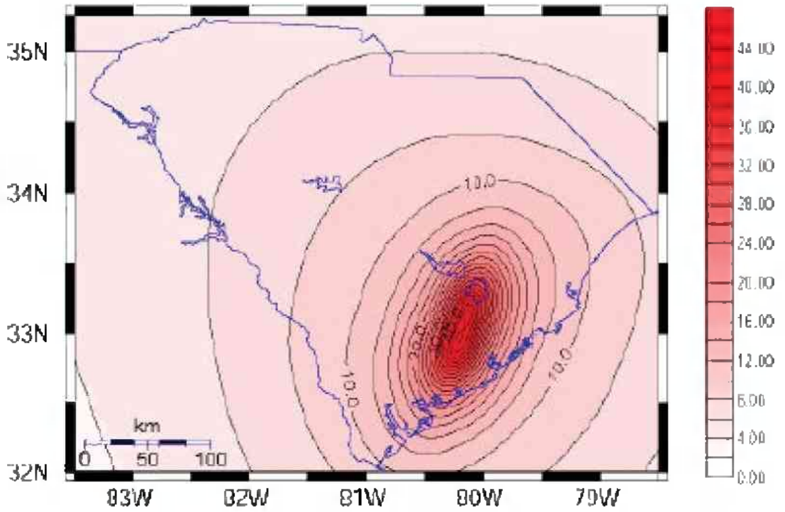
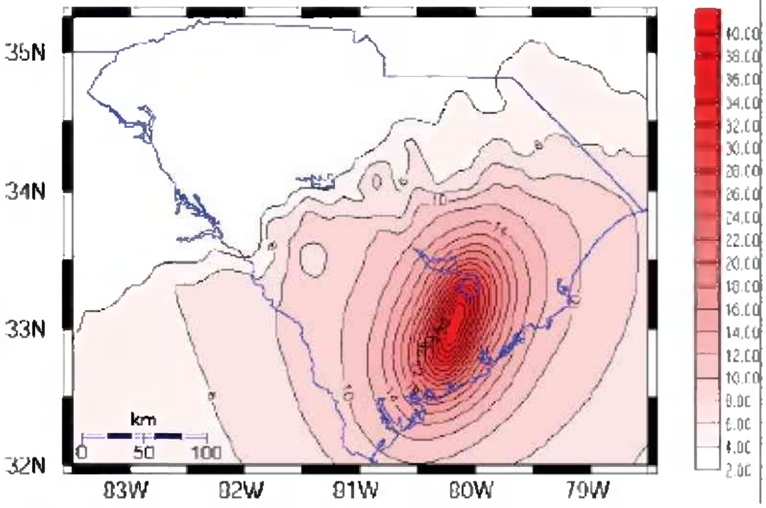


Figure 2-7. Spectral Acceleration (%) for 1 Hz (1 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

0.5 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.



0.5 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

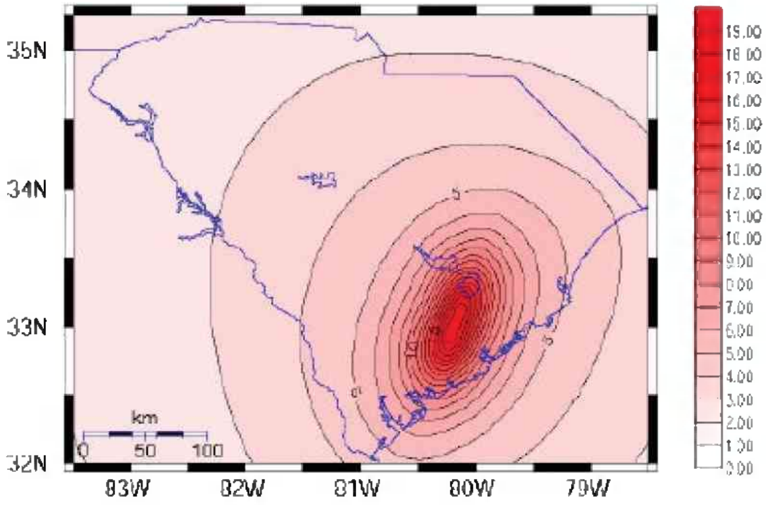


Figure 2-8. Spectral Acceleration (%) for 0.5 Hz (2 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

Appendix 3

Selected Time Histories

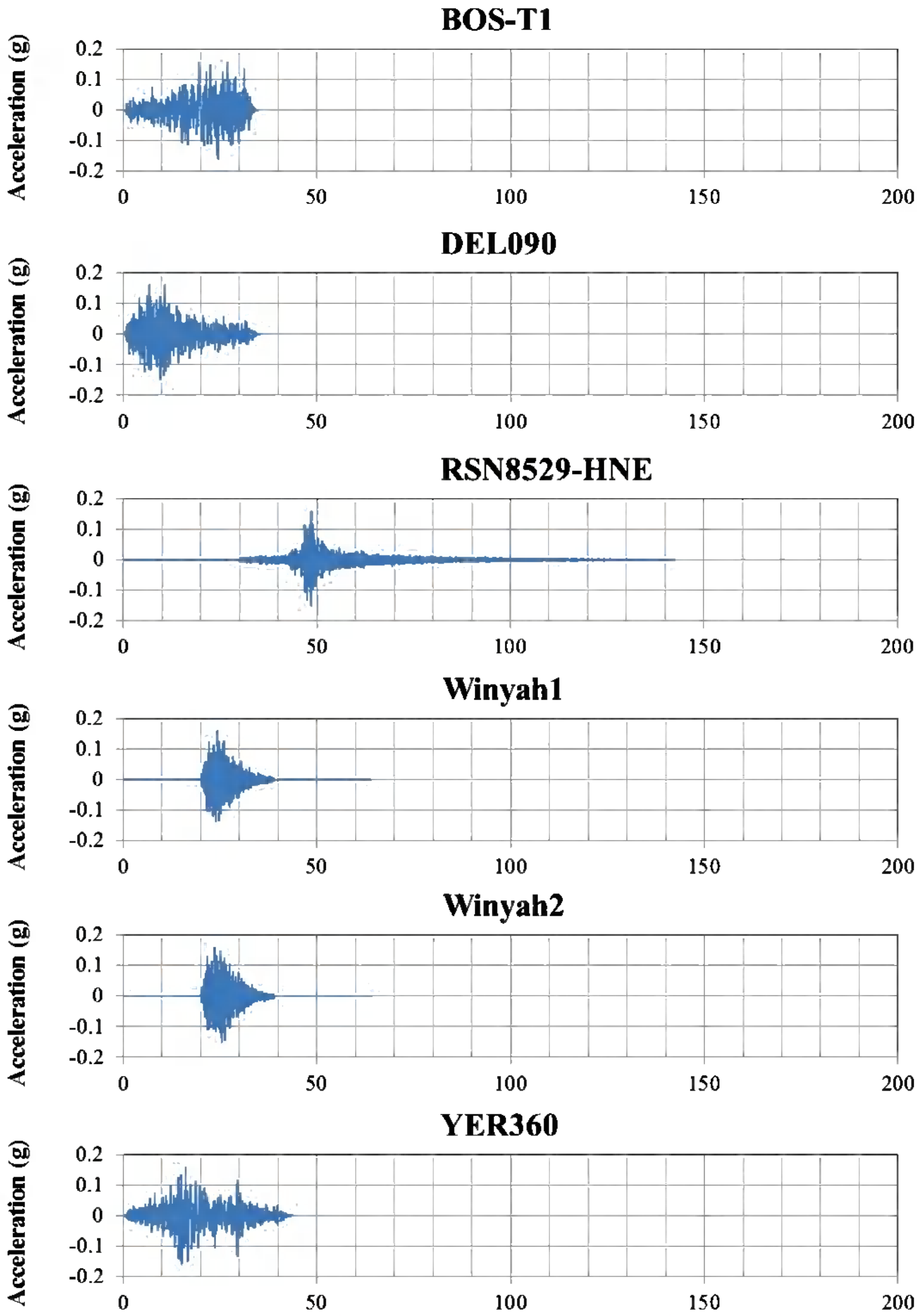


Figure 3-1. Acceleration Time Histories of Selected Earthquake Motions Scaled to PGA of 0.16g

Appendix 4

Shear Modulus Reduction and Damping Curve Selection

As indicated in the package, Geosyntec developed region-specific shear modulus reduction and damping curves based on the procedures presented in SCDOT GDM (2010). Figures 4-1 and 4-5 show the procedures. An example calculation following these procedures is presented as follows.

Shear Modulus Reduction Curve for the upper native soil in Profile 2

(see Figure 4-1 for description on each step; see Figure 4-2 for the profile)

Step 1 – age of the soil layer: Pleistocene deposit.

Step 2 – soil type: sandy soils with PI=0; groundwater table @ 15 ft bgs.

Step 3 – calculate σ_m'@ mid-depth of the layer (37 ft bgs)

$\sigma_v' = \gamma H - \gamma_w H_w = 125\times22 + 115\times15 - 62.4\times22 = 3102.2$ psf

$\sigma_m' = \sigma_v' (1+2K'_o)/3 = 3102.2\times(1+2\times0.47)/3 = 2006.1$ psf

($K_o' = 1- \sin\phi' = 1- \sin(32) = 0.47$, see Figure 5-3 for the equation)

Step 4 – σ_m' for the upper and lower native soils are within ±50% σ_m' value calculated above. The modulus reduction curve developed here can be used for the entire native soils in Profile 2.

Step 5 – select the parameters α, γ_{r1}, k from Figure 4-4.

$\gamma_{r1} = 0.018\%$, $\alpha = 1.00$, $k = 0.454$

Step 6 – compute the reference strain using SCDOT GDM Equation 12-20 (see Figure 4-3 for the equation).

$\gamma_r = \gamma_{r1} (\sigma_m'/P_a)^k = 0.018\times(2006.1/2089)^{0.454} = 0.0177\%$

Step 7 – compute shear modulus reduction curve using SCDOT GDM Equation 12-19 (see Figure 4-3 for the equation)

$$\frac{G}{G_{max}} = \frac{1}{1+(\frac{\gamma}{\gamma_r})^{\alpha}}$$

If $\gamma = 0.001\%$, $G/G_{max} = 1/[1+(0.001/0.0177)] = 0.947$
If $\gamma = 0.01\%$, $G/G_{max} = 1/[1+(0.01/0.0177)] = 0.639$
If $\gamma = 0.1\%$, $G/G_{max} = 1/[1+(0.1/0.0177)] = 0.150$

Damping Curve for the upper native soil in Profile 2

(see Figure 4-5 for description on each step; see Figure 4-2 for the profile)

Steps 1 through 4 are the same as those for modulus reduction curve development.

Step 5 – select small-strain material damping @ $\sigma_m' = 1$ atm, D_{min1} from Figure 4-6.

$D_{min1} = 0.59\%$

Step 6 – compute the small strain material damping, D_{min}, using SCDOT GDM Equation 12-28 (see Figure 4-7 for the equation).

$D_{min} = D_{min1}\ (\sigma_m'/P_a)^{-0.5k} = 0.59\times(2006.1/2089)^{-0.5\times0.454} = 0.595\%$

Step 7-9 – instead of taking Steps 7 through 9, use SCDOT GDM Equation 12-29 to compute damping ratio curve (D).

$D = 12.2\ (G/G_{max})^2 - 34.2\ (G/G_{max}) + 22.0 + D_{min}$

If $\gamma = 0.001\%$, $D = 12.2\times(0.947)^2 - 34.2\times(0.947) + 22.0 + 0.595 = 1.15\%$
If $\gamma = 0.01\%$, $D = 12.2\times(0.639)^2 - 34.2\times(0.639) + 22.0 + 0.595 = 5.72\%$
If $\gamma = 0.1\%$, $D = 12.2\times(0.150)^2 - 34.2\times(0.150) + 22.0 + 0.595 = 17.74\%$

Shear Modulus Reduction and Damping Curves for Chicora / Williamsburg Formation

Figure 4-8 presents shear modulus reduction and damping curves used for Pacific Engineering's site response analyses of the Ammonia tank building located at the WGS.

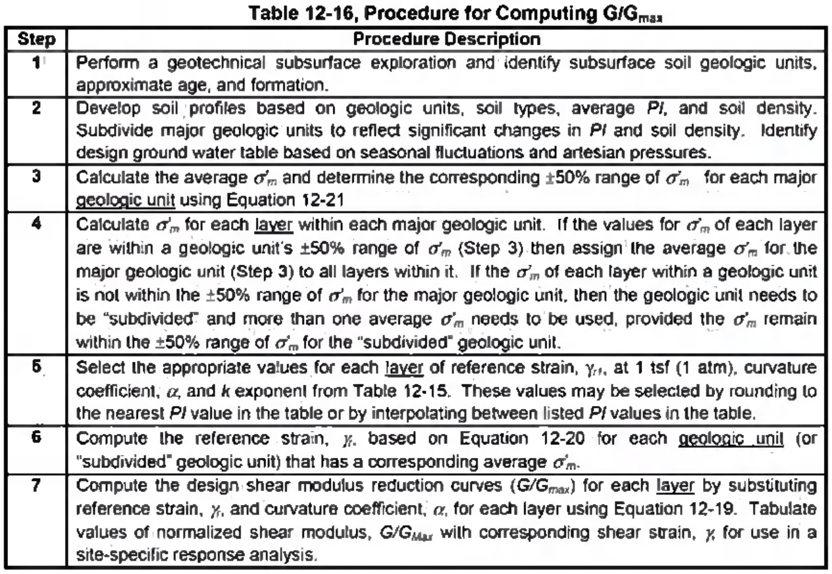


Table 12-16, Procedure for Computing G/G_{max}

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major <u>geologic unit</u> using Equation 12-21
4	Calculate σ'_m for each <u>layer</u> within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select the appropriate values for each <u>layer</u> of reference strain, γ_{r1}, at 1 tsf (1 atm), curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
6	Compute the reference strain, γ_r, based on Equation 12-20 for each <u>geologic unit</u> (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each <u>layer</u> by substituting reference strain, γ_r, and curvature coefficient, α, for each layer using Equation 12-19. Tabulate values of normalized shear modulus, G/G_{Max} with corresponding shear strain, γ for use in a site-specific response analysis.

Figure 4-1. Procedure for Development of Region-specific Modulus Reduction Curve (SCDOT, 2010)

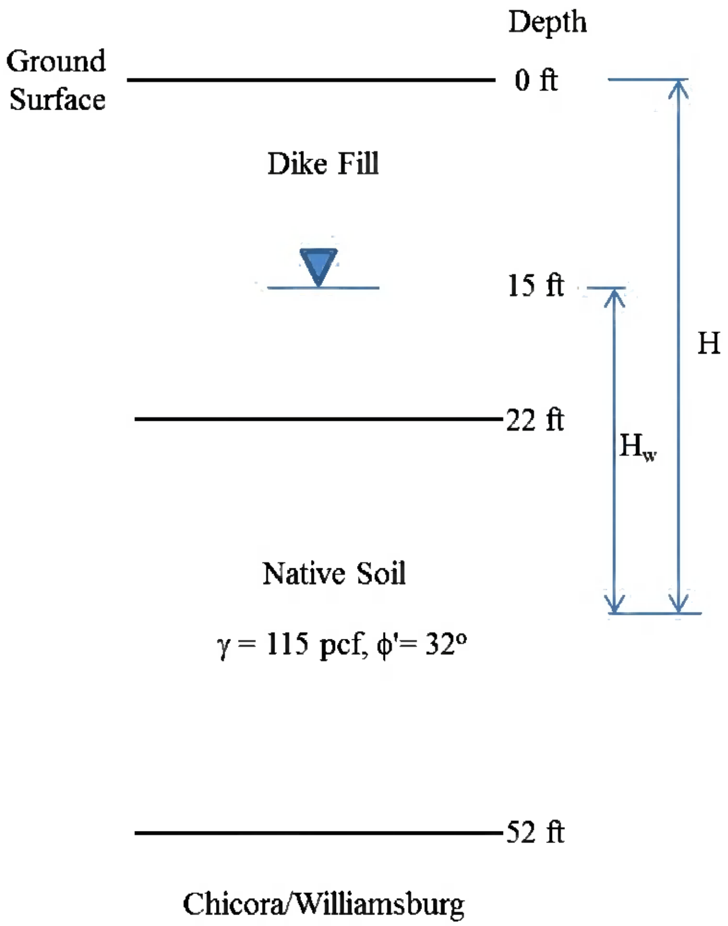
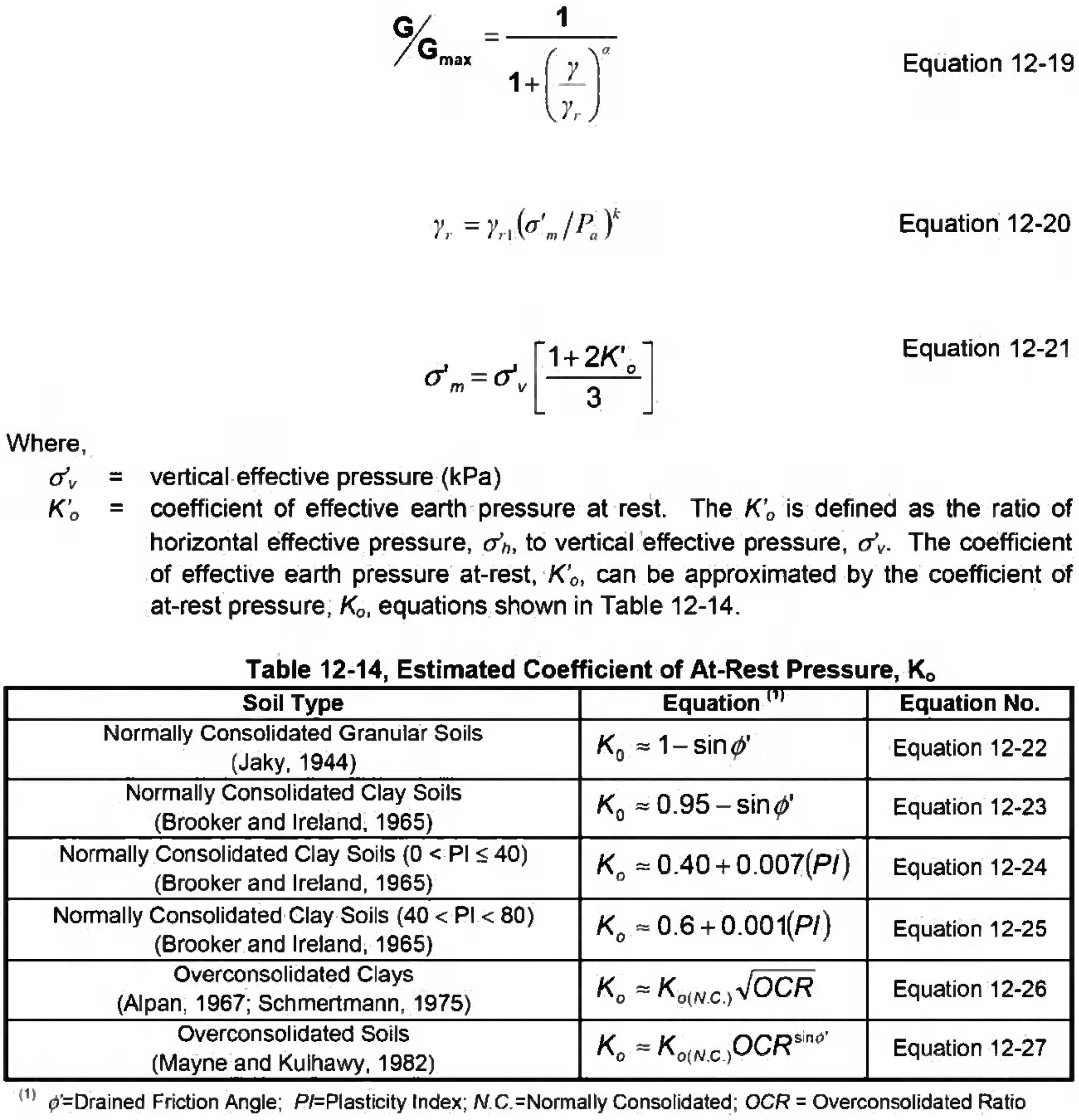


Figure 4-2. Profile 2 for the Example Calculations



$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}$$ Equation 12-19

$$\gamma_r = \gamma_{r1}\left(\sigma'_m / P_a\right)^k$$ Equation 12-20

$$\sigma'_m = \sigma'_v\left[\frac{1+2K'_o}{3}\right]$$ Equation 12-21

Where,

σ'_v = vertical effective pressure (kPa)

K'_o = coefficient of effective earth pressure at rest. The K'_o is defined as the ratio of horizontal effective pressure, σ'_h, to vertical effective pressure, σ'_v. The coefficient of effective earth pressure at-rest, K'_o, can be approximated by the coefficient of at-rest pressure, K_o, equations shown in Table 12-14.

Table 12-14, Estimated Coefficient of At-Rest Pressure, K_o

Soil Type	Equation (1)	Equation No.
Normally Consolidated Granular Soils (Jaky, 1944)	$K_0 \approx 1-\sin\phi'$	Equation 12-22
Normally Consolidated Clay Soils (Brooker and Ireland, 1965)	$K_0 \approx 0.95-\sin\phi'$	Equation 12-23
Normally Consolidated Clay Soils ($0 < PI \leq 40$) (Brooker and Ireland, 1965)	$K_o \approx 0.40+0.007(PI)$	Equation 12-24
Normally Consolidated Clay Soils ($40 < PI < 80$) (Brooker and Ireland, 1965)	$K_o \approx 0.6+0.001(PI)$	Equation 12-25
Overconsolidated Clays (Alpan, 1967; Schmertmann, 1975)	$K_o \approx K_{o(N.C.)}\sqrt{OCR}$	Equation 12-26
Overconsolidated Soils (Mayne and Kulhawy, 1982)	$K_o \approx K_{o(N.C.)}OCR^{\sin\phi'}$	Equation 12-27

(1) ϕ'=Drained Friction Angle; PI=Plasticity Index; $N.C.$=Normally Consolidated; OCR = Overconsolidated Ratio

Figure 4-3. Equations Needed for Shear Modulus Reduction Curve Development (SCDOT, 2010)

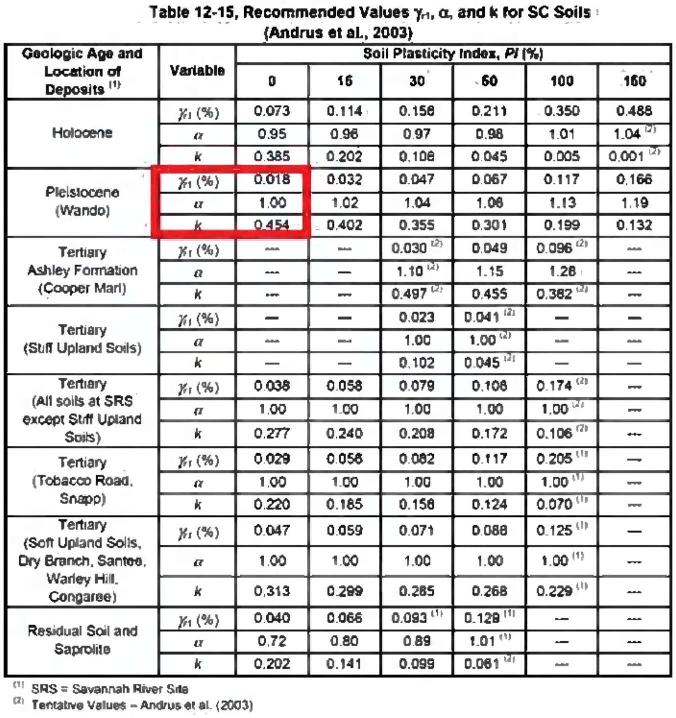


Table 12-15, Recommended Values γ_{r1}, α, and k for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits (1)	Variable	Soil Plasticity Index, *PI* (%)					
		0	16	30	60	100	160
Holocene	γ_{r1} (%)	0.073	0.114	0.156	0.211	0.350	0.488
	α	0.95	0.96	0.97	0.98	1.01	1.04 (2)
	k	0.385	0.202	0.108	0.045	0.005	0.001 (2)
Pleistocene (Wando)	γ_{r1} (%)	0.018	0.032	0.047	0.067	0.117	0.166
	α	1.00	1.02	1.04	1.06	1.13	1.19
	k	0.454	0.402	0.355	0.301	0.199	0.132
Tertiary Ashley Formation (Cooper Marl)	γ_{r1} (%)	—	—	0.030 (2)	0.049	0.096 (2)	—
	α	—	—	1.10 (2)	1.15	1.28	—
	k	—	—	0.497 (2)	0.455	0.382 (2)	—
Tertiary (Stiff Upland Soils)	γ_{r1} (%)	—	—	0.023	0.041 (2)	—	—
	α	—	—	1.00	1.00 (2)	—	—
	k	—	—	0.102	0.045 (2)	—	—
Tertiary (All soils at SRS except Stiff Upland Soils)	γ_{r1} (%)	0.038	0.058	0.079	0.108	0.174 (2)	—
	α	1.00	1.00	1.00	1.00	1.00 (2)	—
	k	0.277	0.240	0.208	0.172	0.106 (2)	—
Tertiary (Tobacco Road, Snapp)	γ_{r1} (%)	0.029	0.056	0.082	0.117	0.205 (1)	—
	α	1.00	1.00	1.00	1.00	1.00 (1)	—
	k	0.220	0.165	0.156	0.124	0.070 (1)	—
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	γ_{r1} (%)	0.047	0.059	0.071	0.088	0.125 (1)	—
	α	1.00	1.00	1.00	1.00	1.00 (1)	—
	k	0.313	0.299	0.285	0.268	0.229 (1)	—
Residual Soil and Saprolite	γ_{r1} (%)	0.040	0.066	0.093 (1)	0.129 (1)	—	—
	α	0.72	0.80	0.89	1.01 (1)	—	—
	k	0.202	0.141	0.099	0.061 (2)	—	—

(1) SRS = Savannah River Site

(2) Tentative Values – Andrus et al. (2003)

Figure 4-4. Recommended Parameters for South Carolina Soils (Table 12-15 of SCDOT, 2010)

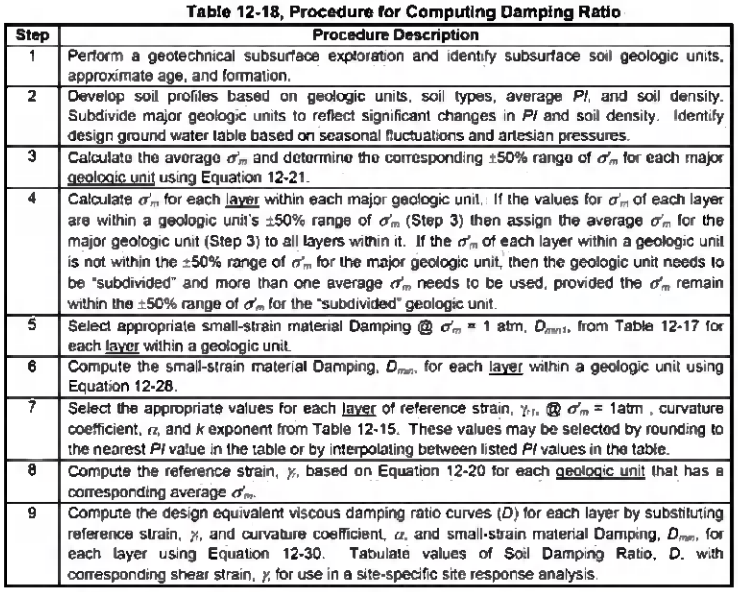


Table 12-18, Procedure for Computing Damping Ratio

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select appropriate small-strain material Damping @ σ'_m = 1 atm, D_{min1}, from Table 12-17 for each layer within a geologic unit.
6	Compute the small-strain material Damping, D_{min}, for each layer within a geologic unit using Equation 12-28.
7	Select the appropriate values for each layer of reference strain, γ_{r1}, @ σ'_m = 1atm , curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
8	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit that has a corresponding average σ'_m.
9	Compute the design equivalent viscous damping ratio curves (*D*) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, and small-strain material Damping, D_{min}, for each layer using Equation 12-30. Tabulate values of Soil Damping Ratio, *D*, with corresponding shear strain, γ, for use in a site-specific site response analysis.

Figure 4-5. Procedure for Development of Region-Specific Damping Curve (SCDOT, 2010)

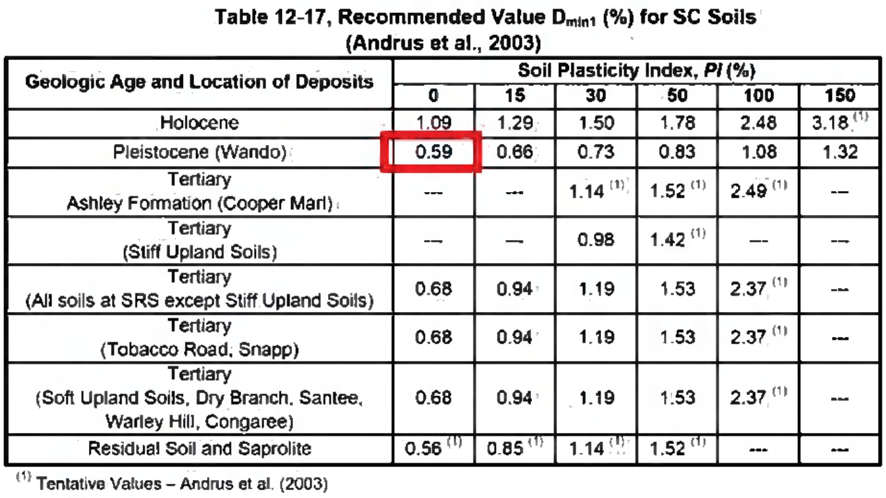
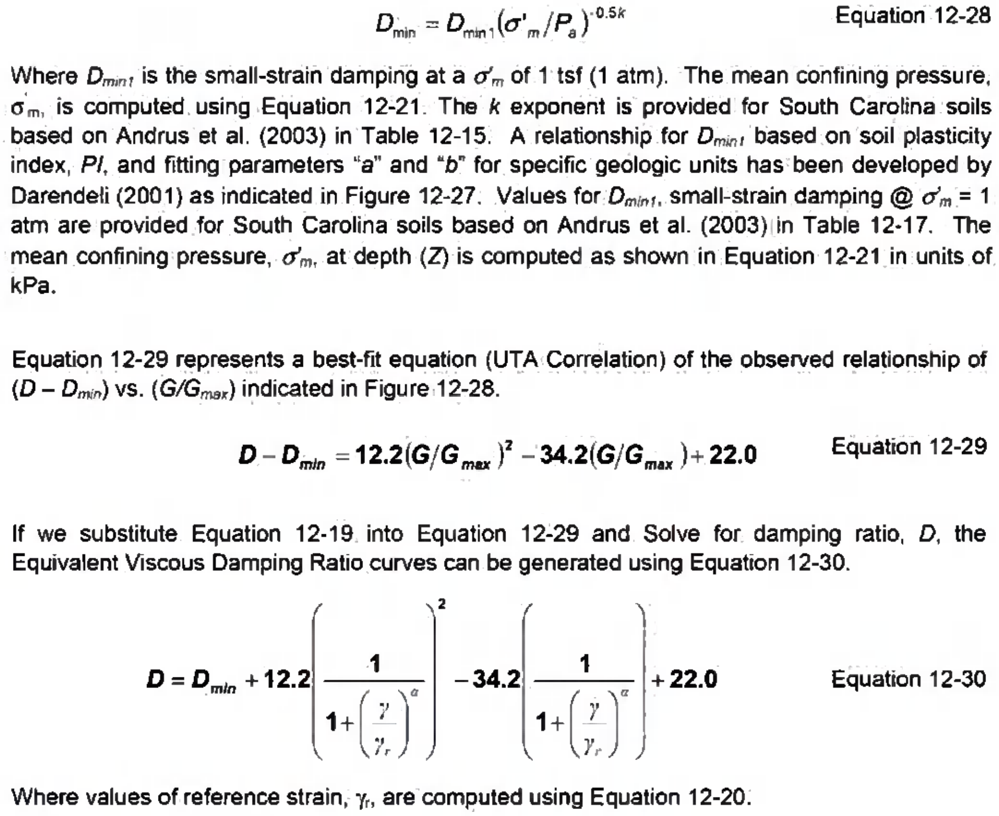


Table 12-17, Recommended Value D_{min1} (%) for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Plasticity Index, *PI* (%)					
	0	15	30	50	100	150
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 (1)
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	---	---	1.14 (1)	1.52 (1)	2.49 (1)	---
Tertiary (Stiff Upland Soils)	---	---	0.98	1.42 (1)	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Tobacco Road, Snapp)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	0.68	0.94	1.19	1.53	2.37 (1)	---
Residual Soil and Saprolite	0.56 (1)	0.85 (1)	1.14 (1)	1.52 (1)	---	---

(1) Tentative Values – Andrus et al. (2003)

Figure 4-6. Recommended D_{min1} for South Carolina Soils (Table 12-17 of SCDOT, 2010)



$$D_{\min} = D_{\min 1}(\sigma'_m/P_a)^{-0.5k}$$ Equation 12-28

Where D_{min1} is the small-strain damping at a σ'_m of 1 tsf (1 atm). The mean confining pressure, σ'_m, is computed using Equation 12-21. The k exponent is provided for South Carolina soils based on Andrus et al. (2003) in Table 12-15. A relationship for D_{min1} based on soil plasticity index, *PI*, and fitting parameters "*a*" and "*b*" for specific geologic units has been developed by Darendeli (2001) as indicated in Figure 12-27. Values for D_{min1}, small-strain damping @ $\sigma'_m = 1$ atm are provided for South Carolina soils based on Andrus et al. (2003) in Table 12-17. The mean confining pressure, σ'_m, at depth (Z) is computed as shown in Equation 12-21 in units of kPa.

Equation 12-29 represents a best-fit equation (UTA Correlation) of the observed relationship of ($D - D_{min}$) vs. (G/G_{max}) indicated in Figure 12-28.

$$D - D_{min} = 12.2(G/G_{max})^2 - 34.2(G/G_{max}) + 22.0$$ Equation 12-29

If we substitute Equation 12-19 into Equation 12-29 and Solve for damping ratio, D, the Equivalent Viscous Damping Ratio curves can be generated using Equation 12-30.

$$D = D_{min} + 12.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right)^2 - 34.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right) + 22.0$$ Equation 12-30

Where values of reference strain, γ_r, are computed using Equation 12-20.

Figure 4-7. Equations Needed for Damping Curve Development (SCDOT, 2010)

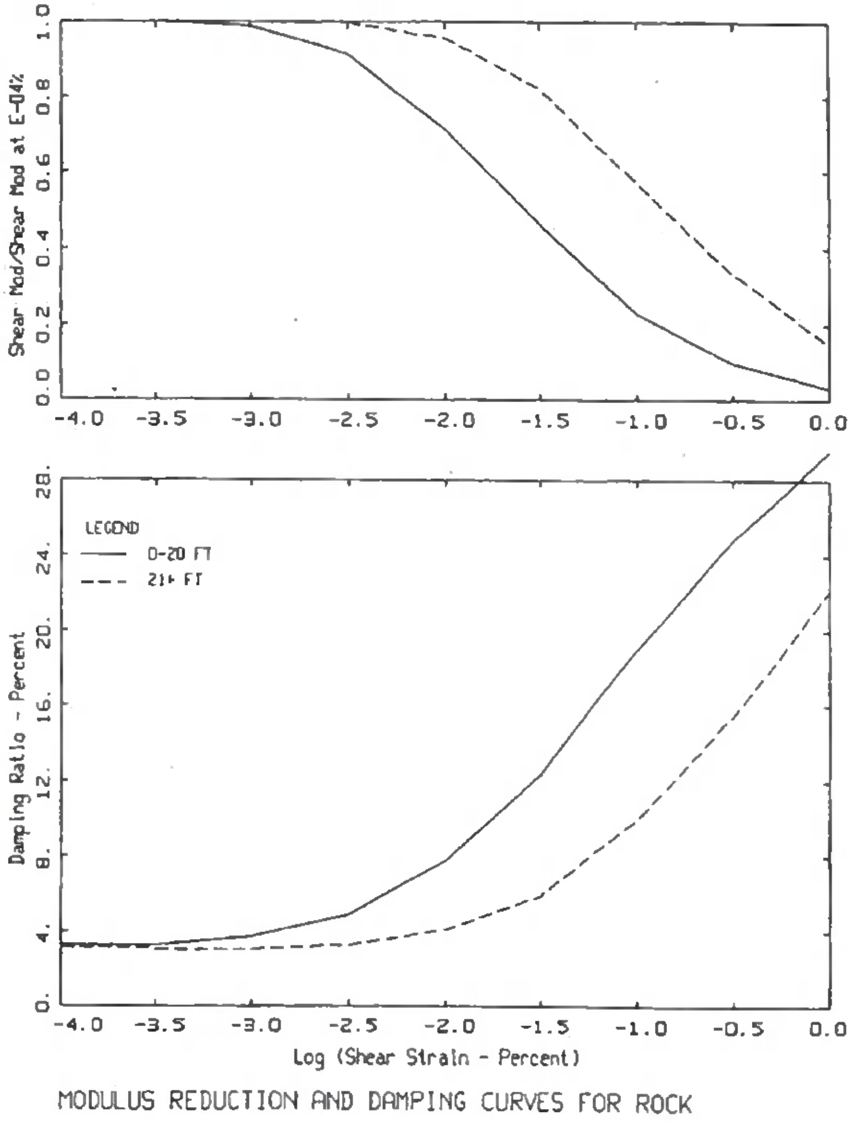


Figure 5b. Generic G/Gmax and hysteretic damping curves for soft rock (Silva et al., 1997).

Figure 4-8. Shear Modulus Reduction and Damping Curves for Chicora/Williamsburg Formation (S&ME, 2001)

Appendix 5

Shear Wave Velocity Profile Selection

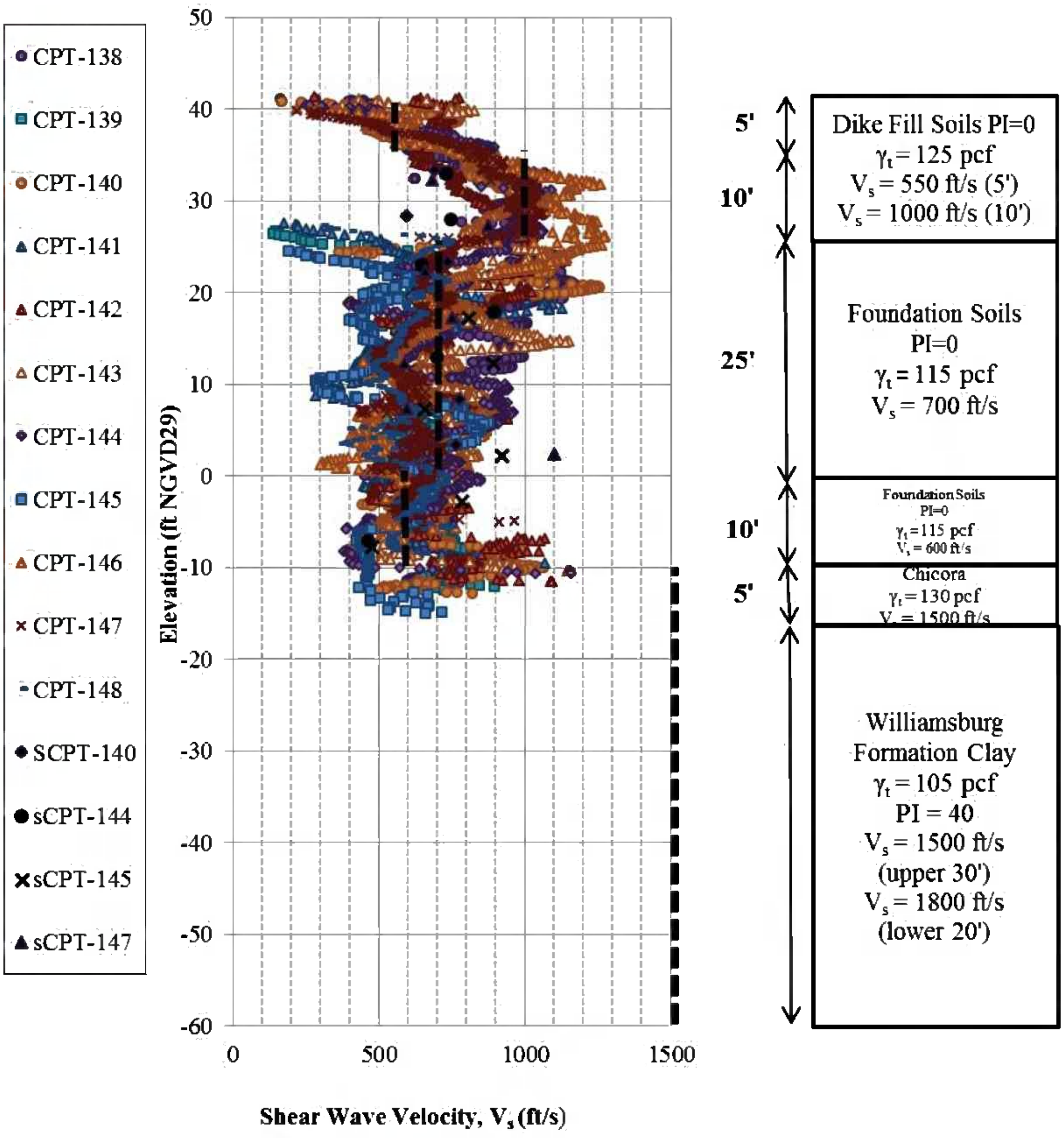


Figure 5-1a. Selected V_s Profile for the Intake/Discharge Canals Dike Model (Profile 1)

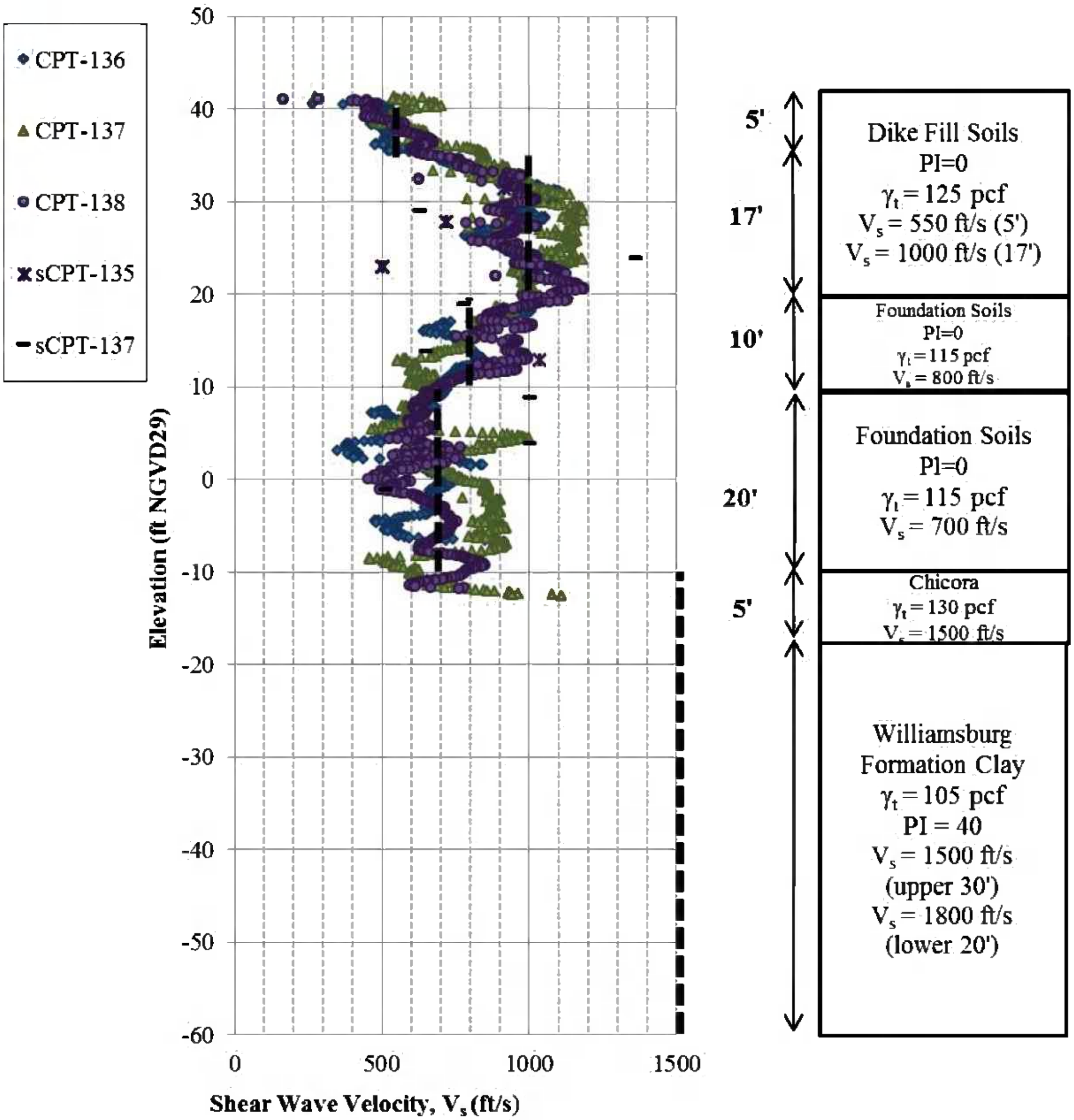


Figure 5-1b. Selected V_s Profile for the Cooling Pond Dike Model (Profile 2)

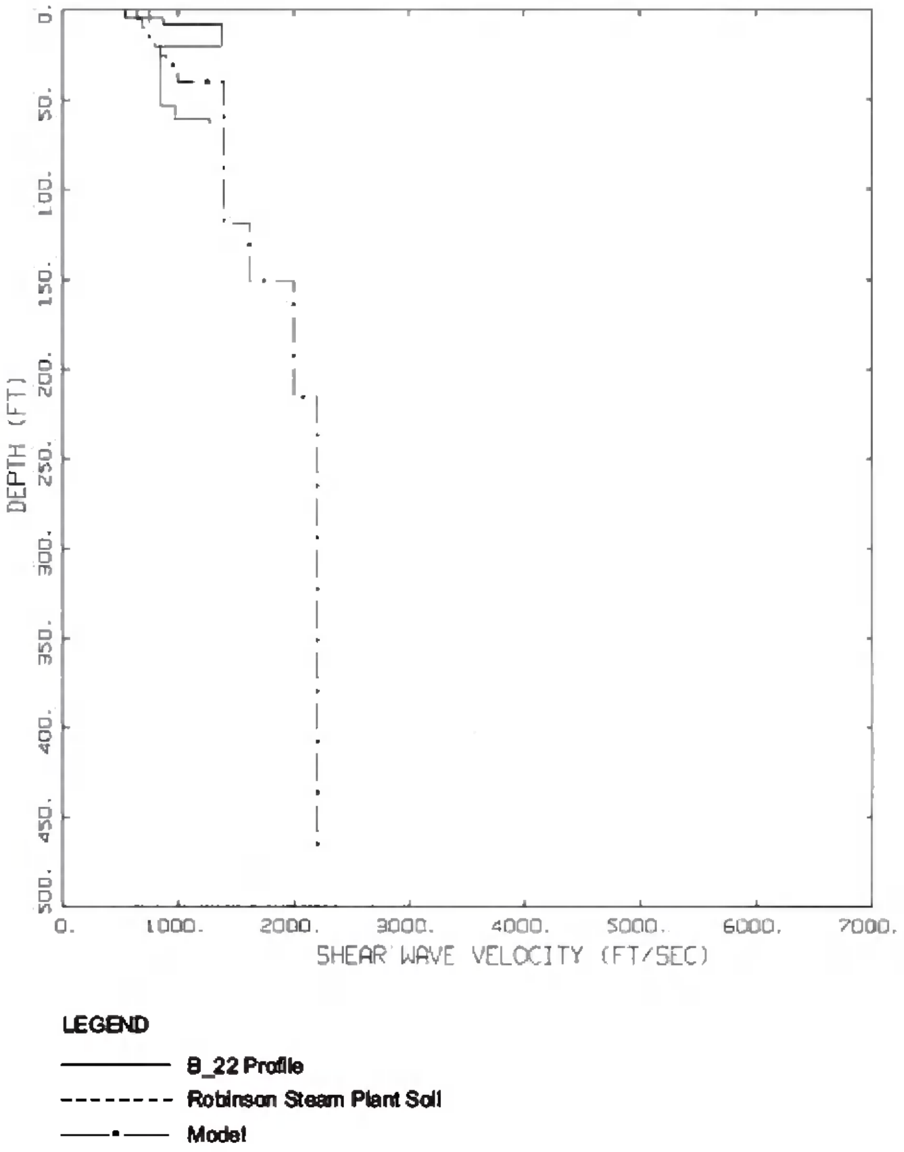


Figure 3-9. Base case shear-wave velocity profile for the Myrtle Beach site response category along with available profiles.

Figure 5-2. Reference V_s Profile for Chicora/Williamsburg Formation (URS, 2001)

Appendix 6

Calculated Acceleration Profiles

Profile 1

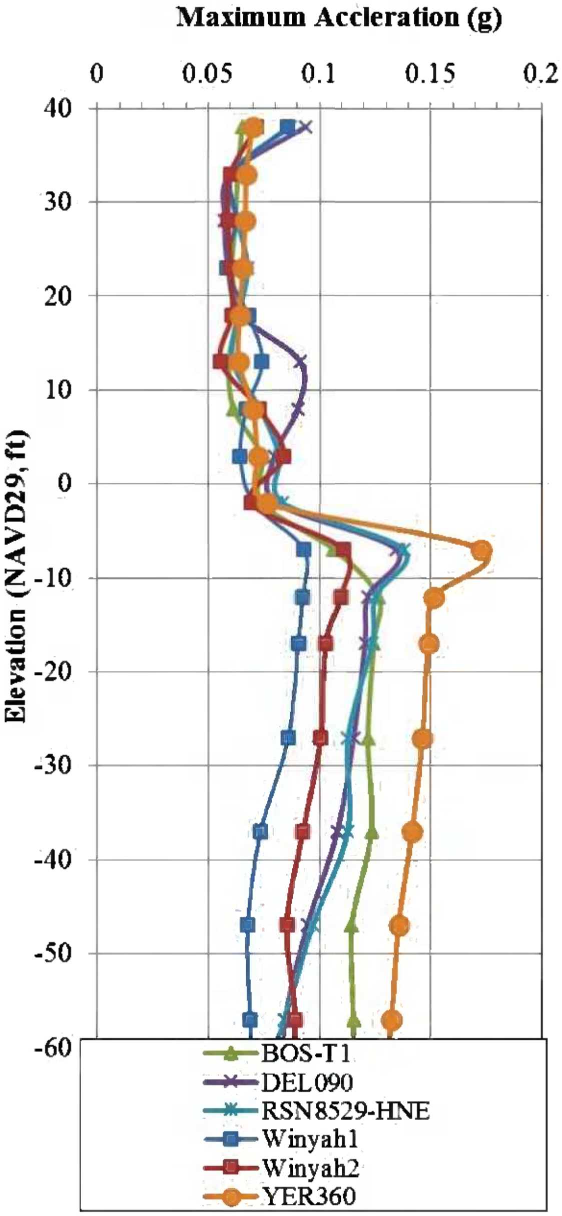


Figure 6-1. Calculated Maximum Acceleration for Profile 1

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.16g.

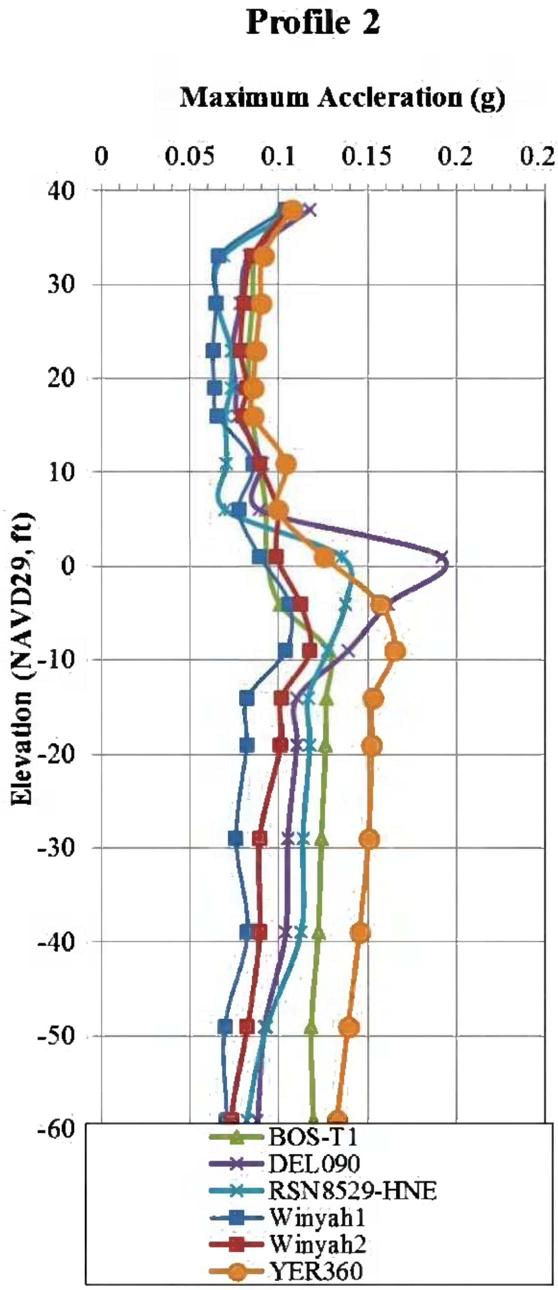


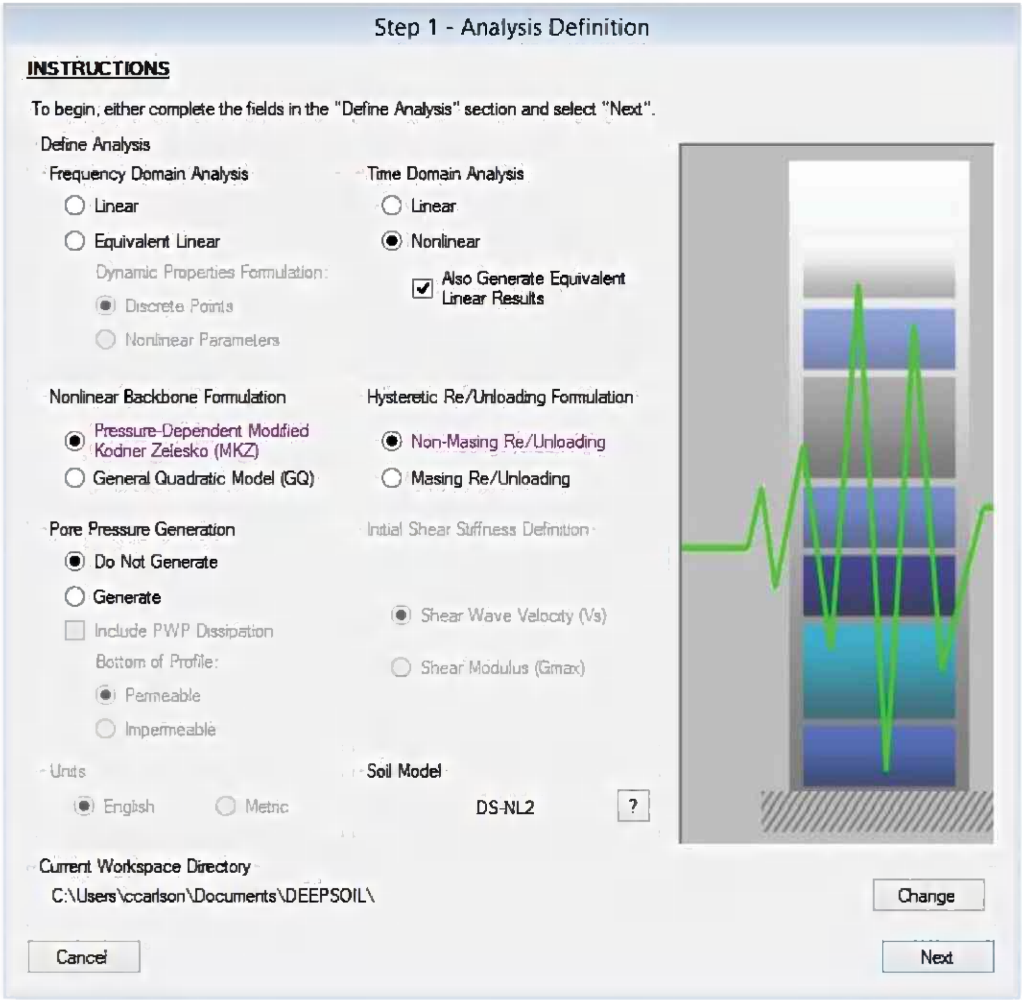
Figure 6-2. Calculated Maximum Acceleration for Profile 2

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.16g.

Appendix 7

DEEPSOIL® Input



Step 1 - Analysis Definition

INSTRUCTIONS

To begin, either complete the fields in the "Define Analysis" section and select "Next".

Define Analysis

Frequency Domain Analysis
- ○ Linear
- ○ Equivalent Linear
  - Dynamic Properties Formulation:
    - ● Discrete Points
    - ○ Nonlinear Parameters

Time Domain Analysis
- ○ Linear
- ● Nonlinear
  - ☑ Also Generate Equivalent Linear Results

Nonlinear Backbone Formulation
- ● Pressure-Dependent Modified Kodner Zelesko (MKZ)
- ○ General Quadratic Model (GQ)

Hysteretic Re/Unloading Formulation
- ● Non-Masing Re/Unloading
- ○ Masing Re/Unloading

Pore Pressure Generation
- ● Do Not Generate
- ○ Generate
  - ☐ Include PWP Dissipation
  - Bottom of Profile:
    - ● Permeable
    - ○ Impermeable

Initial Shear Stiffness Definition
- ● Shear Wave Velocity (Vs)
- ○ Shear Modulus (Gmax)

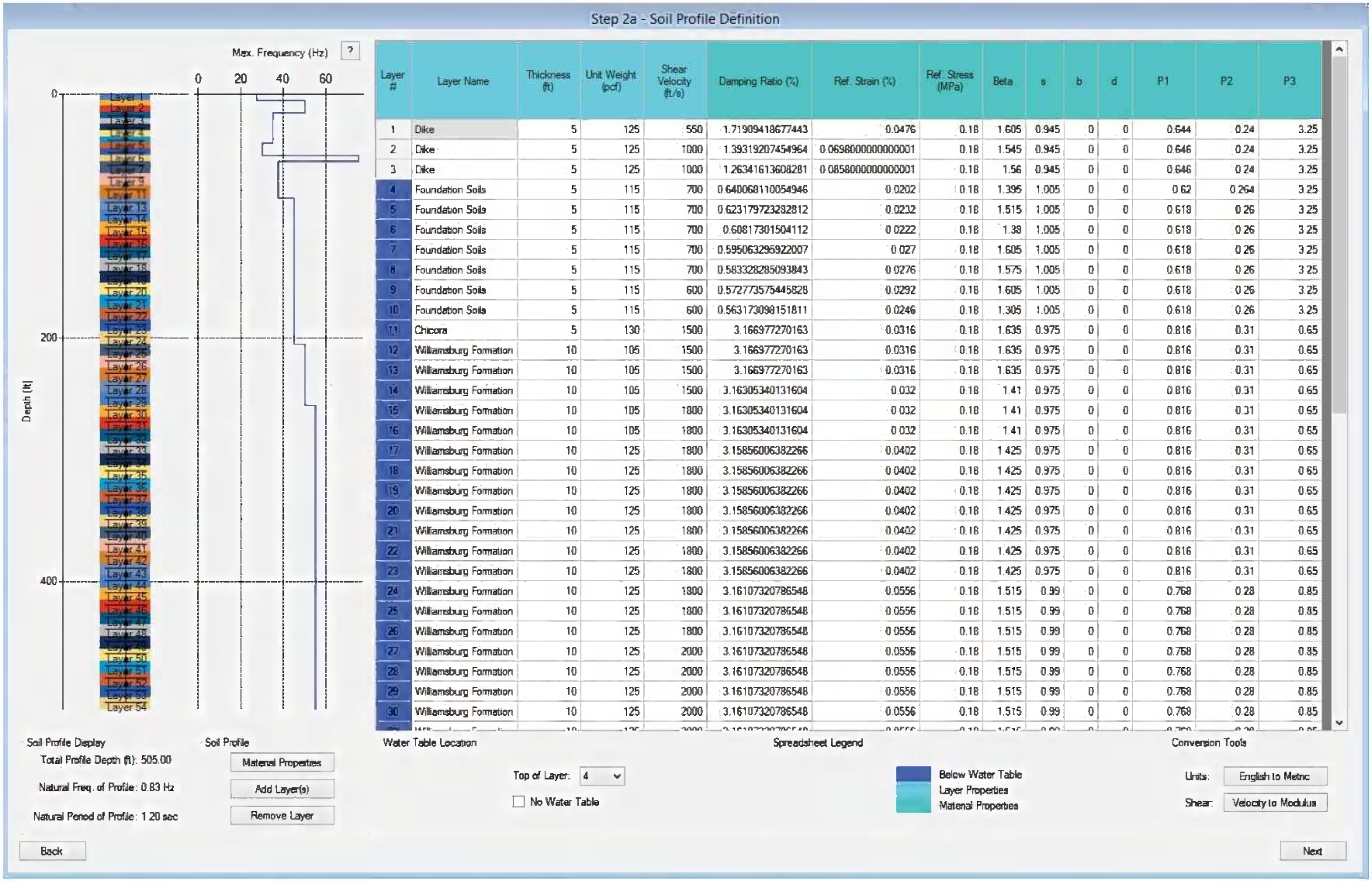
Units
- ● English
- ○ Metric

Soil Model: DS-NL2

Current Workspace Directory
C:\Users\ccarlson\Documents\DEEPSOIL\

Written by: **C. Carlson** Date: **10/12/2016** Reviewed by: **G. Rix** Date: **10/12/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
1	Dike	5	125	550	1.71909418677443	0.0476	0.18	1.605	0.945	0	0	0.644	0.24	3.25
2	Dike	5	125	1000	1.39319207454964	0.0698000000000001	0.18	1.545	0.945	0	0	0.646	0.24	3.25
3	Dike	5	125	1000	1.26341613608281	0.0858000000000001	0.18	1.56	0.945	0	0	0.646	0.24	3.25
4	Foundation Soils	5	115	700	0.640068110054946	0.0202	0.18	1.395	1.005	0	0	0.62	0.264	3.25
5	Foundation Soils	5	115	700	0.623179723282812	0.0232	0.18	1.515	1.005	0	0	0.618	0.26	3.25
6	Foundation Soils	5	115	700	0.60817301504112	0.0222	0.18	1.38	1.005	0	0	0.618	0.26	3.25
7	Foundation Soils	5	115	700	0.595063295922007	0.027	0.18	1.605	1.005	0	0	0.618	0.26	3.25
8	Foundation Soils	5	115	700	0.583328285093843	0.0276	0.18	1.575	1.005	0	0	0.618	0.26	3.25
9	Foundation Soils	5	115	600	0.572773575445828	0.0292	0.18	1.605	1.005	0	0	0.618	0.26	3.25
10	Foundation Soils	5	115	600	0.563173098151811	0.0246	0.18	1.305	1.005	0	0	0.618	0.26	3.25
11	Chicora	5	130	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
12	Williamsburg Formation	10	105	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
13	Williamsburg Formation	10	105	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
14	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
15	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
16	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
17	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
18	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
19	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
20	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
21	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
22	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
23	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
24	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
25	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
26	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
27	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85

Soil Profile Display

Total Profile Depth (ft): 505.00

Natural Freq. of Profile: 0.83 Hz

Natural Period of Profile: 1.20 sec

Top of Layer: 4

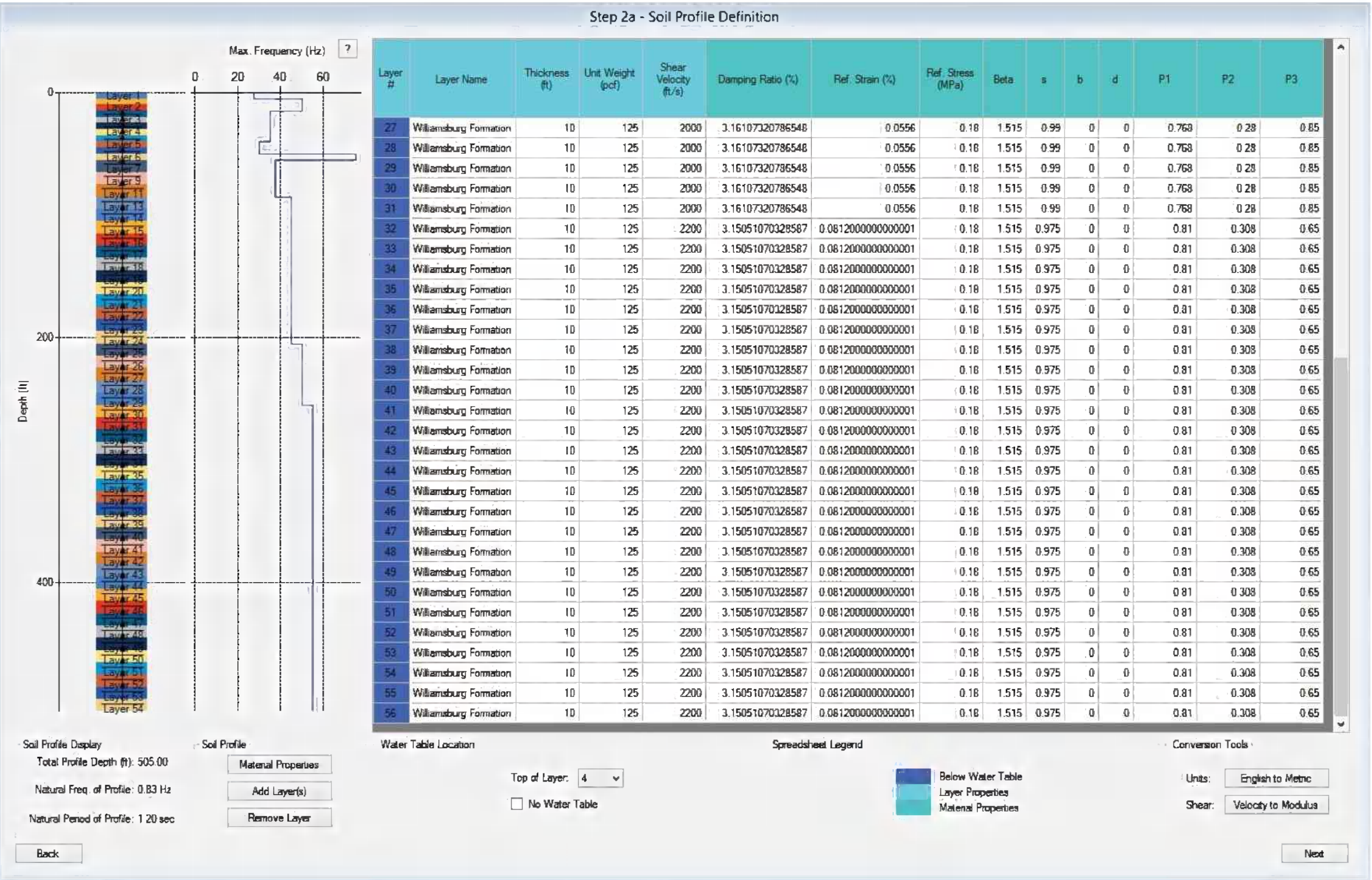
☐ No Water Table

Units: English to Metric

Shear: Velocity to Modulus

Written by: **C. Carlson** Date: **10/12/2016** Reviewed by: **G. Rix** Date: **10/12/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



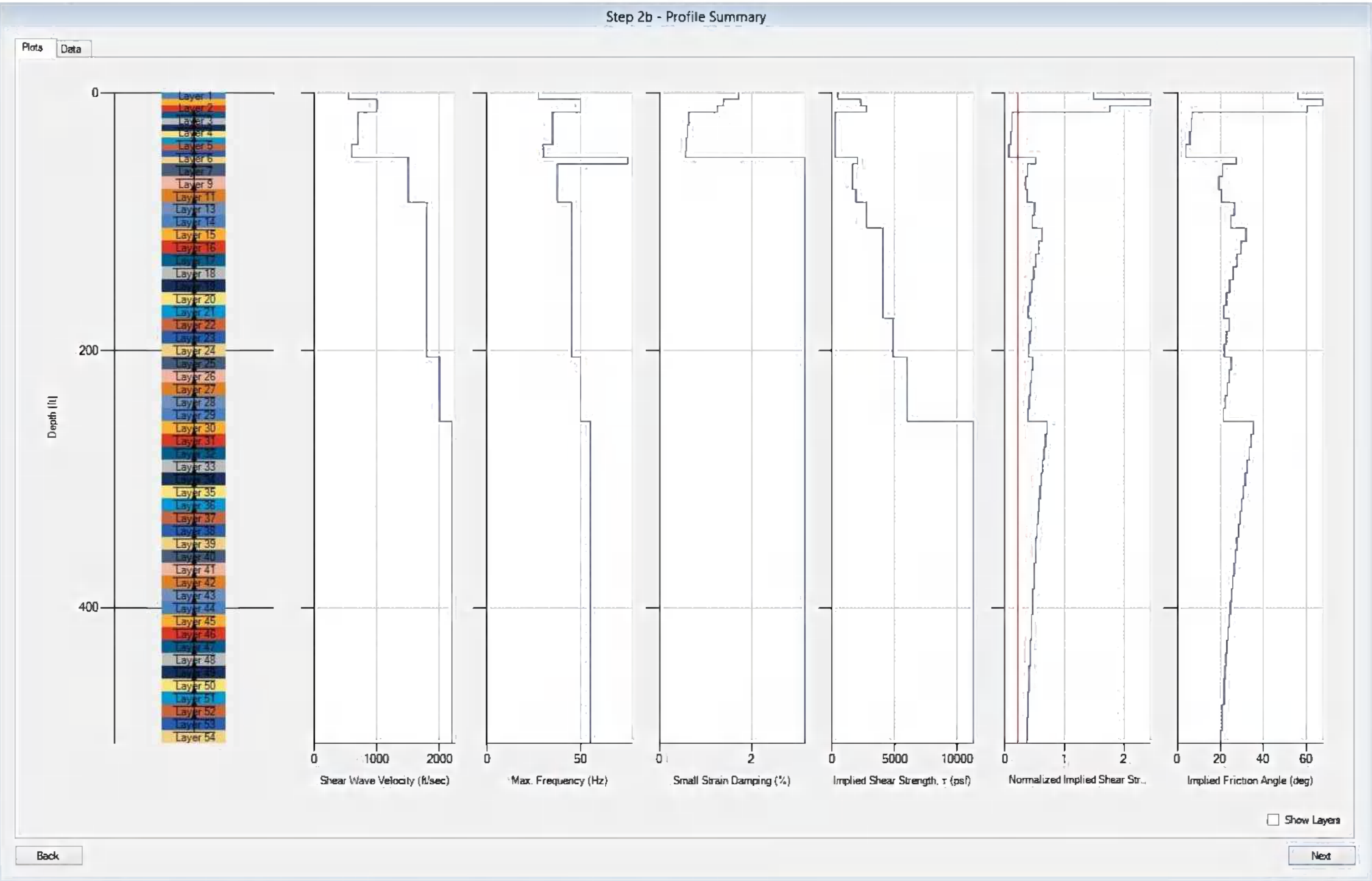
Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
27	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
31	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
32	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
33	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
34	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
35	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
36	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
37	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
38	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
39	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
40	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
41	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
42	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
43	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
44	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
45	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
46	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
47	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
48	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
49	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
50	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
51	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
52	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
53	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
54	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
55	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
56	Williamsburg Formation	10	125	2200	3.15051070328587	0.08120000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65

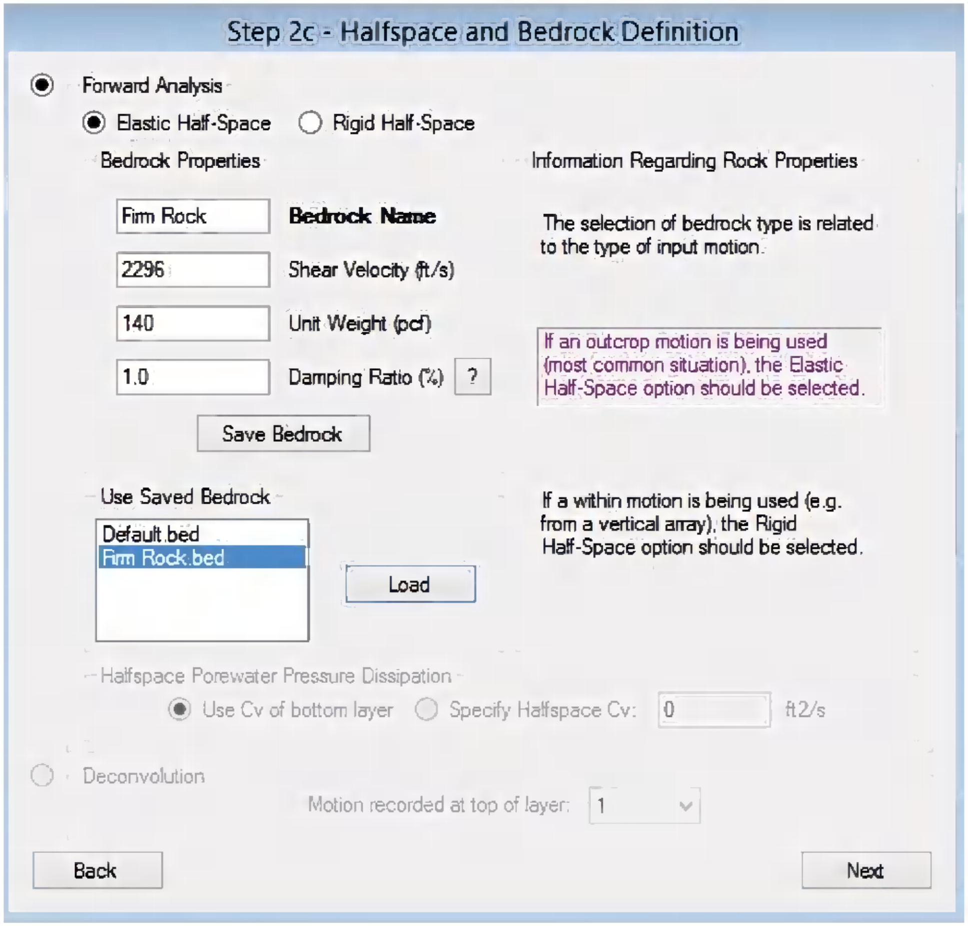
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.83 Hz
Natural Period of Profile: 1.20 sec

Top of Layer: 4
☐ No Water Table

Written by: **C. Carlson** Date: **10/12/2016** Reviewed by: **G. Rix** Date: **10/12/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**





Step 2c - Halfspace and Bedrock Definition

● Forward Analysis

● Elastic Half-Space ○ Rigid Half-Space

Bedrock Properties

Bedrock Name: Firm Rock

Shear Velocity (ft/s): 2296

Unit Weight (pcf): 140

Damping Ratio (%): 1.0

Information Regarding Rock Properties

The selection of bedrock type is related to the type of input motion.

If an outcrop motion is being used (most common situation), the Elastic Half-Space option should be selected.

If a within motion is being used (e.g. from a vertical array), the Rigid Half-Space option should be selected.

Use Saved Bedrock

Default.bed

Firm Rock.bed

Halfspace Porewater Pressure Dissipation

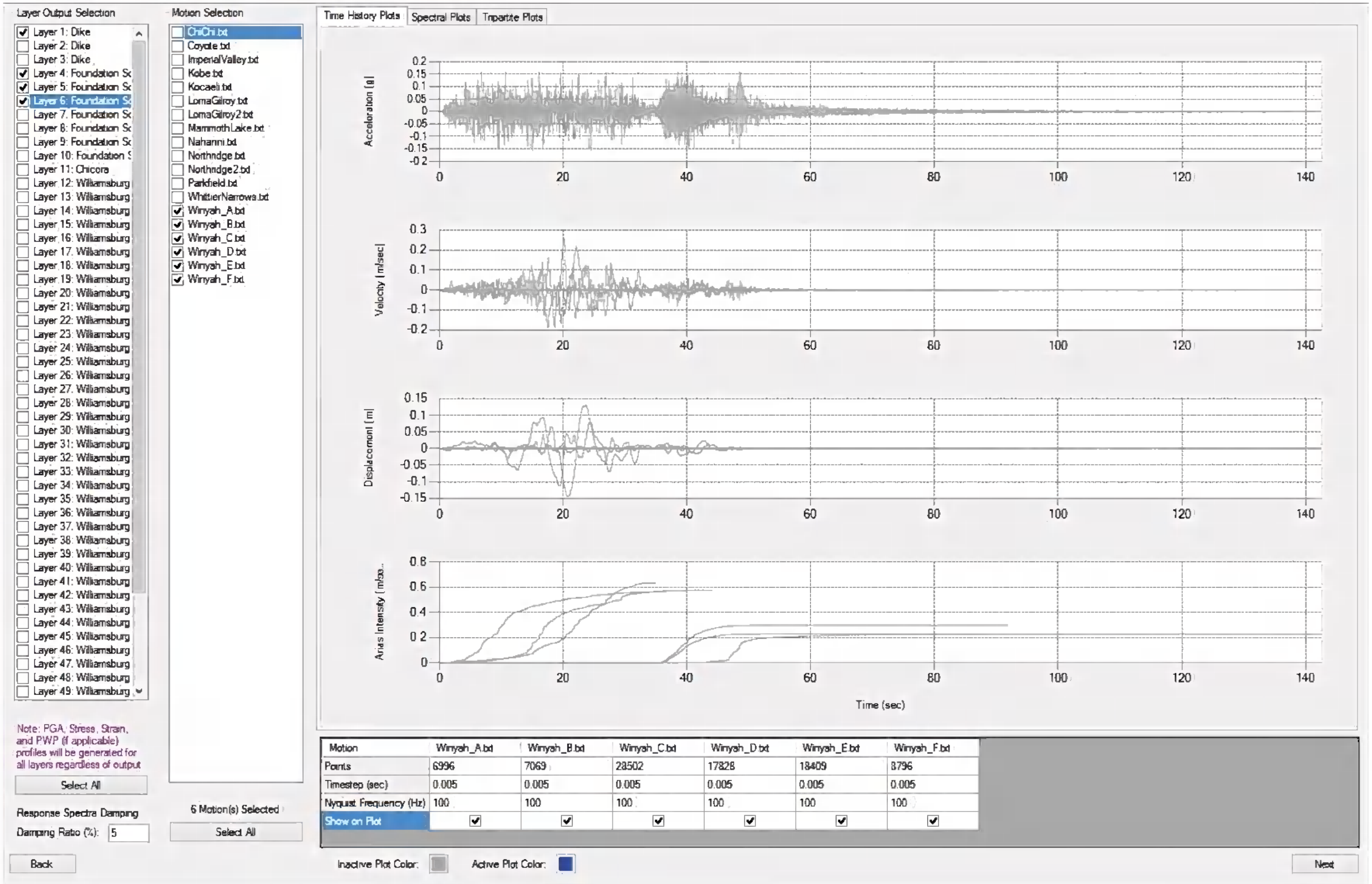
● Use Cv of bottom layer ○ Specify Halfspace Cv: 0 ft2/s

○ Deconvolution

Motion recorded at top of layer: 1

Written by: **C. Carlson** Date: **10/12/2016** Reviewed by: **G. Rix** Date: **10/12/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



Layer Output Selection

☑ Layer 1: Dike
☐ Layer 2: Dike
☐ Layer 3: Dike
☑ Layer 4: Foundation Sc
☑ Layer 5: Foundation Sc
☑ Layer 6: Foundation Sc
☐ Layer 7: Foundation Sc
☐ Layer 8: Foundation Sc
☐ Layer 9: Foundation Sc
☐ Layer 10: Foundation S
☐ Layer 11: Chicora
☐ Layer 12: Williamsburg
☐ Layer 13: Williamsburg
☐ Layer 14: Williamsburg
☐ Layer 15: Williamsburg
☐ Layer 16: Williamsburg
☐ Layer 17: Williamsburg
☐ Layer 18: Williamsburg
☐ Layer 19: Williamsburg
☐ Layer 20: Williamsburg
☐ Layer 21: Williamsburg
☐ Layer 22: Williamsburg
☐ Layer 23: Williamsburg
☐ Layer 24: Williamsburg
☐ Layer 25: Williamsburg
☐ Layer 26: Williamsburg
☐ Layer 27: Williamsburg
☐ Layer 28: Williamsburg
☐ Layer 29: Williamsburg
☐ Layer 30: Williamsburg
☐ Layer 31: Williamsburg
☐ Layer 32: Williamsburg
☐ Layer 33: Williamsburg
☐ Layer 34: Williamsburg
☐ Layer 35: Williamsburg
☐ Layer 36: Williamsburg
☐ Layer 37: Williamsburg
☐ Layer 38: Williamsburg
☐ Layer 39: Williamsburg
☐ Layer 40: Williamsburg
☐ Layer 41: Williamsburg
☐ Layer 42: Williamsburg
☐ Layer 43: Williamsburg
☐ Layer 44: Williamsburg
☐ Layer 45: Williamsburg
☐ Layer 46: Williamsburg
☐ Layer 47: Williamsburg
☐ Layer 48: Williamsburg
☐ Layer 49: Williamsburg

Note: PGA, Stress, Strain, and PWP (if applicable) profiles will be generated for all layers regardless of output

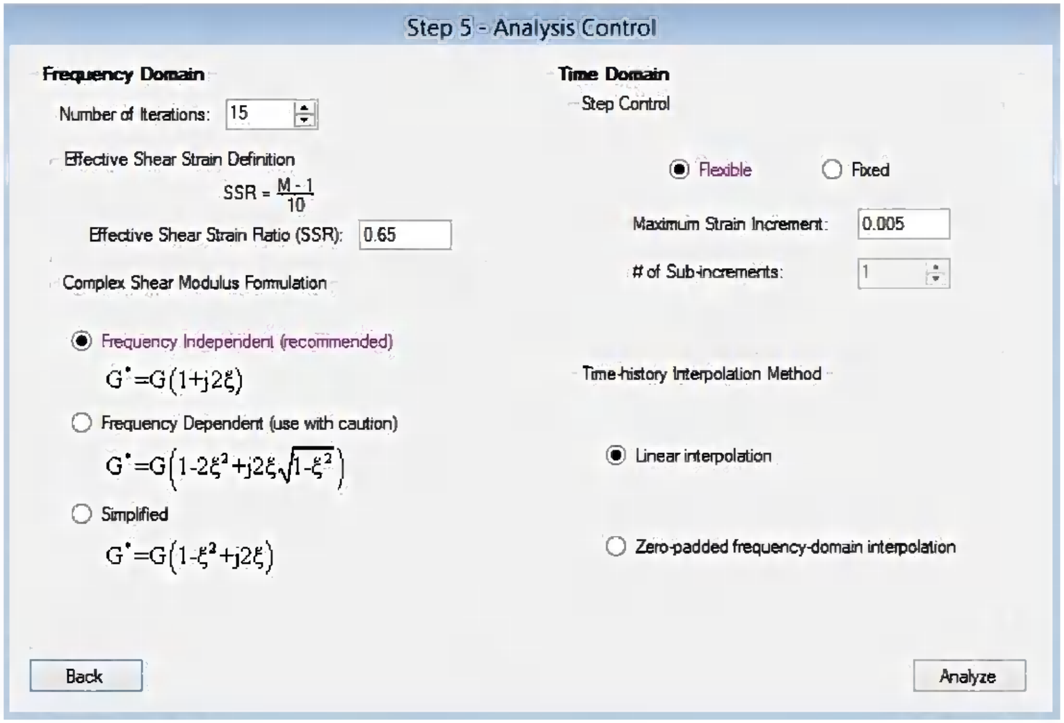
Response Spectra Damping
Damping Ratio (%): 5

Motion Selection

☐ ChiChi.txt
☐ Coyote.txt
☐ ImperialValley.txt
☐ Kobe.txt
☐ Kocaeli.txt
☐ LomaGilroy.txt
☐ LomaGilroy2.txt
☐ MammothLake.txt
☐ Nahanni.txt
☐ Northridge.txt
☐ Northridge2.txt
☐ Parkfield.txt
☐ WhittierNarrows.txt
☑ Winyah_A.txt
☑ Winyah_B.txt
☑ Winyah_C.txt
☑ Winyah_D.txt
☑ Winyah_E.txt
☑ Winyah_F.txt

6 Motion(s) Selected

Motion	Winyah_A.txt	Winyah_B.txt	Winyah_C.txt	Winyah_D.txt	Winyah_E.txt	Winyah_F.txt
Points	6996	7069	28502	17828	18409	8796
Timestep (sec)	0.005	0.005	0.005	0.005	0.005	0.005
Nyquist Frequency (Hz)	100	100	100	100	100	100
Show on Plot	☑	☑	☑	☑	☑	☑



Step 5 - Analysis Control

Frequency Domain

Number of Iterations: 15

Effective Shear Strain Definition

$SSR = \frac{M-1}{10}$

Effective Shear Strain Ratio (SSR): 0.65

Complex Shear Modulus Formulation

● Frequency Independent (recommended)

$G^* = G(1 + j2\xi)$

○ Frequency Dependent (use with caution)

$G^* = G\left(1 - 2\xi^2 + j2\xi\sqrt{1-\xi^2}\right)$

○ Simplified

$G^* = G\left(1 - \xi^2 + j2\xi\right)$

Time Domain

Step Control

● Flexible ○ Fixed

Maximum Strain Increment: 0.005

# of Sub-increments: 1

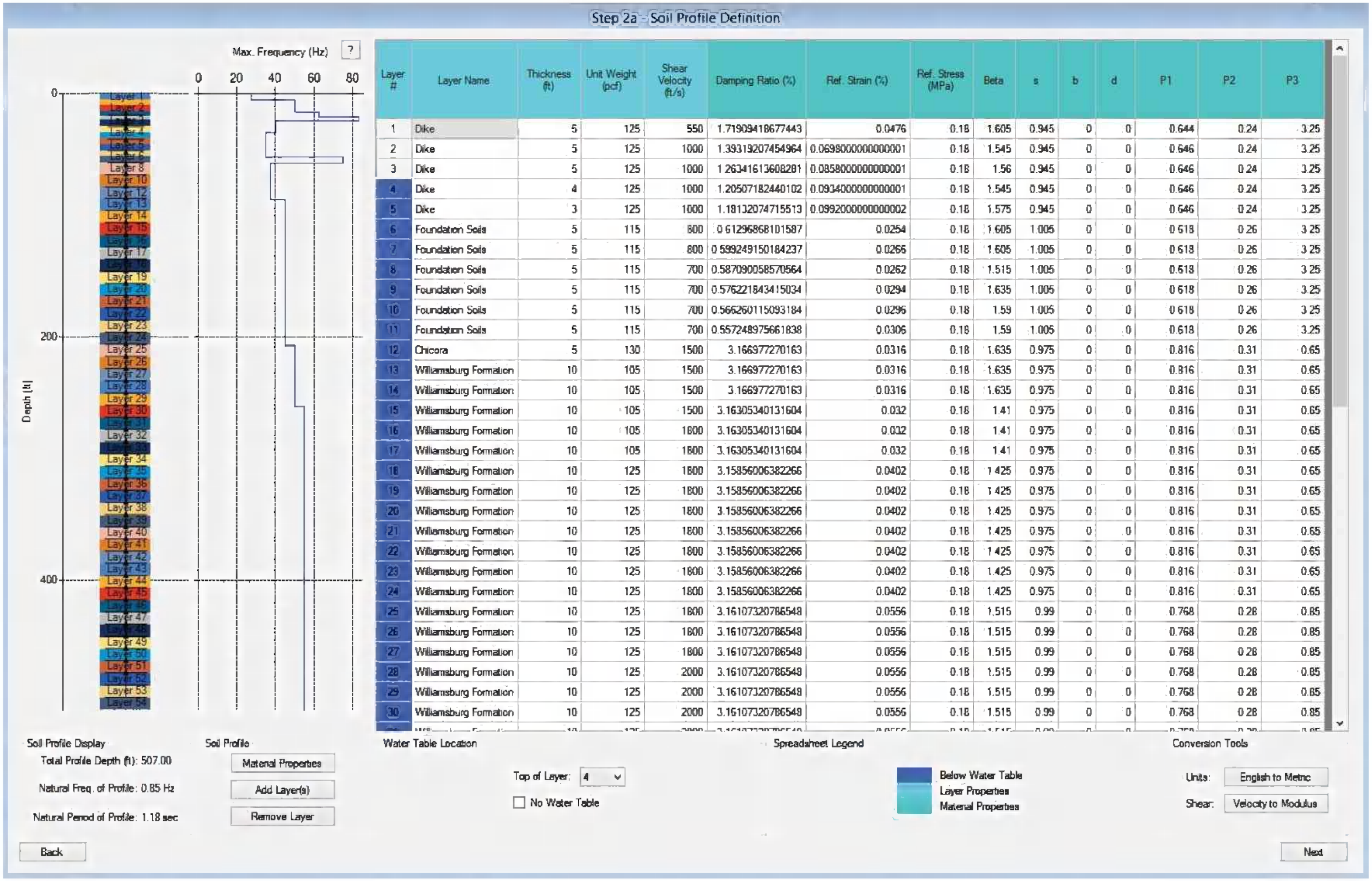
Time-history Interpolation Method

● Linear interpolation

○ Zero-padded frequency-domain interpolation

Written by: **C. Carlson** Date: **10/12/2016** Reviewed by: **G. Rix** Date: **10/12/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
1	Dike	5	125	550	1.71909418677443	0.0476	0.18	1.605	0.945	0	0	0.644	0.24	3.25
2	Dike	5	125	1000	1.39319207454964	0.0698000000000001	0.18	1.545	0.945	0	0	0.646	0.24	3.25
3	Dike	5	125	1000	1.26341613608281	0.0858000000000001	0.18	1.56	0.945	0	0	0.646	0.24	3.25
4	Dike	4	125	1000	1.20507182440102	0.0934000000000001	0.18	1.545	0.945	0	0	0.646	0.24	3.25
5	Dike	3	125	1000	1.18132074715513	0.0992000000000002	0.18	1.575	0.945	0	0	0.646	0.24	3.25
6	Foundation Soils	5	115	800	0.61296868101587	0.0254	0.18	1.605	1.005	0	0	0.618	0.26	3.25
7	Foundation Soils	5	115	800	0.599249150184237	0.0266	0.18	1.605	1.005	0	0	0.618	0.26	3.25
8	Foundation Soils	5	115	700	0.587090058570564	0.0262	0.18	1.515	1.005	0	0	0.618	0.26	3.25
9	Foundation Soils	5	115	700	0.576221843415034	0.0294	0.18	1.635	1.005	0	0	0.618	0.26	3.25
10	Foundation Soils	5	115	700	0.566260115093184	0.0296	0.18	1.59	1.005	0	0	0.618	0.26	3.25
11	Foundation Soils	5	115	700	0.557248975661838	0.0306	0.18	1.59	1.005	0	0	0.618	0.26	3.25
12	Chicora	5	130	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
13	Williamsburg Formation	10	105	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
14	Williamsburg Formation	10	105	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
15	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
16	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
17	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
18	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
19	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
20	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
21	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
22	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
23	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
24	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
25	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
26	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
27	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85

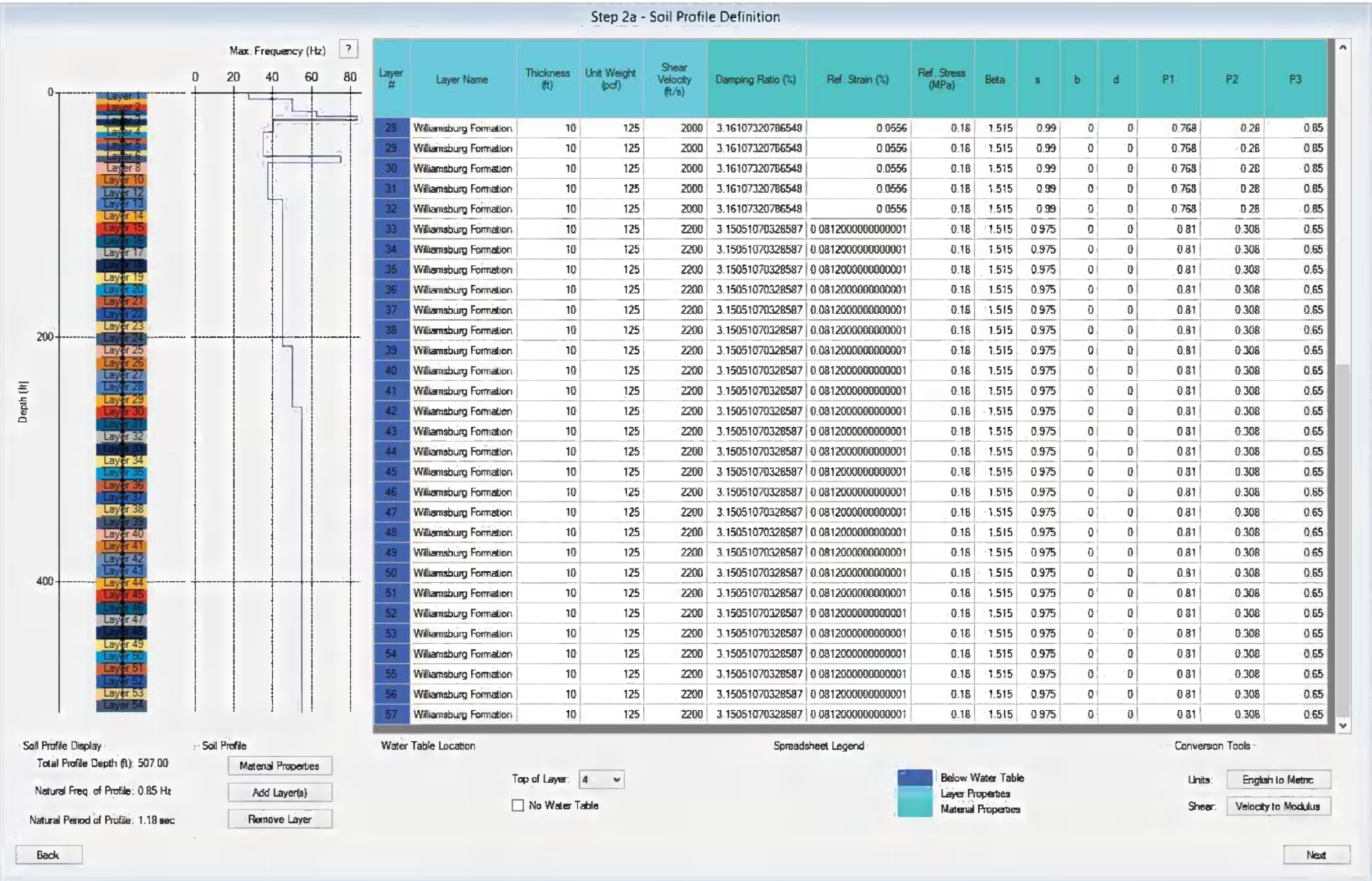
Soil Profile Display
Total Profile Depth (ft): 507.00
Natural Freq. of Profile: 0.85 Hz
Natural Period of Profile: 1.18 sec

Top of Layer: 4
☐ No Water Table

Units: English to Metric
Shear: Velocity to Modulus

Written by: C. Carlson Date: 10/12/2016 Reviewed by: G. Rix Date: 10/12/2016

Client: Santee Cooper Project: Winyah Generating Station Project/ Proposal No.: GSC5242 Task No: 01BT



Step 2a - Soil Profile Definition

Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
31	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
32	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
33	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
34	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
35	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
36	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
37	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
38	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
39	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
40	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
41	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
42	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
43	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
44	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
45	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
46	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
47	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
48	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
49	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
50	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
51	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
52	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
53	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
54	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
55	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
56	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
57	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65

Soil Profile Display

Total Profile Depth (ft): 507.00

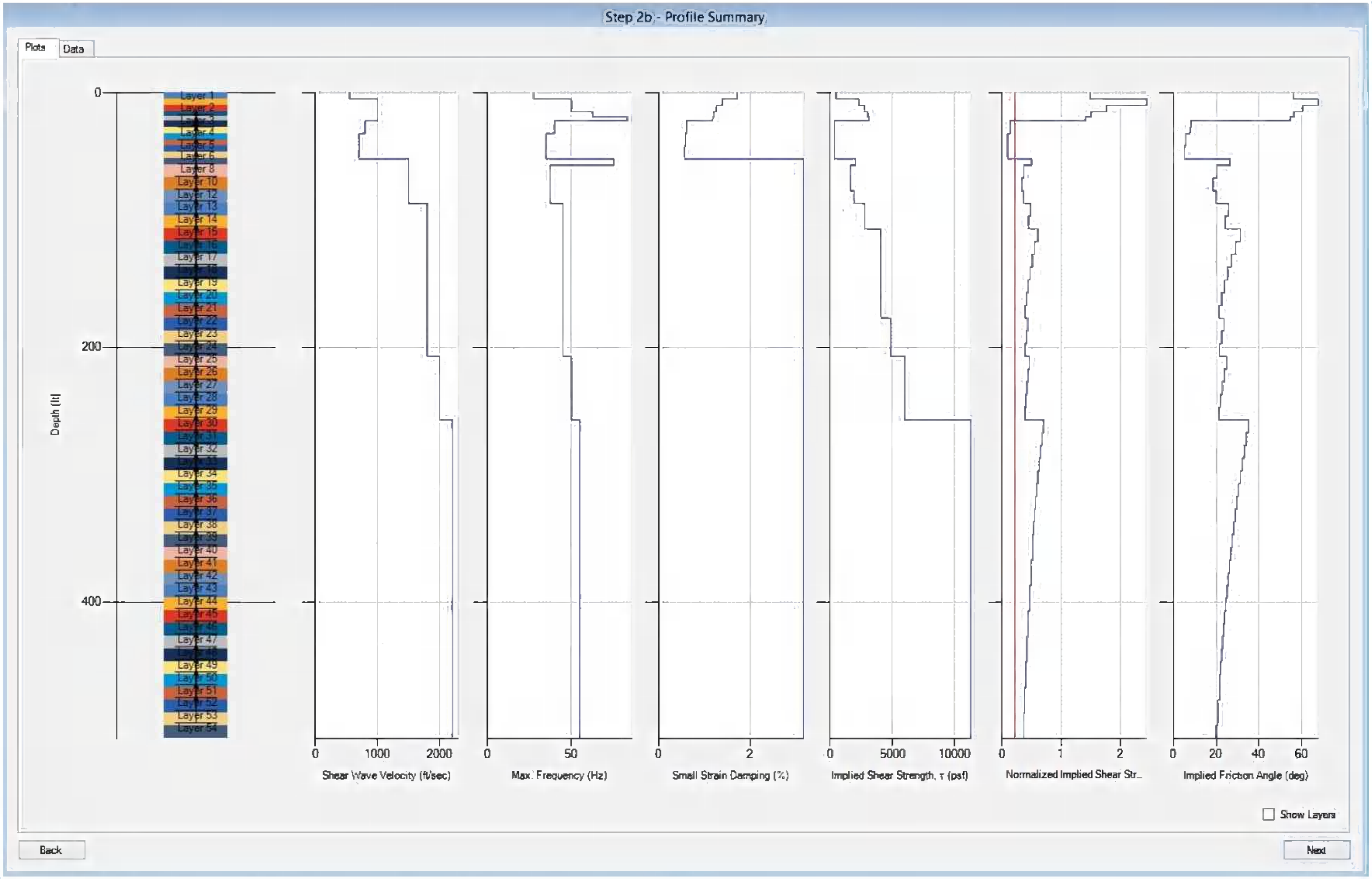
Natural Freq. of Profile: 0.85 Hz

Natural Period of Profile: 1.18 sec

Water Table Location

Top of Layer: 4

☐ No Water Table


Step 2b - Profile Summary
Plots
Data
Depth (ft)
0
200
400
Layer 1
Layer 2
Layer 3
Layer 4
Layer 5
Layer 6
Layer 8
Layer 10
Layer 12
Layer 13
Layer 14
Layer 15
Layer 16
Layer 17
Layer 18
Layer 19
Layer 20
Layer 21
Layer 22
Layer 23
Layer 24
Layer 25
Layer 26
Layer 27
Layer 28
Layer 29
Layer 30
Layer 31
Layer 32
Layer 33
Layer 34
Layer 35
Layer 36
Layer 37
Layer 38
Layer 39
Layer 40
Layer 41
Layer 42
Layer 43
Layer 44
Layer 45
Layer 46
Layer 47
Layer 48
Layer 49
Layer 50
Layer 51
Layer 52
Layer 53
Layer 54
0
1000
2000
Shear Wave Velocity (ft/sec)
0
50
Max. Frequency (Hz)
0
2
Small Strain Damping (%)
0
5000
10000
Implied Shear Strength, τ (psf)
0
1
2
Normalized Implied Shear Str...
0
20
40
60
Implied Friction Angle (deg)
☐ Show Layers
Back
Next

ATTACHMENT 7

Liquefaction Potential Analysis

CALCULATION PACKAGE COVER SHEET

Client: Santee Cooper **Project:** Winyah Generating Station **Project No.** GSC5242

TITLE OF PACKAGE: LIQUEFACTION POTENTIAL ANALYSIS: ASH POND A

Calculation Prepared by:
Signature James McNash
Name James McNash, P.E.
Date 10/10/2016

Assumptions & Procedures Checked by:
(peer reviewer)
Signature
Name Ming Zhu, Ph.D., P.E.
Glenn J. Rix, Ph.D., P.E.
Date 10/10/2016

Computations Checked by:
Signature
Name Josh Colley, EI
Date 10/10/2016

Computations Back-checked by:
Signature James McNash
Name James McNash, P.E.
Date 10/10/2016

Approved by:
(pm or designate)
Signature
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Date 10/10/2016

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LIQUEFACTION POTENTIAL ANALYSIS: ASH POND A

INTRODUCTION

This liquefaction potential analysis calculation package (Liquefaction Package) was prepared to present the evaluation for soil liquefaction potential of the perimeter dike soils forming Ash Pond A at Winyah Generating Station (WGS or Site). This calculation package is Attachment 7 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report* (Safety Factor Assessment Report) prepared by Geosyntec Consultants (Geosyntec) to demonstrate compliance with the United States Environmental Protection Agency's (USEPA) Coal Combustion Residuals (CCR) Rule with respect to the periodic stability and safety factor assessment criteria presented in 40 Code of Federal Regulations (CFR) 257.73(e). Ground motions and resulting cyclic shear stresses for the analyzed design ground motions are presented in Attachment 6 titled "Seismic Hazard Evaluation and Site Response Analysis: Ash Pond A" (Site Response Package) to the Safety Factor Assessment Report. The liquefaction potential was evaluated for soil borings and cone penetration test (CPT) soundings advanced through the Ash Pond A perimeter dike based on geotechnical information collected during Geosyntec's 2013 geotechnical subsurface investigation. Soil borings and CPT soundings located at the perimeter dike toe will be analyzed during an evaluation of "Unstable Areas" at a later time. Details of this investigation are discussed in Attachment 5 titled "Subsurface Stratigraphy and Material Properties: Ash Pond A" (Data Package) to the Safety Factor Assessment Report. The remainder of this Liquefaction Package presents: (i) methodology; (ii) analysis cases; (iii) input parameters; (iv) results; (v) conclusions; and (vi) references.

METHODOLOGY

Current state-of-practice procedures for evaluating the liquefaction potential of a soil were developed based on case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. Occurrences (or non-occurrences) of liquefaction were determined by presence (or absence) of surface manifestations of liquefaction such as sand boils, ground cracking, slope movements, and/or flow failures. Surface manifestations were generally present if large excess pore pressures are generated during seismic loading and "liquefaction" is triggered. Therefore, if soils at a particular site are deemed not to be susceptible to liquefaction based on the state-of-practice or regulatory guidance, additional analyses such as post-liquefaction slope stability or lateral spreading are not necessary for the anticipated seismic ground motions.

It was assumed that soils classified as Organic Peat, Silt, and Clay, or a combination of these materials, are typically not liquefiable. Additionally, soils that exhibit "clay-like" behavior according to data collected during CPT soundings were also screened as not potentially liquefiable. "Clay-like" behavior was defined as a soil with a Soil Behavior Index (I_c) greater than 2.60. The interpretation of

CPT soundings and the computation of I_c are discussed in the Data Package and reiterated below. If a zone of soil that was considered to be non-liquefiable by the above criteria, the soil zone was assigned a factor of safety (FS) against liquefaction triggering of 2.00. The criteria recommended by Bray and Sancio (2006) were applied to evaluate the susceptibility of fine-grained soils to cyclic softening. All of the tested samples were found to be "Not Susceptible" to cyclic softening by these criteria.

The liquefaction analysis described below was performed based on the simplified procedure recommended by Seed and Idriss (1971) and later updated by Idriss and Boulanger (2008). Analyses were performed on both the CPT soundings and soil borings. The methodology to compute the potential of soils to liquefy and the factor of safety against liquefaction are described below.

Cyclic Stress Ratio

The Cyclic Stress Ratio (CSR) is a measure of the shear stresses developed during an earthquake normalized with effective overburden stress. The CSR for a soil interval is calculated as follows:

$$\mathrm{CSR}_{M,\sigma'_{vo}} = 0.65\,\frac{\tau_{max}}{\sigma'_{vo}} \tag{1}$$

where:

$\mathrm{CSR}_{M,\sigma'vo}$ = Cyclic Stress Ratio due to an earthquake with magnitude, M;
τ_{max} = maximum shear stress developed during an earthquake (psf); and
σ'_{vo} = effective vertical stress (psf).

The cyclic shear stress represents the loading or demand on a soil unit during an earthquake.

Corrected Normalized CPT Sounding Interpretation

To evaluate the resistance or capacity of the soil against liquefaction, soil data must be interpreted from each boring or sounding. A discussion of the interpretation of the CPT data is provided in the Data Package, and the equations used in the interpretation are reiterated below.

The normalized cone tip resistance, Q, and friction ratio, F, were calculated by:

$$Q = \left(\frac{q_c-\sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \tag{2}$$

and,

$$F = \left(\frac{f_s}{q_c-\sigma_{vo}}\right) \times 100\% \tag{3}$$

where:

q_c = measured tip resistance (tsf);
σ_{vo} = total vertical stress (tsf);
σ'_{vo} = effective vertical stress (tsf);
P_a = atmospheric pressure ($P_a = 1.058$ tsf);
n = varies from 0.5 in sands to 1.0 in clays; and
f_s = measured sleeve friction (tsf).

It is noted that the tip resistance (q_c) measured in the field must be adjusted for pore pressure effects on the cone tip if the data collection software does not automatically account for the area ratio of the cone. This correction is discussed within the Data Package.

The soil behavior type index, I_c, as derived by Robertson and Wride (1998) is calculated by:

$$I_c = \Big[(3.47 - \log(Q))^2 + (\log(F) + 1.22)^2\Big[^{0.5} \qquad (4)$$

The I_c is used to compute the soil behavior type (SBT) which may be used to infer the type of soil that is present at the data collection interval.

To compute the resistance of a soil interval against liquefaction, the overburden-corrected tip resistance, q_{c1}, must be computed for a soil unit. q_{c1} can be computed as follows:

$$q_{c1} = C_N q_c \qquad (5)$$

where:

C_N = overburden correction factor = $(P_a/\sigma'_{vo})^{1.338-0.249(q_{c1N})^{0.264}}$;
q_{c1N} = normalized tip resistance q_{c1}/P_a (dimensionless).

The computation of C_N was limited to a maximum value of 1.7 and is applicable for values of q_{c1N} between 21 and 254. As evident above, the computation of q_{c1} and q_{c1N} is an iterative procedure, which was performed using an algorithm developed within the MathCAD® computation software.

Corrected Normalized SPT Blow Count

Interpretation of soil test borings and SPT blow counts is discussed within the Data Package, but is briefly reiterated below. The corrected normalized SPT blow count, $(N_1)_{60}$, which is applied in computing resistance of a soil against liquefaction, was calculated by the following equation presented by Idriss and Boulanger (2008).

$$(N_1)_{60} = N_{meas} C_E C_B C_S C_R C_N \tag{6}$$

where:

N_{meas} = measured SPT blow count (blows/ft);
C_E = correction factor for energy ratio;
C_B = correction factor for borehole diameter;
C_R = correction factor for rod length;
C_S = correction factor for sampler; and
C_N = correction factor for overburden pressure.

The correction factor for the applied energy (C_E) is dependent on the type and calibration of the hammer system attached to the drill rig. The correction factor (C_E) converts the measured N-value to a standard value, which assumes a 60 percent efficiency of the hammer system. This correction factor was computed as follows:

$$C_E = \frac{ER}{60} \tag{7}$$

where:

ER = energy ratio of the SPT hammer system.

Energy ratios selected for these analyses are discussed later within this Liquefaction Package. The correction factors above (excluding C_N) are given in Table 1. C_N was calculated for $(N_1)_{60}$ values less than 46 blows per foot, as follows:

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{(0.784-0.0768\sqrt{(N_1)_{60}})} \tag{8}$$

where:

P_a = atmospheric pressure (2,117 psf); and
σ'_{vo} = effective vertical stress (psf).

The computation of C_N was limited to a maximum value of 1.7. As evident above, the computation of $(N_1)_{60}$ is an iterative procedure, which was performed using an algorithm developed within the MathCAD® computation software.

Cyclic Resistance Ratio (CRR)

The CRR is the measure of a soil's resistance to liquefaction. If the CSR > CRR, liquefaction is likely to occur during the analyzed seismic event. The CRR was computed from CPT sounding data based on the corrected tip resistance of clean sand for an earthquake of magnitude = 7.5 and an overburden pressure of one atmosphere, as follows:

$$\mathrm{CRR}_{\mathrm{M}=7.5,\sigma'_{\mathrm{vo}}=1\ \mathrm{atm}} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right) \qquad (9)$$

Equation 9 is considered valid for the equivalent clean sand corrected tip resistance (q_{c1Ncs}) with values less than 211. For clean sands, q_{c1Ncs}, is equivalent to q_{c1N}, but for soils with some percentage of fines, $q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$, where the correction factor, Δq_{c1N}, is given by:

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \times \exp\left(1.63 + \frac{9.7}{\mathrm{FC}+0.01} - \left(\frac{15.7}{\mathrm{FC}+0.01}\right)^2\right) \qquad (10)$$

where:

FC = percent of fines (by mass) within a soil.

The CRR was computed similarly for an earthquake of magnitude, M = 7.5, and an overburden pressure of one atmosphere, using corrected SPT N-values, as follows:

$$\mathrm{CRR}_{\mathrm{M}=7.5,\,\sigma'_{\mathrm{vo}}=1\ \mathrm{atm}} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \qquad (11)$$

For clean sands, the equivalent clean sand value of the SPT penetration resistance $(N_1)_{60cs}$, is equivalent to $(N_1)_{60}$, but for soils with some percentage of fines, $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$, where the correction factor, $\Delta(N_1)_{60}$, is given by:

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{\mathrm{FC}+0.01} - \left(\frac{15.7}{\mathrm{FC}+0.01}\right)^2\right) \qquad (12)$$

The selected fines content (FC) values used in these computations are discussed later within this calculation package. It is noted that $\Delta(N_1)_{60}$ is limited to a maximum value of 5.5.

Overburden Correction Factor

The overburden correction factor, K_σ, was introduced by Seed (1983) to adjust the CRR to a reference value of effective overburden stress because the CRR of sands is dependent on the effective overburden stress (Idriss and Boulanger, 2008). The recommended relationship for K_σ is given by:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right) \leq 1.1 \qquad (13)$$

where:

$$C_\sigma = 1 / (37.3 - 8.27(q_{c1N})^{0.264}) \quad \leq 0.3 \text{ for CPT soundings.} \tag{14}$$

and,

$$C_\sigma = 1 / (18.9 - 2.55((N_1)_{60})^{0.5}) \quad \leq 0.3 \text{ for SPT borings.} \tag{15}$$

Furthermore, Equations 14 and 15 are applicable for q_{c1N} and $(N_1)_{60}$ values less than 211 and 37 blows per foot, respectively. The overburden correction factor is used in liquefaction potential computations to adjust the CRR to a common effective overburden stress as shown by the following equation:

$$CRR_{\sigma'_{vo}} = K_\sigma \times CRR_{\sigma'_{vo}=1\,\text{atm}} \tag{16}$$

Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) is applied to adjust the CRR to a common earthquake magnitude, M (conventionally selected as M = 7.5). For cohesionless soils, the MSF is calculated using the equation proposed by Idriss (1999):

$$MSF = 6.9 \times \exp\left(\frac{-M}{4}\right) - 0.058, \text{ and MSF} \leq 1.8 \tag{17}$$

The MSF was calculated as 1.05 for a magnitude 7.3 earthquake, which was selected based on the deaggregation of the probabilistic seismic hazard as described in the Site Response Package (Attachment 6).

The CRR for a magnitude M earthquake is calculated as follows:

$$CRR_M = MSF \times CRR_{M=7.5} \tag{18}$$

Age Correction Factor (K_{DR})

Correlations associated with liquefaction potential analysis were developed based on case histories of the presence or absence of liquefaction in relatively young soil deposits (i.e., Holocene age). As described in the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (2010), the CRR may be adjusted to account for diagenesis and other age-related effects in older soils that have not previously experienced liquefaction. Equation 13-47 within Chapter 13 of the SCDOT Geotechnical Design Manual computes the Age Correction Factor (K_{DR}) based on its age (t in years) as:

$$K_{DR} = 0.17\log_{10}(t) + 0.83 \tag{19a}$$

Meanwhile, Andrus et al. (2008) presents a similar equation for the K_{DR} as:

$$K_{DR} = 0.19\log_{10}(t) + 0.68 \tag{19b}$$

It is noted that "t" is considered based on the "geotechnical age" instead of the "geologic age". Geologic age is the time since initial soil deposition; whereas geotechnical age is the time since the last significant liquefaction event resulting in re-sedimentation of the soil fabric.

Next, the CRR for sand strata was further adjusted by the age correction factor to account for this aging effect, and is computed as follows.

$$CRR_{M,K} = K_{DR} \times CRR_M \tag{20}$$

Factor of Safety

Finally, the factor of safety against liquefaction (FS_{liq}) triggering for both SPT and CPT analyses was computed by:

$$FS_{liq} = \frac{CRR_{M,\sigma'vo,Kdr}}{CSR_{M,\sigma'vo}} \tag{21}$$

where:

$CRR_{M,\sigma'vo,Kdr}$ = cyclic resistance ratio adjusted for earthquake magnitude, overburden, and aging;

and

$CSR_{M,\sigma'vo}$ = cyclic stress ratio for the same earthquake and overburden stress.

ANALYSIS CASES

As noted previously, liquefaction potential computations were conducted on soil data collected in borings and soundings overseen by Geosyntec in 2013. Computations were limited to soil borings and soundings located through the dike centerline and beneath the dike centerline.

Two representative soil profiles of shear wave velocity (V_s) were developed and presented in the Data Package from the dike fill soils to the Chicora stratum. These profiles for the perimeter dikes adjacent to the Intake and Discharge Canals (Profile 1) and the perimeter dike adjacent to the Cooling Pond (Profile 2) were developed from direct measurements of V_s and by means of a correlation with CPT sounding data. As discussed in the Data Package, these representative V_s profiles were supplemented with historical data to extend the V_s profile into the underlying Chicora and Williamsburg Formation Clay strata during the site response analysis.

For each representative soil profile, a site response analysis, described within the Site Response Package (Attachment 6), was performed using six ground motions selected for the Site. A profile of the maximum cyclic shear stress (τ_{max}) was computed for each ground motion and the maximum value at each depth was selected to create a single profile of τ_{max} for each representative profile. These τ_{max} profiles were applied to compute the CSR at every depth for each soil boring or sounding. The maximum shear stress at each computed depth for each representative profile was tabulated and provided in Table 2. The τ_{max} for measurements in between depth intervals listed within Table 2 were linearly interpolated to calculate τ_{max} at every depth interval.

INPUT PARAMETERS

The following section describes the selection of the input parameters applied for the liquefaction potential analysis.

Total Unit Weight

The total unit weight (γ_T) of a soil interval was applied in liquefaction potential computations to calculate the total and effective stress states for the soil column for each soil boring and sounding analyzed. For the purpose of this analysis, CPT intervals were assigned a unit weight based on the ranges presented for soils in the region provided within the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (GDM) (SCDOT, 2010) and the site laboratory data (Attachment 5). The assigned unit weight is dependent on the initial soil behavior index (I_c), as follows:

- Clays and clayey sand mixtures ($I_c > 2.95$): 100 pcf
- Silt to silty sand mixtures ($2.60 < I_c \leq 2.95$): 100 pcf
- Silty sands to sand mixtures ($2.05 < I_c \leq 2.60$): 110 pcf
- Sands ($1.31 < I_c \leq 2.05$): 120 pcf
- Gravelly sands to sands ($I_c \leq 1.31$): 125 pcf

SPT intervals were assigned total unit weight values based on visual and laboratory observations on the soil type as follows:

- Clays and Silts: 100 pcf
- Loose Sands ($N \leq 10$ blows/foot): 105 pcf
- Medium Dense Sands (10 blows/foot $< N \leq 30$ blows/foot): 115 pcf
- Dense Sands ($N \geq 30$ blows/foot): 120 pcf
- Chicora: 130 pcf
- Williamsburg Formation Clay: 105 pcf

Age Correction Factor

The susceptibility of soil deposits to liquefaction was summarized by type of deposit and geologic age by Youd and Perkins (1978), and is provided in Table 3. Youd and Perkins (1978) observed that younger soils (Holocene age) generally are the most susceptible to liquefaction. In the South Carolina (SC) region, the influence of soil age was investigated locally by Arango et al. (2009) and Andrus et al. (2008) based on cyclic strength testing of high-quality samples of sand and in-situ testing on paleoliquefaction sites, respectively. Each researcher compared observations and results in each study with the case-history-based chart for liquefaction triggering developed by Idriss and Boulanger (2008). Andrus (2008) developed a correlation (Equation 19b) relating soil age to a correction factor to CRR. Additionally, Leon et al. (2005) investigated a site nearby to WGS (Sampit, SC) and identified soil ages for sands encountered between 546 to 450,000 years old. Age Correction Factors (K_{DR}) were computed based on Equations 19a and 19b for the range of soil ages observed in the region presented by Leon et al. (2005), and are provided in Table 4. A K_{DR} was selected from Table 4 and applied to soils in the vicinity of the Ash Pond A perimeter dikes that were evaluated to have geologic and geotechnical ages older than Holocene age (i.e., foundation soils).

As shown in Figure 1, soils immediately surrounding Ash Pond A perimeter dikes were determined by the SC Department of Natural Resources (2012) to be of Pleistocene age. It was assumed that these soils are located beneath the recompacted dike fill soils, which are considered to be of Holocene age due to the relatively "recent" construction. Based on the range of soil ages presented in Table 4, an age correction factor of 1.30 was selected for Pleistocene-aged, foundation soils at WGS. Dike base elevations were approximated based on the surface elevation of borings or soundings located at the dike toe or the prevailing ground surface elevation of the Cooling Pond. Boring information and the top of foundation soil elevation (or dike base elevation) are summarized within Table 5 of this Liquefaction Package. An age correction factor of 1.00 was applied for dike fill soils, as these structures are approximately 30 to 40 years old. As noted previously, "geologic" age differs from "geotechnical" age. Geologic age refers to the overall age of the soil since deposition. Geotechnical age refers to the age of the soil since the last instance of liquefaction. The geotechnical age was considered in the selection of K_{DR}.

Fines Content

As shown in Equations 9 through 12, the Cyclic Resistance Ratio (CRR) is influenced by the fines content (% particles by mass passing a No. 200 sieve) of the soil interval. An increase in fines content of the soil results in higher resistance to liquefaction. As shown in the Attachment 5, fines content data of dike fill and foundation soils is somewhat variable across the Ash Pond A footprint. Physical samples are not collected during CPT soundings and historical soil boring logs with laboratory index testing are not currently available. As it is considered impractical to collect index testing (and fines content data) on every soil sample or soil interval, the index test data was applied to each CPT sounding based on the data collected from the nearest available soil boring with laboratory index testing. Index testing, when available, for soil borings were utilized for each individual SPT N-value. The source of the select fines content for each investigation point is summarized within Table 5.

Phreatic Surface

The phreatic surface through the perimeter dikes to the downstream toe of the dike at the time of this Liquefaction Package was developed for each individual boring or CPT sounding based on depth to water measurements, porewater pressure (u_o) signatures, and dissipation tests. Phreatic surface assumptions through the Ash Pond A perimeter dikes at the time of the boring (TOB) and at the time of analysis (TOA) for this calculation package are also summarized in Table 5.

Energy Calibration for SPT N-Values

As described in the Data Package, the subcontractor during Geosyntec's 2013 investigation, Soil Consultants, Inc. (SCI), reported that the automatic hammer on the utilized drilling rig had an energy ratio of 88%, which was independently evaluated within six months of the investigation.

RESULTS

The methodology discussed previously was applied within a MathCAD® algorithm similar to the spreadsheets presented in Idriss and Boulanger (2008). Computations were performed on soil borings and soundings located at the dike centerline. The factor of safety against liquefaction (FS_{Liq}) was computed at every depth interval where data was collected for soil test borings (in 2-ft or 5-ft intervals) and CPT soundings (in 0.16-ft intervals). The computed FS_{liq} for the soil borings and CPT soundings and the approximate base of the perimeter dike structure which was developed from top of sounding elevations or prevailing grade of the Cooling Pond are shown in Figures 2 through 6. Figure 2 shows SPT-116, CPT-137, and SPT-117, which are located in the southeast corner of the Ash Pond A immediately north of the divider dike. Subsequent figures depict calculation results for soil borings and CPT soundings positioned progressively in a counter-clockwise direction around the surface impoundment. Example calculations are provided within Appendix 1.

The liquefaction potential calculation results can be generally summarized as follows:

- The computed FS_{liq} in the southeast corner of the Ash Pond A adjacent to the Cooling Pond ranged between 1.4 and 1.6 (when not greater than 2.0) between elevations 5.0 ft and 13.0 ft NGVD29.

- The computed FS_{liq} from the northern perimeter dike adjacent to the Intake Canal were found to range between 1.7 and 1.8 (when not greater than 2.0) at elevations between 0.0 ft and 10.0 ft NGVD29.

- The computed FS_{liq} beneath the perimeter dikes in the southwestern corner of Ash Pond A were found to range between 1.5 and 1.8 (when not greater than 2.0) at elevations 5.0 to 15.0 ft NGVD29.

- The FS_{liq} computed from SPTs within soil borings and from CPT soundings were found to be generally consistent between investigation points adjacently located (i.e., lower FS_{liq} computed at SPT-116 and CPT-137 at similar elevations).

It is noted that other zones within the foundation soils were computed intermittently with FS_{liq} between 1.4 and 2.0 at elevations other than those listed above. No zones of liquefiable soils (FS_{liq} below 1.0) were indicated by the evaluation results.

CONCLUSIONS

Based the liquefaction potential computations presented within this calculation package, liquefiable soils were not observed in the dike fill soils (i.e., native soils recompacted to form impounding perimeter dikes) or foundation soils beneath the perimeter dikes of the Ash Pond A. Soil borings and CPT soundings advanced at the downstream toe of the Ash Pond A perimeter dikes were not evaluated within this calculation package and will be included during an evaluation of "Unstable Areas" for the Ash Pond A. Since liquefiable zones were not identified for borings and CPT soundings advanced through the perimeter dikes within the dike fill or foundation soils beneath the perimeter dikes, additional post-liquefaction stability and displacement analyses are not warranted for the Ash Pond A perimeter dikes at this time.

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TABLES

Table 1. Correction Factors for Interpretation of SPT for Liquefaction Potential Analysis (Idriss and Boulanger, 2008)

Factor	**Description**
Energy ratio	Energy measurements are required to determine the delivered energy ratios or to calibrate the specific equipment being used. The correction factor is then computed as $C_E = \frac{ER_m}{60}$ where ER_m is the measured energy ratio as a percentage of the theoretical maximum. Empirical estimates of C_E (for rod lengths of 10 m or more) involve considerable uncertainty, as reflected by the following ranges: Doughnut hammer $C_E = 0.5–1.0$ Safety hammer $C_E = 0.7–1.2$ Automatic triphammer $C_E = 0.8–1.3$ (Seed et al. 1984, Skempton 1986, NCEER 1997)
Borehole diameter	Borehole diameter of 65–115 mm $C_B = 1.0$ Borehole diameter of 150 mm $C_B = 1.05$ Borehole diameter of 200 mm $C_B = 1.15$ (Skempton 1986)
Rod length	Where the ER_m is based on rod lengths of 10 m or more, the ER delivered with shorter rod lengths may be smaller. Recommended values from Youd et al. (2001) are as follows: Rod length < 3 m $C_R = 0.75$ Rod length 3–4 m $C_R = 0.80$ Rod length 4–6 m $C_R = 0.85$ Rod length 6–10 m $C_R = 0.95$ Rod length 10–30 m $C_R = 1.00$
Sampler	Standard split spoon without room for liners (the inside diameter is a constant $1^3/_8$ in.), $C_S = 1.0$. Split-spoon sampler with room for liners but with the liners absent (this increases the inside diameter to $1^1/_2$ in. behind the driving shoe): $C_S = 1.1$ for $(N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100}$ for $10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3$ for $(N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Table 2. Summary of Representative Profiles for the Dike Centerline

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	31.4	2.5	40.3
7.5	60.3	7.5	98.7
12.5	94.6	12.5	140.9
17.5	130.7	17.0	176.0
22.5	163.1	20.5	205.8
27.5	192.5	24.5	238.9
32.5	211.7	29.5	269.7
37.5	224.6	34.5	291.5
42.5	226.7	39.5	305.4
47.5	235.3	44.5	314.2
52.5	316.9	49.5	327.4
60.0	391.9	54.5	417.6
70.0	523.4	62.0	525.1
80.0	582.9	72.0	631.5
90.0	659.9	82.0	735.0
100.0	782.6	92.0	827.3
-	-	102.0	908.2

Notes:

1. Profiles were developed in the Site Response Package provided as Attachment 6.
2. For calculation points located in between the depth intervals listed above, the average τ_{max} was linearly interpolated for liquefaction potential computations.
3. Profile 1 corresponds to the Ash Pond A perimeter dikes adjacent to the Intake and Discharge Canals; while, Profile 2 corresponds to the Ash Pond A perimeter dikes adjacent to the Cooling Pond.

Table 3. Susceptibility of Soil Deposits to Liquefaction during Strong Seismic Shaking (Youd and Perkins, 1978)

Type of deposit	**Distribution of cohesionless sediments in deposit**	**Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction**			
		< 500 years	**Holocene**	**Pleistocene**	**Pre-Pleistocene**
Continental					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plains	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebkha	Locally variable	High	Moderate	Low	Very low
Coastal zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach—high wave energy	Widespread	Moderate	Low	Very low	Very low
Beach—low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
Artificial fill					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 4. Age Correction Factor (K_{DR}) based on Soil Age

Soil Age, t (years)	K_{DR} [1]	K_{DR} [2]
126	1.19	1.08
546	1.30	1.20
5,038	1.46	1.38
10,000	1.51	1.44
450,000	1.79	1.75

Notes:

1. K_{DR} computed by SCDOT Geotechnical Design Manual (SCDOT, 2010), as provided in Equation 19a.
2. K_{DR} computed by Andrus et al (2008) as provided in Equation 19b.

Table 5. Summary of Borings and Soundings Analyzed for Liquefaction Potential

Borehole ID	Northing	Easting	Elevation	Dike Bottom Elevation	Dike Bottom Basis	GWT Elevation at TOB	GWT Depth at TOB	FC Basis	τ_{max} Profile
-	ft	ft	ft	ft	-	ft	ft	-	psf
CPT-137	547703.191	2505294.214	41.46	19.00	Cooling Pond	23.46	18.0	SPT-116	Profile 2
CPT-138	548155.774	2504838.203	41.21	26.84	CPT-139	24.41	16.8	SPT-117/118	Profile 1
CPT-139	548218.303	2504840.798	26.84	N/A	N/A	21.84	5.0	-	-
CPT-140	548288.662	2503958.006	40.90	27.86	CPT-141	26.90	14.0	SPT-118	Profile 1
CPT-141	548346.887	2503958.139	27.86	N/A	N/A	22.86	5.0	-	-
CPT-142	548376.683	2503360.939	41.38	27.86	CPT-141	26.38	15.0	SPT-119	Profile 1
CPT-143	548439.003	2503044.506	41.16	27.86	CPT-141	23.16	18.0	SPT-119	Profile 1
CPT-144	547405.679	2502965.016	40.56	24.71	CPT-145	22.56	18.0	SPT-120	Profile 1
CPT-145	547387.258	2502889.227	24.71	N/A	N/A	21.21	3.5	-	-
CPT-146	546487.647	2503169.548	40.21	27.48	CPT-148	22.11	18.1	SPT-120	Profile 1
CPT-147	546182.273	2503236.916	39.89	27.48	CPT-148	21.79	18.1	SPT-120	Profile 1
CPT-148	546166.928	2503176.896	27.48	N/A	N/A	21.98	5.5	-	-
CPT-228	547361.796	2505212.702	24.32	N/A	N/A	22.82	1.5	-	-
CPT-229	547899.588	2505442.330	21.11	N/A	N/A	20.61	0.5	-	-
CPT-229A	547931.644	2505420.761	38.72	19.0	Cooling Pond	20.72	18.0	SPT-117/118	Profile 2
SPT-116	547438.185	2505180.259	41.44	19.00	Cooling Pond	26.44	15.0	SPT-116	Profile 2
SPT-117	547997.578	2505412.560	39.74	19.00	Cooling Pond	21.54	18.2	SPT-117	Profile 2
SPT-118	548238.478	2504339.686	39.67	26.84	CPT-139	28.07	11.6	SPT-118	Profile 1
SPT-119	548265.398	2502982.728	42.72	24.71	CPT-145	24.32	18.4	SPT-119	Profile 1
SPT-120	546980.520	2503057.008	41.06	24.71	CPT-145	28.86	12.2	SPT-120	Profile 1
SPT-121	546076.868	2503720.319	40.82	27.48	CPT-148	29.32	11.5	SPT-121	Profile 1

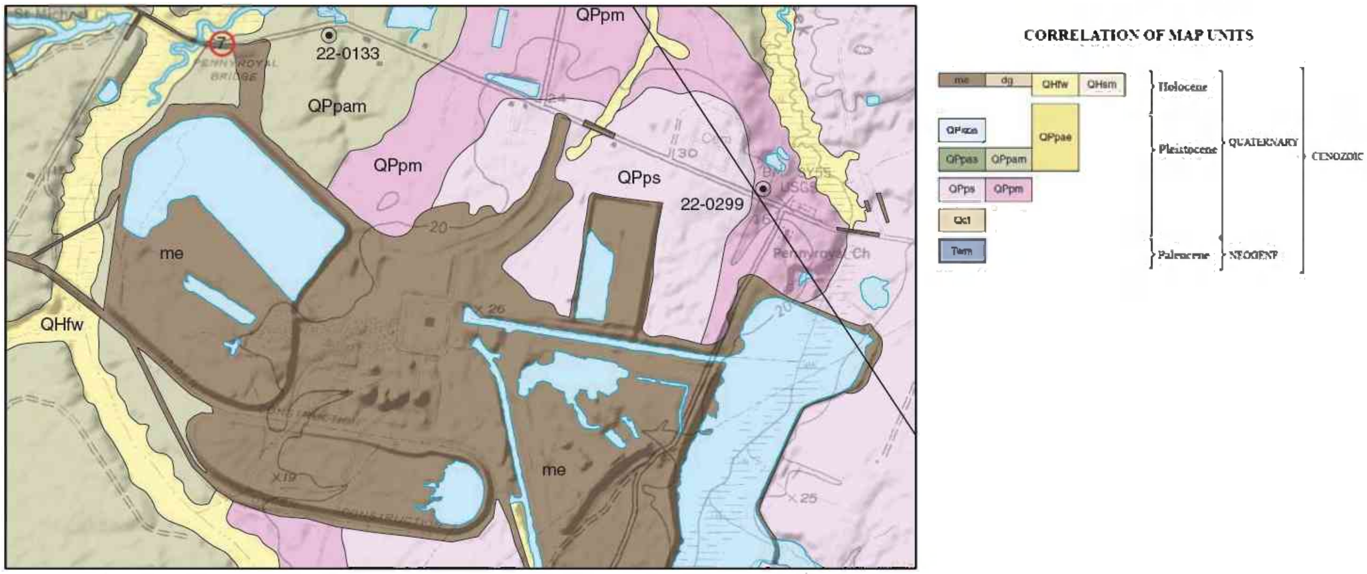
Notes:

1. ft NGVD29 - feet National Geodetic Vertical Datum of 1929; TOB - Time of Boring; GWT - Groundwater Table; FC - Fines Content; N/A = Not Applicable.
2. Dike bottom elevation was estimated based on the elevation of the nearest toe boring/sounding or the average Cooling Pond (19.0 ft NGVD29) prevailing surface elevation.
3. Elevations provided are in ft NGVD29.
4. Borings or CPT soundings performed in the interior of Ash Pond A (i.e., CPT-155 through CPT-158 or SPT-123) were not evaluated for liquefaction potential.
5. CPT-139, CPT-141, CPT-145, CPT-147, CPT-228, and CPT-229 are situated at the downstream perimeter dike toe and were not included as a part of this evaluation.
6. τ_{max} Profiles 1 and 2 correspond to the site response analysis for the perimeter dikes adjacent to the Discharge/Intake Canal and Cooling Pond, respectively.
7. FC Basis refers to the source of the fines content profile for each investigation point. Fines content data is provided within the Data Package (Attachment 5).

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **M. Zhu/G. Rix** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No.: **01BT**

FIGURES



QPpam **Estuarine deposits or marine deposits or both (Pleistocene) –** In its lower part, quartz and phosphatic sand, medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse grained with trace amounts of very fine grained heavy minerals. Lower part 10 to 20 feet thick. In upper part, muddy sand to sandy mud, clay, silt, silty sand, clayey sand, phosphatic sand and quartz sand and shells, medium light-gray (N6) to medium bluish-gray (5B 5/1). Some zones contain broken and intact *Oliva, Polinices, Terebra, Mercenaria and Dosinia*. Upper part 1 to 10 feet thick.

Figure 1. Geologic Map of Areas Surrounding Ash Pond A
(Map taken from SC Department of Natural Resources: Geological Survey, 2012)

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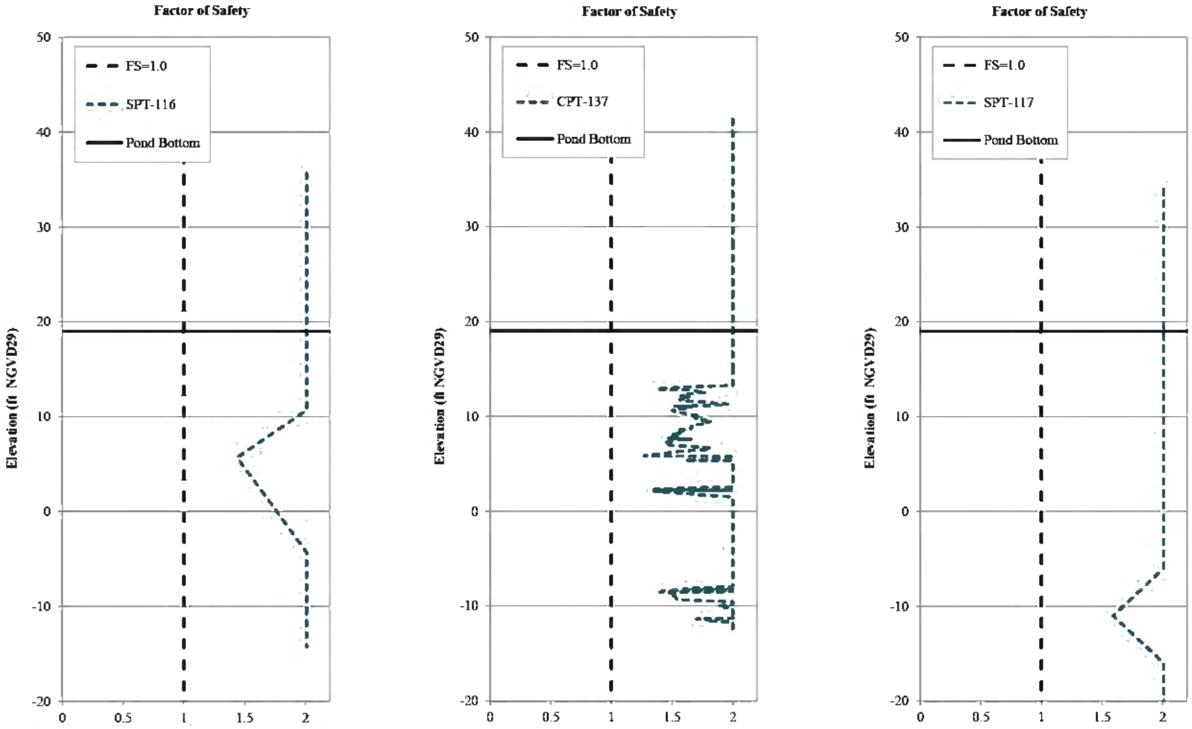

Figure 2. Liquefaction Results for Dike Fill and Foundation Soils for SPT-116, CPT-137, and SPT-117

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on average ground surface elevation within the Cooling Pond near the perimeter dikes, as provided in Table 5.

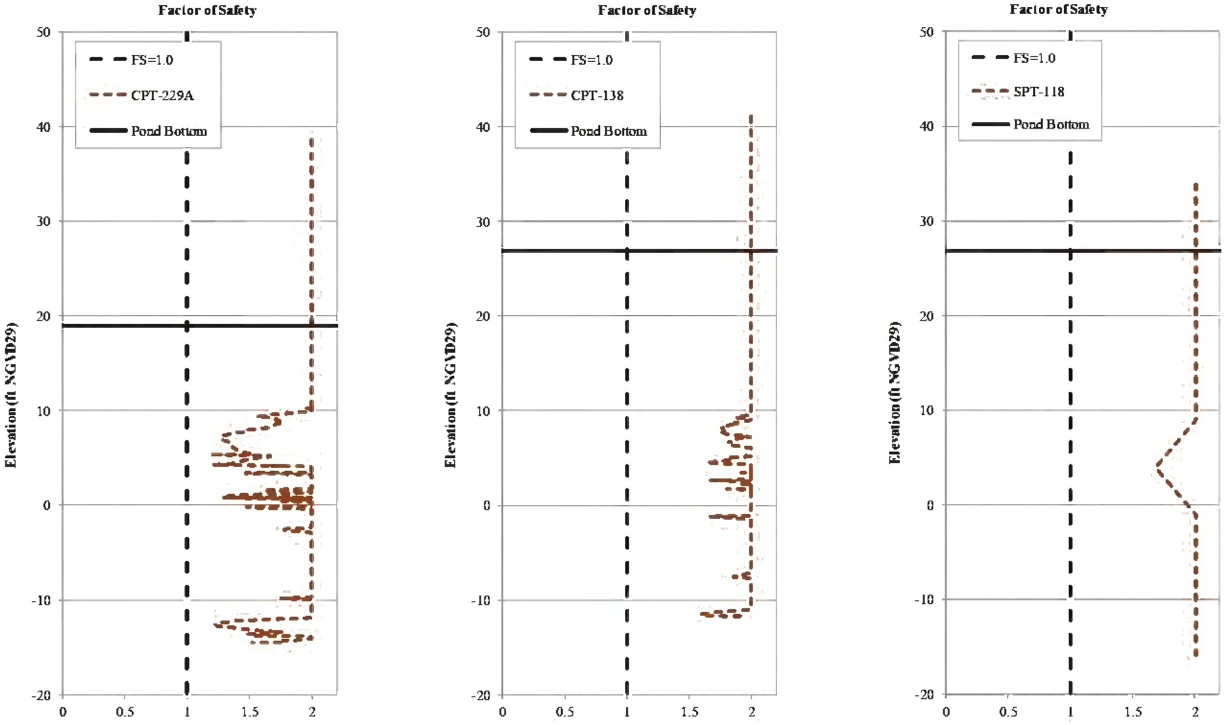

Figure 3. Liquefaction Results for Dike Fill and Foundation Soils for CPT-229A, CPT-138, and SPT-118

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the Cooling Pond (CPT-229A) and the surface elevations of toe soundings CPT-139 (CPT-138 and SPT-118), as provided in Table 5.

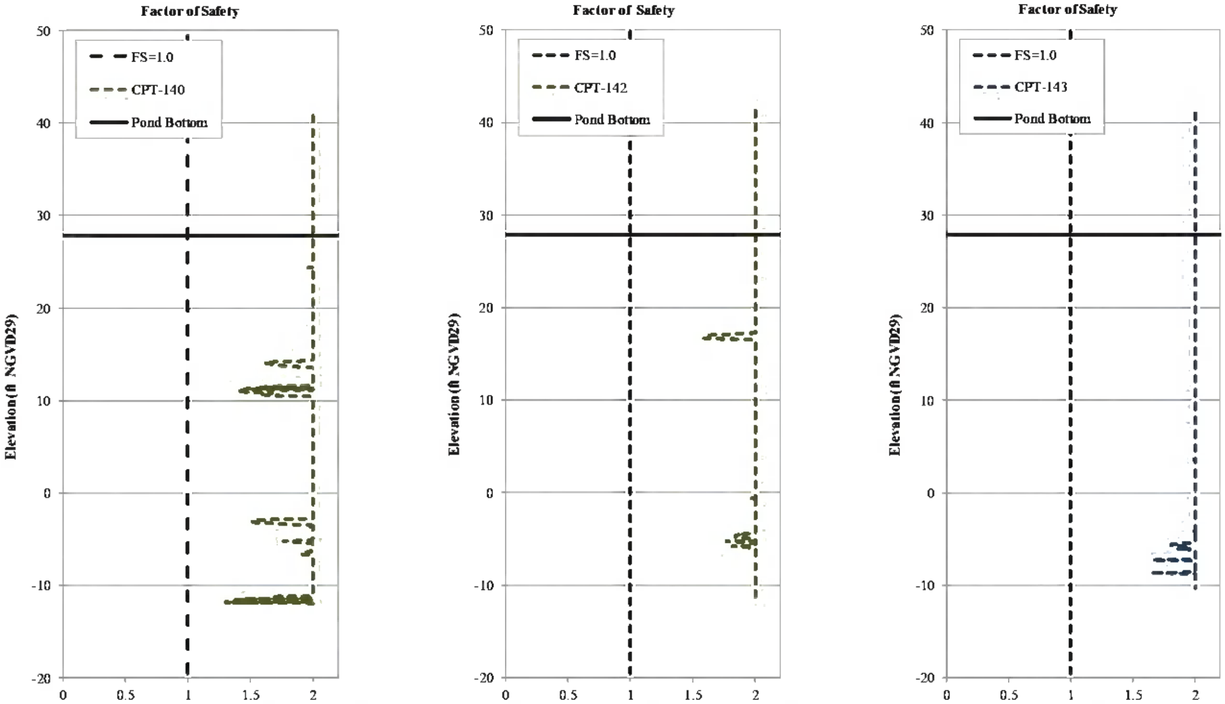

Figure 4. Liquefaction Results for Dike and Foundation Soils for CPT-140, CPT-142, and CPT-143

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevations of toe sounding CPT-141 as provided in Table 5.

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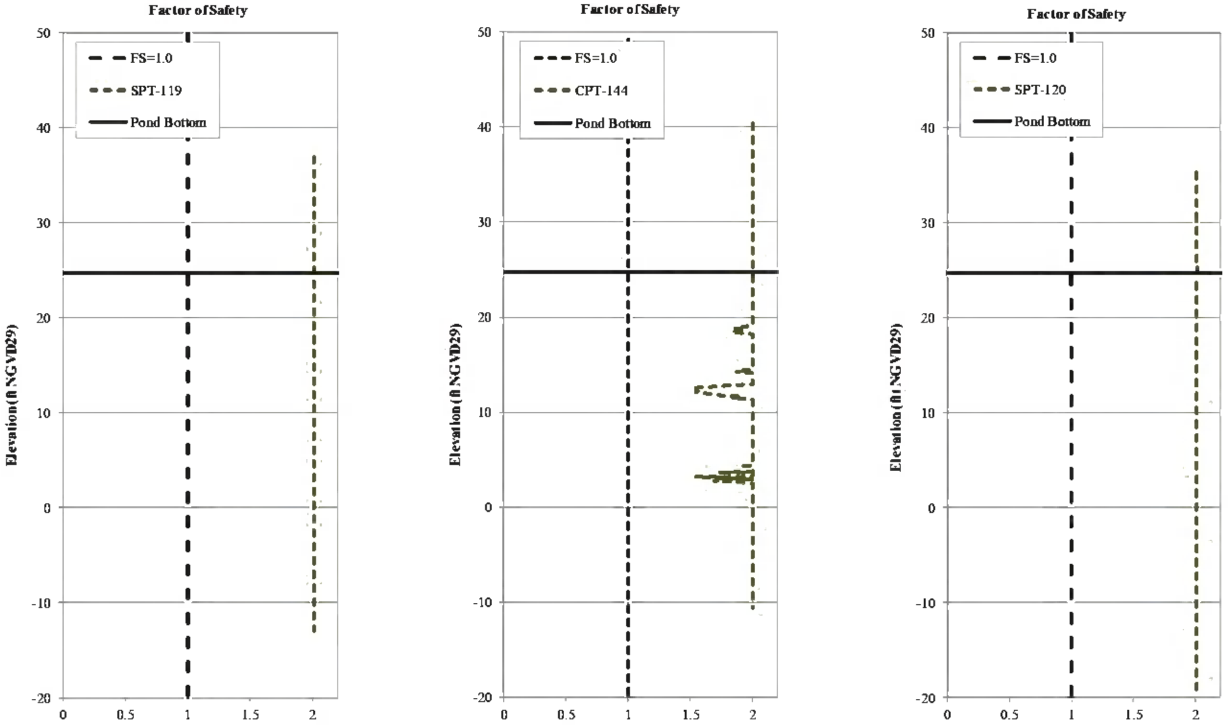

Figure 5. Liquefaction Results for Dike and Foundation Soils for SPT-119, CPT-144, and SPT-120

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of CPT-145, as provided in Table 5.

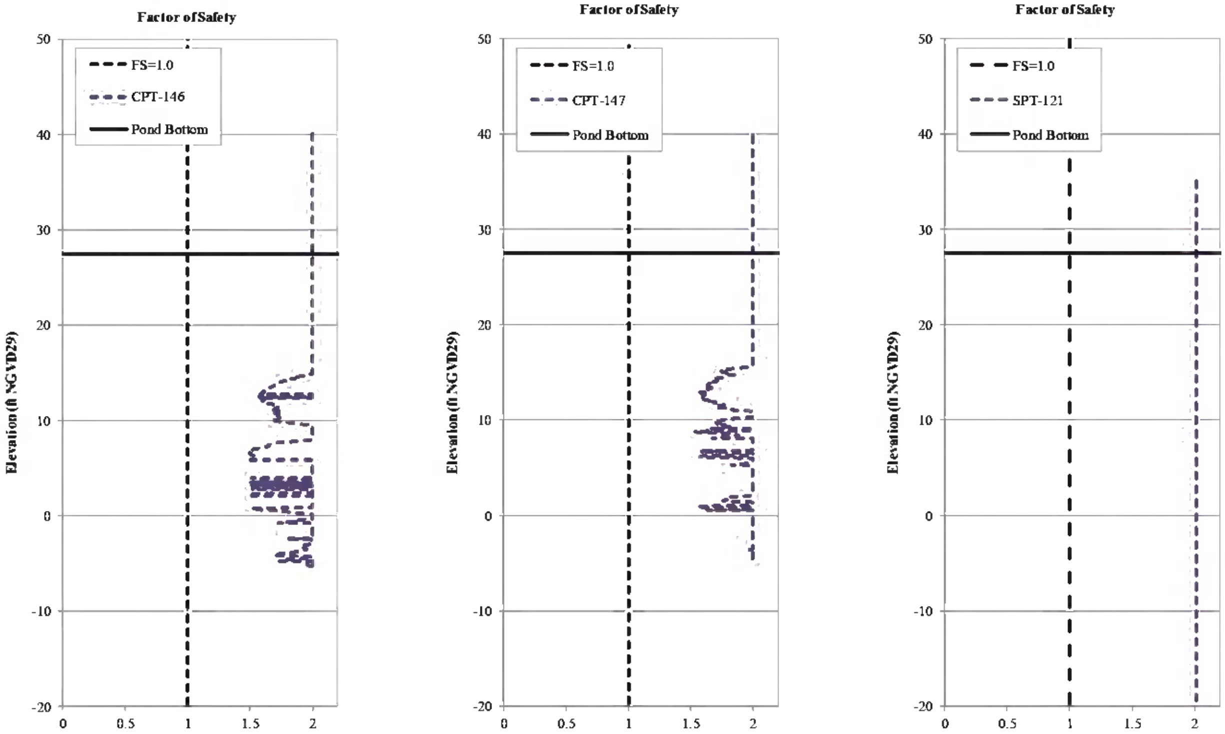

Figure 6. Liquefaction Results for Dike and Foundation Soils for CPT-146, CPT-147 and SPT-121

Note:

1. Foundation soils were assumed to begin at the dike bottom based on CPT-148 as provided in Table 5.

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Appendix 1

MathCAD® Example Calculation

CPT - based Liquefaction Analysis

$\text{BoringID} := \text{"CPT\_137"}$

Site Parameters:

Age Correction Factor of Pleistocene Soils: $K_{dr} := 1.3$

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $\text{Prof} := \text{"Profile2"}$

$$\text{CyclicStress} := \left| \begin{array}{l} \text{READEXCEL("APA\_Profile\_1.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile1"} \\ \text{READEXCEL("APA\_Profile\_2.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile2"} \end{array} \right.$$

Defining external units:

CPT-Specific data:

Import the CPT-Specific Data in the form of Depth, tip resistance (tsf), sleeve friction (tsf), porepressure (tsf), and fines content profile (%) with headers and units:

$\text{Full} := \text{READEXCEL}(\text{concat}(\text{BoringID}, \text{".xlsx"}))$

$\text{Data} := \text{submatrix}(\text{Full}, 2, \text{rows}(\text{Full}) - 1, 0, \text{cols}(\text{Full}) - 1)$

$\text{depth} := \text{Data}^{\langle 0 \rangle} \cdot \text{ft}$ $\quad \text{qc} := \text{Data}^{\langle 1 \rangle} \cdot \text{tsf}$ $\quad f_s := \text{Data}^{\langle 2 \rangle} \ \text{tsf}$ $\quad u_2 := \text{Data}^{\langle 3 \rangle} \ \text{tsf}$ $\quad \text{Fines} := \text{Data}^{\langle 4 \rangle}$

Simple counter used in the Algorithm: $i := 0 .. \text{rows}(\text{Data}) - 1$

Tip net area ratio: $a := 1$ 0.8 tip area ratio correction applied when converting Hogentogler Data (.cpt) to Excel (.xls) format.

Boring Information Inputs:

Boring Elevation: $\text{Elevation} := 41.46\text{ft}$ NGVD 29

Groundwater Depth at Time of Boring (TOB): $\text{GWT}_b := 18.00\text{ft}$bgs

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $\text{Elev}_h := 19.0\text{ft}$ NGVD 29

Sounding elevation profile: $\text{Elev} := \text{Elevation} - \text{depth}$

▼ Initial Total Unit Weight Assignments

Initial Unit Weight Estimates to be used with Robertson and Campanella (1983):

Adjust according to specific site conditions

1. Sand	$\gamma_1 := 115\text{pcf}$
2. Silty Sand	$\gamma_2 := 105\text{pcf}$
3. Sandy silt and silt	$\gamma_3 := 100\text{pcf}$
4. Silty clay/Clayey silt	$\gamma_4 := 90\text{pcf}$
5. Clay	$\gamma_5 := 90\text{pcf}$
Water	$\gamma_{water} := 62.4\text{pcf}$

Tip resistance back calculated from q_t and tip net area ratio a provided in the original data:

$$qt_i := qc_i - (1-a)\cdot u2_i$$

Average friction ratio: $Rf_i := \left(\frac{f_{s_i}}{qt_i}\right)\cdot 100\%$

▲ Initial Total Unit Weight Assignments

Robertson and Campanella 1983 Plot data:

▼ Extract Robertson (1983) plot lines based on values extracted from original plot:

sand-silty sand	S01 := submatrix(READPRN("Robertson1983.txt"), 0, 11, 0, 1)
silty sand-silts	S02 := submatrix(READPRN("Robertson1983.txt"), 0, 12, 2, 3)
silts-silty clay	S03 := submatrix(READPRN("Robertson1983.txt"), 0, 18, 4, 5)
clay	S04 := submatrix(READPRN("Robertson1983.txt"), 0, 19, 6, 7)

Linear interpolation used to evaluate Qt as a function of depth based on plot lines:

$$s01(x) := \text{linterp}\left(S01^{\langle 0\rangle}, S01^{\langle 1\rangle}, x\right) \qquad s02(x) := \text{linterp}\left(S02^{\langle 0\rangle}, S02^{\langle 1\rangle}, x\right)$$

$$s03(x) := \text{linterp}\left(S03^{\langle 0\rangle}, S03^{\langle 1\rangle}, x\right) \qquad s04(x) := \text{linterp}\left(S04^{\langle 0\rangle}, S04^{\langle 1\rangle}, x\right)$$

Rough estimate (initial guess) of unit weight based on Robertson 1983 soil classification:

```
$class_{1983}$ :=
    for $i \in 0 .. rows(qt) - 1$
        $class_i \leftarrow 5$
        $class_i \leftarrow 4$  if  $\frac{qt_i}{100 \cdot kPa} \ge s04(Rf_i)$
        $class_i \leftarrow 3$  if  $\frac{qt_i}{100 \cdot kPa} \ge s03(Rf_i)$
        $class_i \leftarrow 2$  if  $\frac{qt_i}{100 \cdot kPa} \ge s02(Rf_i)$
        $class_i \leftarrow 1$  if  $\frac{qt_i}{100 \cdot kPa} \ge s01(Rf_i)$
    class
```

```
$\gamma 1$ :=
    for $i \in 0 .. rows(qt) - 1$
        for $m \in 1 .. 5$
            $\gamma 1_i \leftarrow \gamma_m$  if  $class_{1983_i} = m$
    $\gamma 1$
```

▲ Extract Robertson (1983) plot lines based on values extracted from original plot:

Refined soil classification using Robertson and Cabal 2010:

▼ Calculation of Robertson (1990) plot parameters

Calculating Static Pore Pressures at time of Sounding:

$$u_{0_i} := \left| \begin{array}{l} (depth_i - GWT_b) \cdot \gamma_{water} \quad \text{if } depth_i > GWT_b \\ 0 \quad \text{otherwise} \end{array} \right.$$

Calculating Total and Effective Overburden Pressure

$$\sigma_{v0_i} := \left| \begin{array}{l} (depth_i - depth_{i-1}) \cdot \left(\frac{\gamma 1_i + \gamma 1_{i-1}}{2} \right) + \sigma_{v0_{i-1}} \quad \text{if } i > 0 \\ depth_i \cdot \gamma 1_0 \quad \text{otherwise} \end{array} \right.$$

$$\sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}} \cdot 100$$

Calculation of Robertson (1990) plot parameters

Unit weight values to be assigned to Robertson (1990) classification:

Unit weight adjusted to according to specific site conditions:

1. Sensitive, fine grained	$\gamma_1 := 85\text{pcf}$
2. Organic Soils-peat to Clay	$\gamma_2 := 100\text{pcf}$
3. Clay mixtures	$\gamma_3 := 100\text{pcf}$
4. Silt mixtures	$\gamma_4 := 100\text{pcf}$
5. Sand mixtures	$\gamma_5 := 110\text{pcf}$
6. Sands	$\gamma_6 := 120\text{pcf}$
7. Gravelly sand to sand	$\gamma_7 := 125\text{pcf}$
8. Very stiff sand to clayey sand	$\gamma_8 := 105\text{pcf}$
9. Very stiff fine grained	$\gamma_9 := 105\text{pcf}$

Refined soil classification and assigning unit weights

Compute Soil Behavior Index (I_c) corresponding to initial unit weight classification:

$$I_{c_i} := \left[\left(3.47 - \log\left(Q_{t_i}\right)\right)^2 + \left(\log\left(F_{r_i}\right) + 1.22\right)^2\right]^{.5}$$

Soil classification routine for Robertson (2010) (updated from Robertson, 1990) plot:

```
$\text{class}_{2010} :=$ for $i \in 0..\text{rows}(Q_t) - 1$
    $\text{class}_i \leftarrow 2$
    $\text{class}_i \leftarrow 3$ if $2.95 < I_{c_i} \le 3.6$
    $\text{class}_i \leftarrow 4$ if $2.60 < I_{c_i} \le 2.95$
    $\text{class}_i \leftarrow 5$ if $2.05 < I_{c_i} \le 2.60$
    $\text{class}_i \leftarrow 6$ if $1.31 < I_{c_i} \le 2.05$
    $\text{class}_i \leftarrow 7$ if $I_{c_i} \le 1.31$
  class
```

Assigning unit weight based on soil classification:

```
$\gamma_{\text{fin}} :=$ for $i \in 0..\text{rows}(Q_t) - 1$
    for $m \in 1..9$
      $\gamma2_i \leftarrow \gamma_m$ if $\text{class}_{2010_i} = m$
  $\gamma2$
```

▲ Refined soil classification and assigning unit weights

▼ Applying Robertson (2010) values for remaining calculations:

$\gamma := \gamma_{fin}$ $\qquad$ $\text{class} := \text{class}_{2010}$

Final Static Pore Pressure Calculation for CPT interpretation:

$$u_{0_i} := \begin{vmatrix} \left(\text{depth}_i - \text{GWT}_b\right)\cdot\gamma_{water} & \text{if depth}_i > \text{GWT}_b \\ 0 & \text{otherwise} \end{vmatrix}$$

Total and Effective Overburden Pressure Final Calculation for CPT interpretaiton:

$$\sigma_{v0_i} := \begin{vmatrix} \left(\text{depth}_i - \text{depth}_{i-1}\right)\cdot\left(\frac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} & \text{if } i > 0 \\ \text{depth}_0\cdot\gamma_0 & \text{otherwise} \end{vmatrix}$$

$$\sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}}\cdot 100$$

$$Q_i := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}}$$

Recompute Soil Behavior Index (I_c) corresponding to final unit weight classification:

$$I_{c_i} := \left[\left(3.47 - \log\left(Q_{t_i}\right)\right)^2 + \left(\log\left(F_{r_i}\right) + 1.22\right)^2\right]^{.5}$$

▲ Applying Robertson (2010) values for remaining calculations:

Corrected Normalized CPT Sounding:

▼ Overburden corrected tip resistance calculations

Overburden corrected tip resistance:

```
$q_{c1\_it} :=$
  $c \leftarrow 0$
  "initial CN"
  for $i \in 0..rows(qt) - 1$
    $C_{N_i} \leftarrow 1.7$
  for $i \in 0..rows(qt) - 1$
    while $c < 500$
      $q_{c1_i} \leftarrow C_{N_i} \cdot qt_i$
      $q_{c1N_i} \leftarrow \frac{q_{c1_i}}{1atm}$
      $C_{N_i} \leftarrow \min\left[1.7, \left(\frac{1atm}{\sigma_{v0eff_i}}\right)^{1.338-0.249\cdot\left(\max\left(21,\min\left(q_{c1N_i},254\right)\right)\right)^{0.264}}\right]$
      $c \leftarrow c + 1$
    $c \leftarrow 0$
  $\begin{pmatrix} \frac{q_{c1}}{psf} & q_{c1N} \end{pmatrix}$
```

$$q_{c1} := \left(q_{c1\_it}{}^{\langle 0 \rangle}\right)_0 psf \qquad q_{c1N} := \left(q_{c1\_it}{}^{\langle 1 \rangle}\right)_0$$

▲ Overburden corrected tip resistance calculations

▼ Compute CRR with Overburden, MSF, and Kdr Corrections

<u>Compute the CRR (M_w=7.5, 1 atm) based on the CPT values:</u>

Cyclic Resistance Ratio (CRR):

$i := 0..rows(qc) - 1$

Correction factor for soils with fines:

$$\Delta q_{c1N_i} := \left(5.4 + \frac{q_{c1N_i}}{16}\right)\cdot \exp\left[1.63 + \frac{9.7}{Fines_i + 0.01} - \left(\frac{15.7}{Fines_i + 0.01}\right)^2\right]$$

Equivalent clean sand corrected tip resistance: $q_{c1Ncs_i} := q_{c1N_i} + \Delta q_{c1N_i}$

$$CRR_i := \left| \begin{array}{l} \exp\left[\frac{q_{c1Ncs_i}}{540} + \left(\frac{q_{c1Ncs_i}}{67}\right)^2 - \left(\frac{q_{c1Ncs_i}}{80}\right)^3 + \left(\frac{q_{c1Ncs_i}}{114}\right)^4 - 3\right] \text{ if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} < 211 \\ 2.0 \text{ if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} > 211 \\ 2.0 \text{ otherwise} \end{array} \right.$$

Overburden Correction Factor (Kσ) for Sands:

$$C_{\sigma_i} := \min\left[\frac{1}{37.3 - 8.27\cdot\left(\min\left(q_{c1N_i}, 211\right)\right)^{0.264}}, 0.3\right]$$

$$K_{\sigma_i} := \left| \begin{array}{l} \min\left(1 - C_{\sigma_i}\cdot\ln\left(\frac{\sigma_{v0eff_i}}{1\text{tsf}}\right), 1.1\right) \text{ if } I_{c_i} \le 2.60 \\ 1.0 \text{ otherwise} \end{array} \right.$$

Corrected CRR: $CRR1_i := CRR_i \cdot K_{\sigma_i}$

Magnitude Scaling Factor (MSF) [SCDOT 2010, pg. 13-44]:

MSF is dependent on material type and for cyclic softening calculations, two MSF correlations are applicable.

$$MSF_i := \min(1.80, 6.9\cdot\exp(-0.25\cdot M) - 0.058)$$

$$CRR2_i := CRR1_i \cdot MSF_i$$

Adjust CRR for Age Correction Factor for Pleistocene Sands [SCDOT, 2010 - pg. 13-60 & 13-61]:

K_{dr} is only applicbale for Sands that are of Pleistocene-Age or older.

$$CRR_{final_i} := \left| \begin{array}{l} CRR2_i \cdot K_{dr} \text{ if } I_{c_i} \le 2.60 \wedge Elev_i < Elev_h \\ CRR2_i \text{ otherwise} \end{array} \right.$$

▲ Compute CRR with Overburden, MSF, and Kdr Corrections

▼ Compute CSR and FS

Compute the CSR for the Soil Profile

$$\tau_{cyc} := \text{submatrix}(\text{CyclicStress}, 1, \text{rows}(\text{CyclicStress}) - 1, 1, 1)\,\text{psf}$$

$d_{cyc} := \text{submatrix}(\text{CyclicStress}, 1, \text{rows}(\text{CyclicStress}) - 1, 0, 0)\ \text{ft}$

$\tau_{max} := \text{linterp}\left(d_{cyc}, \tau_{cyc}, \text{depth}\right)$

$$CSR_i := \frac{0.65\,\tau_{max_i}}{\sigma_{v0eff_i}}$$

Compute Factor of Safety

$$FS_i := \left| \begin{array}{l} 2.00 \quad \text{if } \ \text{depth}_i < GWT_b \\ \min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.00\right) \quad \text{otherwise} \end{array} \right.$$

Compute CSR and FS

Export Results:

Headers := augment("Depth" , "Elevation" , "qc1N" , "SBT Index" , "FScyclic")

Units := augment("ft" , "ft NGVD29" , "-" , "-" , "-")

$$\text{Export} := \text{augment}\left(\frac{\text{depth}}{\text{ft}}, \frac{\text{Elev}}{\text{ft}}, q_{c1N}, I_c, FS\right)$$

Export2 := stack(Headers, Units, Export)

FileName := concat(BoringID, "_Results" , ".xlsx")

Export3 := WRITEEXCEL(Export2, FileName)

SPT - based Liquefaction Analysis

BoringID := "SPT-116"

Site Parameters:

Age Correction Factor: $K_{dr} := 1.3$ (Geosyntec, 2013)

Earthquake Magnitude: $M := 7.3$

Site Response Profile: Prof := "Profile2"

$$\text{CyclicStress} := \left| \begin{array}{l} \text{READEXCEL("APA\_Profile\_1.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile1"} \\ \text{READEXCEL("APA\_Profile\_2.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile2"} \end{array} \right.$$

SPT-Specific data:

Import the SPT-Specific Data in the form of Depth, Blow Count, Visual Classification ("Sand-Like"/"Clay-Like"), fines content, and USCS Classification. Upper two rows contain the headers and units for each field:

Full := READEXCEL(concat(BoringID, ".xlsx"))

Data := submatrix(Full, 2, rows(Full) − 1, 0, cols(Full) − 1)

$\text{depth} := \text{Data}^{\langle 0 \rangle} \cdot \text{ft}$ $\quad N_{blows} := \text{Data}^{\langle 1 \rangle}$ $\quad \text{Class} := \text{Data}^{\langle 2 \rangle}$ $\quad \text{Fines} := \text{Data}^{\langle 3 \rangle}$ $\quad \text{USCS} := \text{Data}^{\langle 4 \rangle}$

Boring Information:

Boring Elevation: Elevation := 41.44ft NGVD29

Groundwater Depth: GWT := 15.0ft bgs

Boring Diameter: Diameter := 4 inches

Holocene Elevation: $\text{Elev}_h := 19.00\text{ft}$ NGVD29

Energy Calibration: ER := 88 % (SCI, 2014)

Sampling Method: $C_S := 1.0$

RodDepth := depth + 5ft (Assume 5 ft of rod stick up during SPT test)

Miscelaneous Constants:

Defining external units: $\text{tsf} := \dfrac{\text{tonf}}{\text{ft}^2}$ $\quad \text{kPa} := \dfrac{1}{95.760518}\text{tsf}$

▼ Compute Calibration Factors and N60

Compute Calibration Factors

$$C_E := \frac{ER}{60}$$

```
$C_B :=$
    $1.0$  if  $Diameter \le 4.0$
    $1.05$  if  $4.0 < Diameter < 6.0$
    $1.15$  otherwise
```

```
$C_R :=$
    for  $i \in 0 .. rows(depth) - 1$
        $rod_i \leftarrow 0.75$  if  $RodDepth_i \le 13ft$
        $rod_i \leftarrow 0.85$  if  $13ft < RodDepth_i \le 20ft$
        $rod_i \leftarrow 0.95$  if  $20ft < RodDepth_i \le 33ft$
        $rod_i \leftarrow 1$  otherwise
    $rod$
```

Compute N_{60}:

```
$N_{60} :=$
    for  $i \in 0 .. rows(depth) - 1$
        $x_i \leftarrow C_B \cdot C_E \cdot C_S \cdot N_{blows_i} \cdot C_{R_i}$
    $x$
```

▲ Compute Calibration Factors and N60

▼ Calculation of CN and Effective Overburden Stress

Compute C_N:

Develop Representative Unit Weight Profile

Unit weight values to be assigned based on density and material class:

Adjust according to specific site conditions

1. Coal Combustion Residuals	$\gamma_1 := 100pcf$
2. Loose Sands (N_{blows} <10)	$\gamma_2 := 105pcf$
3. Medium Dense Sands (10<N_{blows} <30)	$\gamma_3 := 115pcf$
4. Dense Sands	$\gamma_4 := 120pcf$
5. Soft Clays	$\gamma_5 := 100pcf$
6. Chicora Member	$\gamma_6 := 130pcf$
7. Williamsburg Formation Clay	$\gamma_7 := 105pcf$

Relate depth to elevation to screen unit weights for Williamsburg Formation Clay

$Elev := (Elevation - depth)$ $\qquad$ $WMElev := -8ft$ $\qquad$ (Approx. Top of Williamsburg Formation Clay)

```
$Class_2 :=$ for $i \in 0..rows(depth) - 1$
                 $y_i \leftarrow 7$
                 $y_i \leftarrow 1$ if $Class_i = "SILT"$
                 $y_i \leftarrow 2$ if $Class_i = "SAND" \wedge N_{blows_i} \le 10$
                 $y_i \leftarrow 3$ if $Class_i = "SAND" \wedge 10 < N_{blows_i} < 30$
                 $y_i \leftarrow 4$ if $Class_i = "SAND" \wedge 30 < N_{blows_i}$
                 $y_i \leftarrow 5$ if $Class_i = "CLAY" \wedge Elev_i > WMElev$
                 $y_i \leftarrow 6$ if $Class_i = "CHICORA"$
             $y$
```

Assign unit weight based on soil classification:

```
$\gamma_{fin} :=$ for $i \in 0..rows(depth) - 1$
                    for $m \in 1..7$
                      $\gamma2_i \leftarrow \gamma_m$ if $Class_{2_i} = m$
                  $\gamma2$
```

$\gamma := \gamma_{fin}$ $\qquad$ $\gamma_{water} := 62.4pcf$

$i := 0..rows(depth) - 1$

Final Static Pore Pressure Calculation:

$$u_{0_i} := \begin{cases} \left(depth_i - GWT\right)\cdot\gamma_{water} & \text{if } depth_i > GWT \\ 0 & \text{otherwise} \end{cases}$$

Total and Effective Overburden Pressure Final Calculation:

$$\sigma_{v0_i} := \begin{cases} \left(depth_i - depth_{i-1}\right)\cdot\left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} & \text{if } i > 0 \\ depth_0\cdot\gamma_0 & \text{otherwise} \end{cases}$$

$$\sigma_{v0eff} := \sigma_{v0} - u_0$$

Calculation of C_{NL} (For Liquefaction) [SCDOT, 2010 - pg. 13-48] Calulation limitted to a maximum N-value = 46 blows/ft)

```
$C_{NLit} :=$
    $c \leftarrow 0$
    "initial CN"
    for  $i \in 0 .. \text{rows}(\text{depth}) - 1$
      $C_{N_i} \leftarrow 1.7$
    for  $i \in 0 .. \text{rows}(\text{depth}) - 1$
        while  $c < 600$
            $N_{160L_i} \leftarrow C_{N_i} \cdot N_{60_i}$
            $C_{N_i} \leftarrow \min\left[1.7, \left(\frac{1\text{atm}}{\sigma_{v0eff_i}}\right)^{\left(0.784 - 0.0768 \cdot \sqrt{\min\left(46, N_{160L_i}\right)}\right)}\right]$
            $c \leftarrow c + 1$
        $c \leftarrow 0$
    $\left(C_N \;\; N_{160L}\right)$
```

$$C_{NL} := \left(C_{NLit}{}^{\langle 0 \rangle}\right)_0 \qquad\qquad N_{160_i} := C_{NL_i} \cdot N_{60_i}$$

▲ Calculation of CN and Effective Overburden Stress

▼ Compute N160 for Liquefaction

Compute $(N_1)_{60\text{-}L}$ (For Liquefaction):

Correct N_{160} for influence of fines [SCDOT, 2010 pg. 13-51]

```
$\Delta N_{160L} :=$
    for  $i \in 0 .. \text{rows}(\text{depth}) - 1$
      $x_i \leftarrow \min\left[5.5, \exp\left[1.63 + \left[\frac{9.7}{\left(\text{Fines}_i + 0.01\right)}\right] - \left[\frac{15.7}{\left(\text{Fines}_i + 0.01\right)}\right]^2\right]\right]$
    $x$
```

$$N_{160cs_i} := N_{160_i} + \Delta N_{160L_i}$$

▲ Compute N160 for Liquefaction

▼ Compute CRR with Overburden, MSF, and Kdr Corrections

Compute the CRR (M_w=7.5, 1 atm) based on the SPT values [SCDOT, 2010 - pg. 13-54 & 13-55 - and is consistent with Idriss and Boulanger 2008]:

$$CRR1_i := \exp\left[\left(\frac{N_{160cs_i}}{14.1}\right) + \left(\frac{N_{160cs_i}}{126}\right)^2 - \left(\frac{N_{160cs_i}}{23.6}\right)^3 + \left(\frac{N_{160cs_i}}{25.4}\right)^4 - 2.8\right]$$

Overburden Correction Factor (Kσ):

$$C_{\sigma_i} := \min\left[\frac{1}{18.9 - 2.55\cdot\left(\min\left(N_{160_i}, 37\right)\right)^{0.50}}, 0.3\right]$$

$$K_{\sigma_i} := \min\left(1 - C_{\sigma_i}\cdot\ln\left(\frac{\sigma_{v0eff_i}}{2117psf}\right), 1.1\right)$$

$$CRR2_i := CRR1_i\cdot K_{\sigma_i}$$

Magnitude Scaling Factor (MSF) [SCDOT 2010, pg. 13-44]:

MSF is dependent on material type and for cyclic softening calculations, two MSF correlations are applicable.

$$MSF_i := \min(1.80, 6.9\cdot\exp(-0.25\cdot M) - 0.058)$$

$$CRR3_i := CRR2_i\cdot MSF_i$$

Adjust CRR for Age Correction Factor for Pleistocene Sands [SCDOT, 2010 - pg. 13-60 & 13-61]:

K_{dr} is only applicbale for Sands that are of Pleistocene-Age (assumed to be below bottom of dike fill):

$$CRR_{final_i} := \begin{cases} CRR3_i\cdot K_{dr} & \text{if } Class_i = \text{"SAND"} \wedge Elev_i < Elev_h \\ CRR3_i & \text{otherwise} \end{cases}$$

▲ Compute CRR with Overburden, MSF, and Kdr Corrections

▼ Compute CSR and FS

Compute the CSR for the Soil Profile

$\tau_{cyc} := \text{submatrix}(\text{CyclicStress}, 1, \text{rows}(\text{CyclicStress}) - 1, 1, 1)\,\text{psf}$

$d_{cyc} := \text{submatrix}(\text{CyclicStress}, 1, \text{rows}(\text{CyclicStress}) - 1, 0, 0)\,\text{ft}$

$\tau_{max} := \text{linterp}(d_{cyc}, \tau_{cyc}, \text{depth})$

$$CSR_i := \frac{0.65\,\tau_{max_i}}{\sigma_{v0eff_i}}$$

Compute Factor of Safety

$$FS_i := \left|\begin{array}{ll} 2.01 \text{ if } Class_i = \text{"CHICORA"} \\ 2.01 \text{ if } depth_i < GWT \\ 2.01 \text{ if } Class_i = \text{"CLAY"} \\ \min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.01\right) \text{ otherwise} \end{array}\right.$$

-Assume Chicora statum does NOT Liquefy

▲ Compute CSR and FS

▶ Evaluate the Soil Strength Loss (SSL) due to Cyclic Liquefaction or Cyclic Softening:

Export Results:

Headers := augment("Depth" , "Elevation" , "N160" , "Class" , "FScyclic")

Units := augment("ft" , "ft NGVD29" , "-" , "-" , "-")

$$Export := \text{augment}\left(\frac{depth}{ft}, \frac{Elev}{ft}, N_{160}, Class, FS\right)$$

Export2 := stack(Headers, Units, Export)

FileName := concat(BoringID, "_Results" , ".xlsx")

Export3 := WRITEEXCEL(Export2, FileName)

Export Liquefaction Results

ATTACHMENT 8

Safety Factor Assessment

Geosyntec
consultants

CALCULATION PACKAGE COVER SHEET

Client: Santee Cooper **Project:** Winyah Generating Station **Project No.** GSC5242

TITLE OF PACKAGE: SAFETY FACTOR ASSESSMENT: ASH POND A

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Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

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Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**

SAFETY FACTOR ASSESSMENT: ASH POND A

INTRODUCTION

This calculation package was prepared as Attachment 8 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* (Safety Factor Assessment Report) and presents the slope stability analyses for the Ash Pond A perimeter dikes at Winyah Generating Station (WGS). Ash Pond A is a 90-acre surface impoundment, which manages coal combustion residuals (CCR) in the form of fly ash, boiler slag, and bottom ash produced as by-products during electric generating activities. On 17 April 2015, the United States Environmental Protection Agency (USEPA) published the CCR Rule (40 Code of Federal Regulations [CFR] Parts 257 and 261). Under the CCR Rule, Ash Pond A is classified as an "existing surface impoundment" and must meet specific requirements with respect to periodic safety factor assessments. This calculation package presents the slope stability analysis performed as a part of the periodic safety factor assessment required by §257.73(e)(1) for existing CCR surface impoundments. The remainder of this calculation package presents: (i) safety factor criteria; (ii) methodology; (iii) cross section geometry; (iv) engineering parameters; (v) results; (vi) conclusions; and (vii) references.

SAFETY FACTOR CRITERIA

Slope stability analyses were conducted to assess whether the Ash Pond A perimeter dikes achieve the safety factor (also referred to as "factor of safety") criteria described within §257.73(e)(1) of the CCR Rule. §257.73(e)(1) requires that:

> *(i) "The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.*
> *(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*
> *(iii) The calculated seismic factor of safety must equal or exceed 1.00.*
> *(iv) For embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20."*

It is noted that the liquefaction potential analysis results presented in Attachment 7: *Liquefaction Potential Analysis: Ash Pond A* (Liquefaction Package) of this Safety Factor Assessment Report did not indicate that the Ash Pond A dike fill or foundation soils immediately beneath the perimeter dikes are susceptible to liquefaction. Therefore, the liquefaction factor of safety (FS) for the Ash Pond A perimeter dikes utilizing post-liquefaction residual shear strengths was not evaluated as a part of this safety factor assessment.

METHODOLOGY

Static Slope Stability

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE®, version 6.037 (Rocscience, 2015). Spencer's method, which satisfies vertical and horizontal force equilibrium as well as moment equilibrium, is considered to be more rigorous than other methods, such as the simplified Janbu method (Janbu, 1973) and the simplified Bishop method (Bishop, 1955).

Both rotational mode (i.e., the circular slip surface mode) and non-rotational (i.e., the block slip surface mode) were considered during these analyses, and the slip mechanism resulting in the lowest calculated FS is reported. SLIDE® generates potential slip surfaces, calculates the FS for each of these surfaces, and identifies the critical slip surface with the lowest calculated FS. Information required for these analyses include the slope geometry, the subsurface soil stratigraphy, the phreatic surface elevation, the external loading conditions, and the engineering properties of subsurface materials.

Seismic Slope Stability

Pseudo-static slope stability analyses were performed to evaluate the seismic performance of the perimeter dike structures using a procedure consistent with Hynes-Griffin and Franklin (1984). The procedure is described as follows:

1. Estimate the maximum horizontal equivalent acceleration (MHEA) for the potential critical slip surfaces of the perimeter dike system based on results from the site response analyses presented in Attachment 6: *Seismic Hazard Evaluation and Site Response Analysis: Ash Pond A* (Site Response Package) of this Safety Factor Assessment Report.

2. Compute the seismic horizontal force coefficient (k_h) using the ratio of the critical acceleration (N) to the peak value of earthquake acceleration (A) based on an allowable displacement (u) in which the perimeter dikes are considered stable (from Figure 6 of Hynes-Griffin and Franklin [1984]). The critical acceleration, N, was selected as the k_h for the purposes of this analysis and the MHEA at the depth of the critical slip surface was selected as the peak earthquake acceleration, A.

3. Perform slope stability analysis applying the seismic horizontal force coefficient to compute a horizontal force ($F = k_h \times W$), on each slice based on slice weight (W), and evaluate the resulting FS. If the calculated FS meets or exceeds the target FS (i.e., $FS \geq 1.0$), the slope is considered to be stable and to meet the requirements of the CCR Rule.

It is noted that during pseudo-static slope stability analyses, undrained shear strengths should be reduced by 20 percent to account for potential strength degradation during cyclic loading.

CROSS SECTION GEOMETRY

The following section describes the development for the: (i) external geometry; (ii) subsurface stratigraphy; and (iii) water levels and phreatic surface for the cross sections evaluated as a part of this safety factor assessment.

External Geometry

The height of the Ash Pond A perimeter dikes is approximately 15 to 20 feet (ft) adjacent to the Intake and Discharge Canals and approximately 25 ft adjacent to the Cooling Pond. The upstream and downstream side slopes range from 2 horizontal to 1 vertical (2H:1V) in the west to 3H:1V adjacent to the Cooling Pond in the east, while the dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012).

Five cross sections were developed and evaluated as a part of this safety factor assessment. These cross sections were selected based on the critical slope geometry, engineering parameters of subsurface materials, and phreatic conditions. Cross sections were also selected to evaluate at least one cross section for each side of the Ash Pond A perimeter dikes. The external geometry of each cross section was based on a topographic survey prepared by Thomas and Hutton (2012) and a limited bathymetric survey within the Cooling Pond at the downstream toe of the perimeter dikes. Parker Land Surveying, LLC visited WGS in November 2015 to collect survey transects within the Cooling Pond at the base of the Ash Pond A perimeter dikes where the Cooling Pond appeared to be the deepest based on aerial photography and prior site visit observations. Contours of the topographic survey were modeled as a triangular-irregular-network (TIN) surface within the computer program AutoCAD® and the surveyed transects within the Cooling Pond were subsequently incorporated into the TIN surface. The five cross sections (Cross Section A through Cross Section E) were developed within AutoCAD® and exported directly into the SLIDE® program. The location and extent of each analyzed cross section are depicted in Figure 1.

Subsurface Stratigraphy

The subsurface stratigraphy for each cross section was developed based on soil borings and cone penetration tests (CPTs) conducted as a part of Geosyntec's 2013 subsurface investigation. Generally, the subsurface in the depth of interest for slope stability analyses consists of the following strata (from top to bottom): Dike Fill, Foundation Soils, Chicora Member, and Williamsburg Formation Clay. Further discussion on the development of subsurface conditions can be found in Attachment 5: *Subsurface Stratigraphy and Material Properties: Ash Pond A* (Data Package) of the Safety Factor Assessment Report.

Water Levels:

The CCR Rule requires the evaluation of safety factors considering static slope stability analysis for long-term "Maximum Normal Storage Pool" conditions and long-term "Maximum Surcharge Pool" conditions, and seismic slope stability analyses for "Maximum Normal Storage Pool" conditions. As described within the Hydrologic and Hydraulic (H&H) analysis for Ash Pond A provided in Attachment 1 of this Safety Factor Assessment Report, the water level within Ash Pond B, located to the south of Ash Pond A, is maintained at an elevation of 34.9 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a 4-ft by 4-ft concrete riser structure. Ash Pond A does not contain free water, but conveys stormwater and process water through a series of rim ditches to Ash Pond B. A 30-in. diameter corrugated metal pipe (CMP), a 48-in. diameter smooth steel pipe, and a 42-in. diameter smooth steel pipe convey free water from the rim ditches through the northeast corner of divider dike into Ash Pond B. The concrete riser structure in Ash Pond B discharges free water westward into the Discharge Canal. An operating water level of 34.9 ft NGVD29 was selected as the "Maximum Normal Storage Pool" within Ash Pond A based on the invert elevation of the concrete riser structure in Ash Pond B. Since Ash Pond A is considered a "Low Hazard Potential" surface impoundment (Geosyntec, 2016), the 100-yr rainfall event was selected as the Inflow Design Flood (IDF), as required by §257.73(d)(1)(B). The maximum free water elevation within Ash Pond A was computed as 38.2 ft NGVD29, which was selected as the "Maximum Surcharge Pool" for this safety factor assessment.

The phreatic surface through the perimeter dikes to the downstream toe at the time of this factor of safety assessment was predominantly developed based on water levels collected from CPT sounding dissipation tests, supplemental monitoring wells installed within the perimeter dikes, depth to water measurements within boreholes, and the Cooling Pond free water elevation. Temporary piezometers installed within the Ash Pond A shows that the phreatic surface elevation within the residual fly ash at the center of Ash Pond A has ranged between 36.0 and 37.2 ft NGVD29 during recent operations. Thus, the phreatic surface within the center of the surface pond was selected as 37.2 ft NGVD29 and assumed to transition to 34.9 ft NGVD29 adjacent to the perimeter dikes. The water level of the Cooling Pond was similarly selected as 19.13 ft NGVD29 based on the operating pool level of the Cooling Pond required to manage the 25-yr, 24-hr rainfall event and from free water elevation measurements. In both the "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions, the phreatic surface through the Ash Pond A perimeter dikes was assumed to reach steady-state conditions.

Final Cross Section Geometry

The final geometric models during "Maximum Normal Storage Pool" conditions as implemented within SLIDE® for Cross Sections A through E are provided in Figures 2 through 6, respectively.

ENGINEERING PARAMETERS

The following sections describe the engineering parameters selected for the safety factor analyses presented within this calculation package.

Material Parameters

Material parameters for dike fill, foundation soils, and underlying strata have been evaluated in the Data Package (Attachment 5) using in-situ and laboratory data collected in the vicinity of Ash Pond A. Table 1 provides a summary of the common material properties selected for each evaluated cross section as a part of this safety factor assessment. For each cross section, specific dike fill and sandy foundation soil drained shear strength parameters were developed from in-situ measurements (i.e., Standard Penetration Test (SPT) N-values and CPT soundings, etc.). The interpretation and selection of properties for each cross section are shown on Figures 7 through 10 for Cross Sections A through E, respectively.

It was assumed that seismic waves generated during a potential seismic event would load clayey foundation soils rapidly enough to induce an undrained loading condition within clayey soils. In accordance with recommendations made within Hynes-Griffin and Franklin (1984), the selected undrained shear strength values were reduced by 20 percent for the seismic safety factor case to account for potential cyclic degradation during an earthquake. Thus, both drained and undrained strength parameters for clayey foundation soils were developed for each cross section from in-situ testing and laboratory results, as shown on Figures 7 through 10.

Seismic Loading and Allowable Displacement

An evaluation of the seismic hazard for WGS and the site response analysis for the Ash Pond A perimeter dikes is presented in the *Seismic Hazard Evaluation and Site Response Analysis: Ash Pond A* (Attachment 6) of this Safety Factor Assessment Report. Within Attachment 6, six ground motions for WGS were evaluated for two representative dike soil profiles for the Ash Pond A, and the profiles of the cyclic shear stress were computed. These computed cyclic shear stress profiles were utilized to compute the profiles of MHEA in general accordance with Bray et al (1995). Preliminary pseudo-static analyses of the perimeter dikes structures of Ash Pond A indicated that the typical critical depth of the anticipated slip surface is approximately 30 ft to 40 ft below the dike crest. Thus, the maximum MHEA at the critical slip surface below the dike crest computed from the set of ground motions for each cross section was selected to compute the k_h. The MHEA for each ground motion and representative soil profile to an approximate depth of 100 ft bgs is provided in Table 2. A MHEA of 0.050g, 0.054g, 0.070g, 0.075g, and 0.059g was selected for Cross Sections A through E, respectively.

As described in the Methodology section, the k_h must be computed assuming an allowable displacement (u). An allowable displacement of 12 inches (30.48 centimeters) was selected for the Ash Pond A perimeter dike structures. Using the Hynes-Griffin and Franklin (1984) chart and assuming the "Upper Bound" displacement, the ratio of N/A (or k_h/MHEA) was conservatively selected as 0.50, as shown in Figure 11. k_h for each cross section was computed as 0.5 × MHEA which was calculated as 0.025g, 0.027g, 0.035g, 0.0375g, and 0.0295g for Cross Sections A through E, respectively.

RESULTS

The safety factor evaluation for Cross Sections A through E was performed according to the methodology and parameters outlined within this calculation package, and the results are summarized within Table 3. Computed safety factors were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. The critical cross section, i.e., the section with the lowest computed safety factor, was found to be Cross Section A for the seismic safety factor case and Cross Section D for the static safety factor cases. Figures 12 through 14 and Figures 15 through 17 depict the computed critical values of FS for Cross Section A and Cross Section D, respectively.

CONCLUSIONS

Based on the assumptions, analyses, and results presented within this calculation package, Ash Pond A at WGS meets the periodic safety factor requirements described within the CCR Rule for existing CCR surface impoundments.

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TABLES

Table 1. Selected Material Parameters for Analysis

Material	**Total Unit Weight (pcf)[2]**	**Drained Parameters**		**Undrained Parameters[1]**	
		ϕ' (°)	**c' (psf)**	**S_u/σ'_{vo}**	**$S_{u,min}$ (psf)**
Dike Fill	125	38 to 40[3]	0	-	-
Clayey Foundation Soils	100	18	250	Varies[4]	100
Sandy Foundation Soils	115	31 to 34[3]	0	-	-
Loose Foundation Soils	110	20[3]	0	-	-
Chicora	130	50[2]	0	-	-
Williamsburg Formation Clay	105	50[2]	-	-	-
Fly Ash	100	0[2]	34	-	-

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only. Dike fill soils were observed to consist primarily of poorly graded silty sands in the vicinity of Ash Pond A.
2. The selection of shear strength parameters for Chicora, Williamsburg Formation Clay, and Fly Ash, as well as total unit weights for all materials, is discussed in the Data Package.
3. These drained shear strengths (ϕ') vary by location. Interpretation of in-situ results applied in the selection is provided in Figures 7 through 10.
4. The selected undrained strength ratio (S_u/σ'_{vo}) varies between locations and ranges from 0.25 to 0.40 for the selected cross section. Interpretation of in-situ results applied in the selection is provided in Figures 7 through 10. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in Attachment 5. These undrained shear strengths were reduced by 20 percent during pseudo-static analyses (i.e., seismic safety factor assessment) to account for cyclic degradation during a potential ground motion.

Table 2. Maximum Equivalent Horizontal Acceleration (MHEA) from Site Response Analysis for Ash Pond A Perimeter Dikes

Representative Profile 1 (Intake and Discharge Canal)							Representative Profile 2 (Cooling Pond)						
	Maximum Horizontal Equivalent Acceleration (g)							Maximum Horizontal Equivalent Acceleration (g)					
Depth (ft)	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360	Depth (ft)	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360
2.5	0.079	0.100	0.087	0.079	0.079	0.093	2.5	0.088	0.126	0.098	0.107	0.119	0.129
7.5	0.064	0.062	0.061	0.063	0.061	0.063	7.5	0.074	0.092	0.071	0.067	0.081	0.105
12.5	0.061	0.058	0.057	0.058	0.058	0.059	12.5	0.067	0.081	0.067	0.064	0.076	0.090
15	0.063	0.061	0.057	0.063	0.062	0.060	17	0.067	0.079	0.064	0.063	0.074	0.083
17.5	0.059	0.058	0.058	0.058	0.057	0.060	20.5	0.068	0.075	0.061	0.063	0.072	0.080
20	**0.059**	0.057	0.051	0.059	0.056	0.058	22	0.072	0.076	0.064	0.065	0.074	0.082
22.5	0.056	0.056	0.057	0.056	0.054	0.060	24.5	0.069	0.073	0.061	0.062	0.071	0.079
25	0.056	0.054	0.047	0.054	0.052	0.056	27	0.070	0.072	0.060	0.062	0.071	0.077
27.5	0.054	0.052	0.053	0.052	0.052	0.058	29.5	0.067	0.070	0.058	0.060	0.068	**0.075**
32.5	0.051	0.051	0.047	0.048	0.048	**0.054**	32	0.067	0.067	0.056	0.059	0.064	0.071
37.5	0.047	0.047	0.043	0.045	0.045	**0.050**	34.5	0.066	0.066	0.055	0.058	0.063	**0.070**
42.5	0.045	0.044	0.041	0.042	0.042	0.045	39.5	0.063	0.062	0.051	0.053	0.057	0.064
47.5	0.042	0.041	0.039	0.039	0.040	0.042	44.5	0.059	0.057	0.048	0.051	0.053	0.059
52.5	0.051	0.039	0.037	0.040	0.039	0.046	49.5	0.055	0.053	0.047	0.047	0.050	0.055
60	0.055	0.041	0.035	0.044	0.038	0.050	54.5	0.064	0.051	0.045	0.047	0.048	0.057
70	0.064	0.046	0.043	0.044	0.042	0.060	62	0.071	0.048	0.044	0.048	0.048	0.061
80	0.064	0.050	0.048	0.047	0.045	0.060	72	0.075	0.048	0.042	0.049	0.046	0.066
90	0.065	0.053	0.051	0.045	0.047	0.064	82	0.078	0.050	0.044	0.048	0.049	0.072
100	0.069	0.056	0.054	0.043	0.048	0.067	92	0.079	0.053	0.047	0.046	0.049	0.075
							102	0.078	0.054	0.050	0.044	0.049	0.077

Notes:

1. Cross Section A, Cross Section B, and Cross Section E, which are located adjacent to the Intake or Discharge Canals, were found to have depths to the critical slip surface of 37.5 ft, 32.5 ft, and 20 ft, respectively. A MHEA of 0.050g, 0.054g, and 0.059g was selected for Cross Section A, Cross Section B, and Cross Section E, respectively. Eager
2. Cross Section C and Cross Section D, which are located adjacent to the Cooling Pond, were found to have depths to the critical slip surface of 34.5 ft and 29.5 ft, respectively. A MHEA of 0.070g and 0.075g was selected for Cross Section C and Cross Section D, respectively.

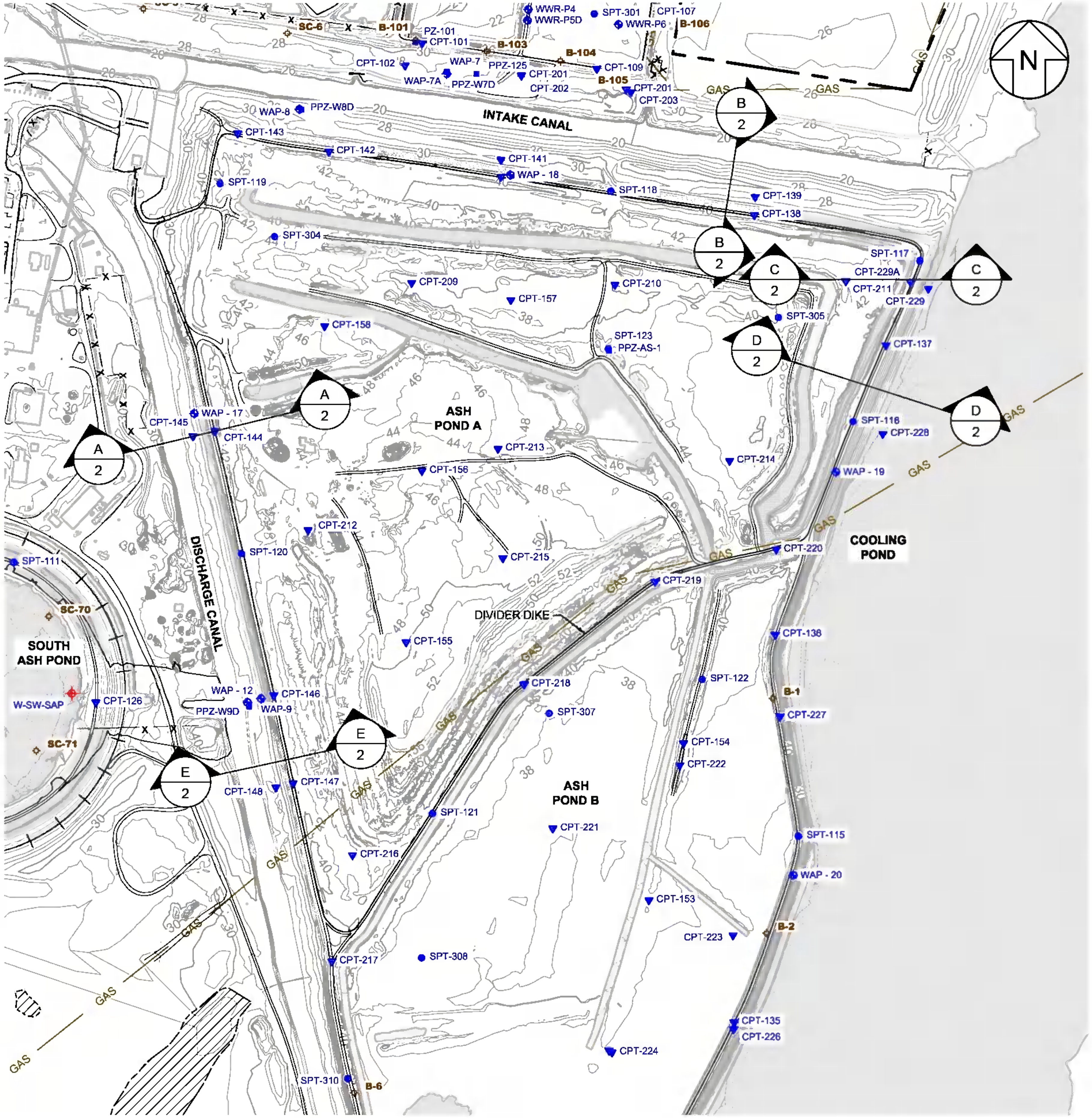
Table 3. Summary of Safety Factor Analysis Results

Safety Factor Case	**Target FS**	**Cross Section A**	**Cross Section B**	**Cross Section C**	**Cross Section D**	**Cross Section E**
Static - Maximum Normal Storage Pool	1.50	2.07	3.16	2.08	*1.58*[2]	2.16
Static FS- Maximum Surcharge Pool	1.40	1.92	2.88	1.89	*1.46*[2]	2.01
Seismic - Maximum Normal Storage Pool	1.00	*1.20*[2]	2.58	1.43	1.23	1.30
Liquefaction[1]	1.20	-	-	-	-	-

Notes:

1. The liquefaction safety factor was not evaluated since dike fill soils were not found to be liquefiable (Attachment 7).
2. The lowest computed safety factor for each analysis case was *italicized.* Critical FS's for Cross Section A were found to contain the lowest computed FS for the seismic case and are shown in Figures 12 through 14. Critical FS's for Cross Section D were found to contain the lowest computed FS for the static cases and are shown on Figures 15 through 17.
3. The lowest computed critical potential slip surfaces are reported above which generally pass through the perimeter dikes and subsurface (i.e., global slip surface) or fly ash materials.

FIGURES



LEGEND

— GAS — GAS —	EXISTING GAS LINE
— 20 —	EXISTING MAJOR GRADE CONTOUR
┼┼┼┼┼	EXISTING RAILROAD
▒	EXISTING PONDED WATER
⌖ W-SW-SAP	EXISTING STAFF GAUGE
▼ CPT-101	GEOSYNTEC CONE PENETRATION TEST
● SPT-111	GEOSYNTEC SOIL BORING
⌖ B-1	HISTORICAL BORING
◈ WAP-7, WWR-P4	MONITORING WELL
■ PPZ-125, PPZ-AS-1, PZ-101	PIEZOMETER

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
3. THE POSITION OF UNDERGROUND UTILITIES SHOWN ON THIS DRAWING IS BASED UPON THE LOCATION OF SURFACE APPURTENANCES AND/OR SURFACE MARKINGS AND SHOULD BE CONSIDERED APPROXIMATE.

0 400' 800'

SCALE IN FEET

ASH POND A
CROSS SECTION LOCATION MAP

Geosyntec consultants

FIGURE

1

PROJECT NO: GSC5242

OCTOBER 2016

M:\S\SANTEE COOPER\SANTEE COOPER-WINYAH\0028-WINYAH H&H ANALYSES\FIGURES\F-0-SC-585-00-F0028-040

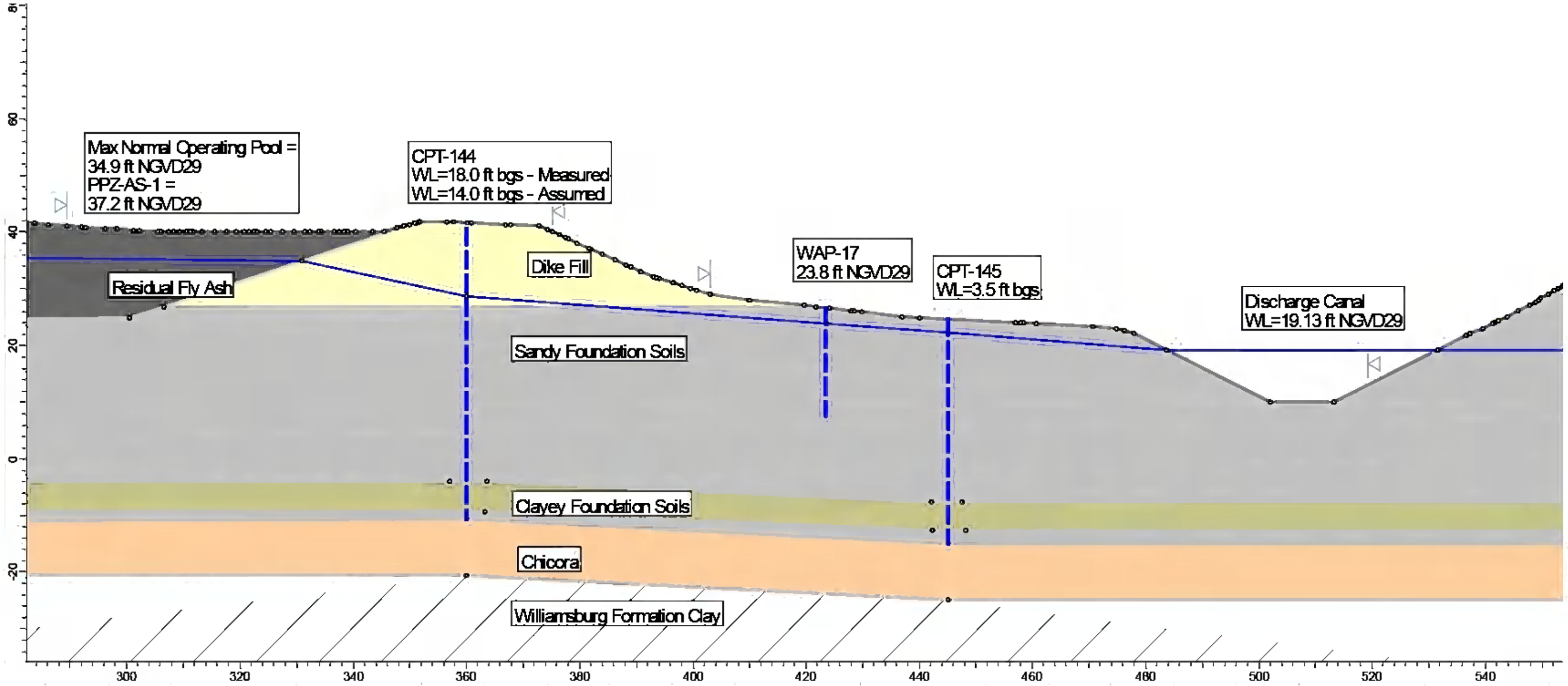


Figure 2. Cross Section A Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-144 was interpreted as 18 ft bgs but was conservatively assumed as 14.0 ft bgs for these analyses. The water level at WAP-17 (dike toe) was measured as 23.8 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the residual fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

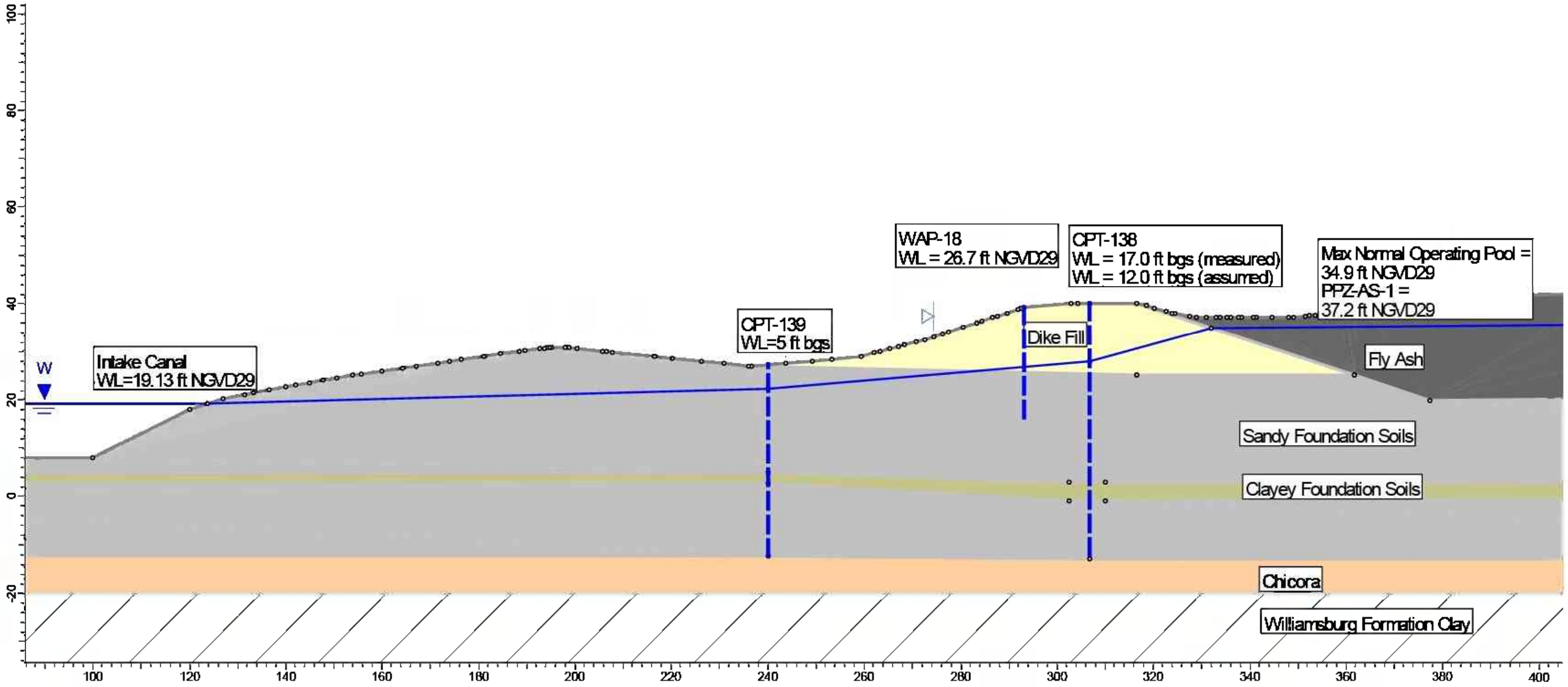


Figure 3. Cross Section B Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-138 was interpreted as 17.0 ft bgs but was conservatively assumed as 12.0 ft bgs for these analyses. The water level at WAP-18 (dike crest) was measured as 26.7 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the residual fly ash may be lower than the normal storage pool and the underlying sandy soils may draw down the phreatic surface through the dike.
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

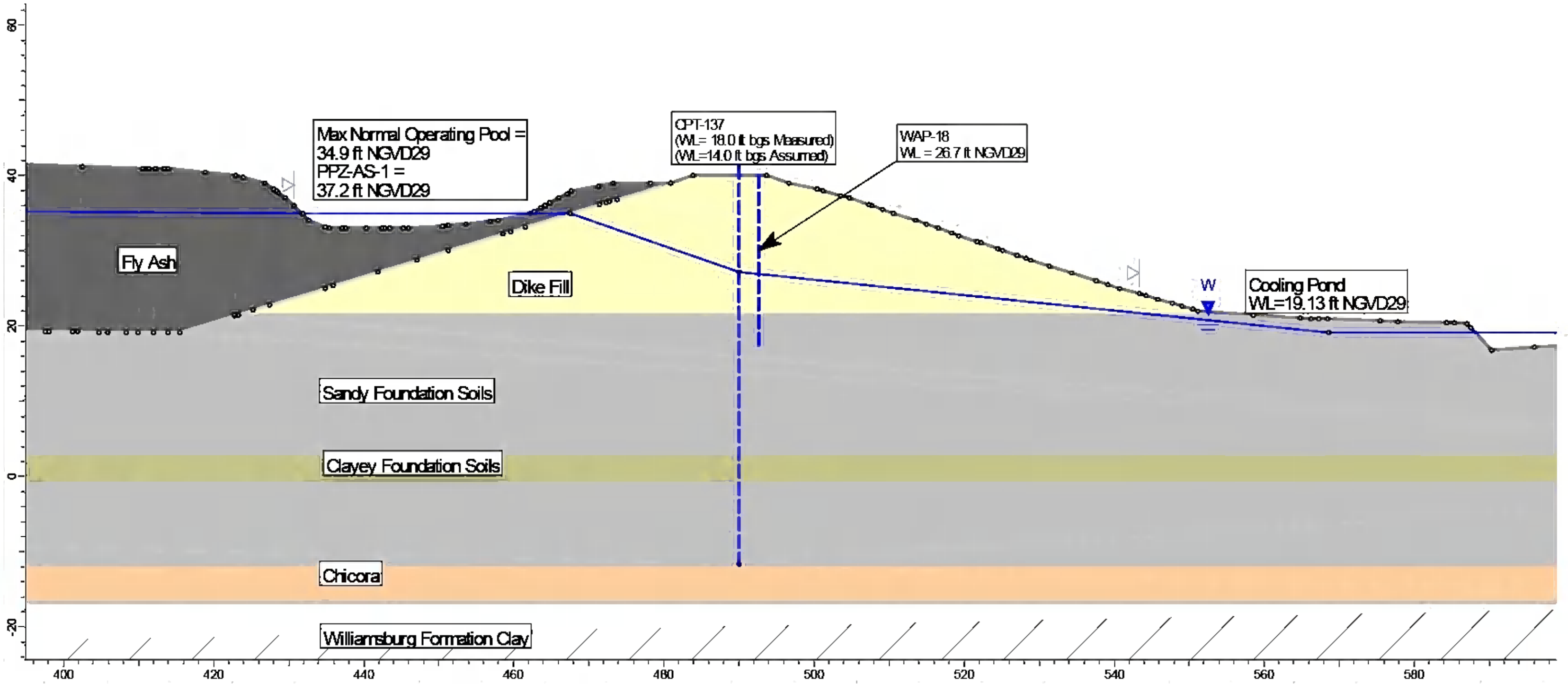


Figure 4. Cross Section C Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-137 was interpreted as 18.0 ft bgs but was conservatively assumed as 14.0 ft bgs for these analyses. The water level at WAP-18 (dike crest) was measured as 26.7 ft NGVD29 on 20 June 2016 (Attachment 5). Since the cross section is located between WAP-17 and WAP-18, WAP-18 was selected for modeling purposes due to the higher measured water level. The phreatic surface within the residual fly ash may be lower than the normal storage pool.
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

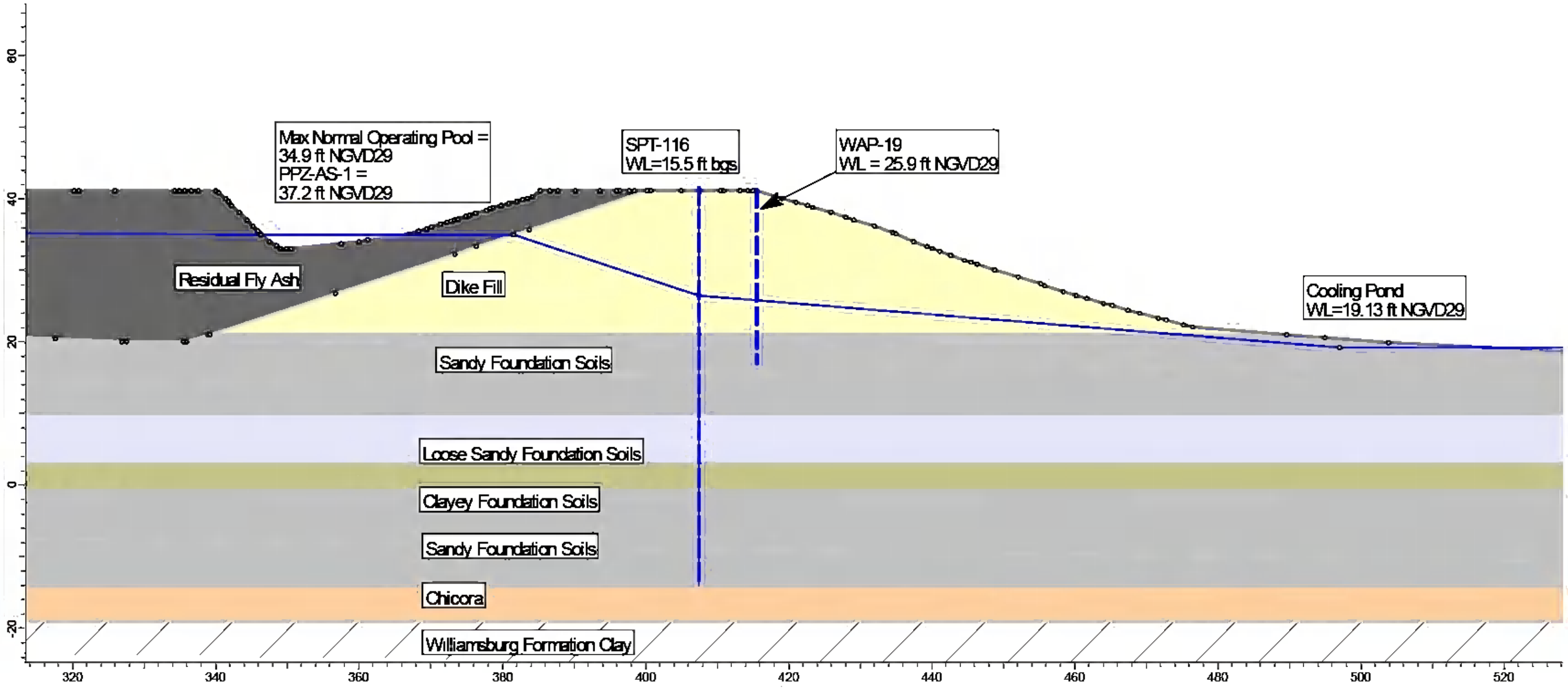


Figure 5. Cross Section D Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. The water level at WAP-19 (dike crest) was measured as 25.9 ft NGVD29 on 20 June 2016 (Attachment 5).
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

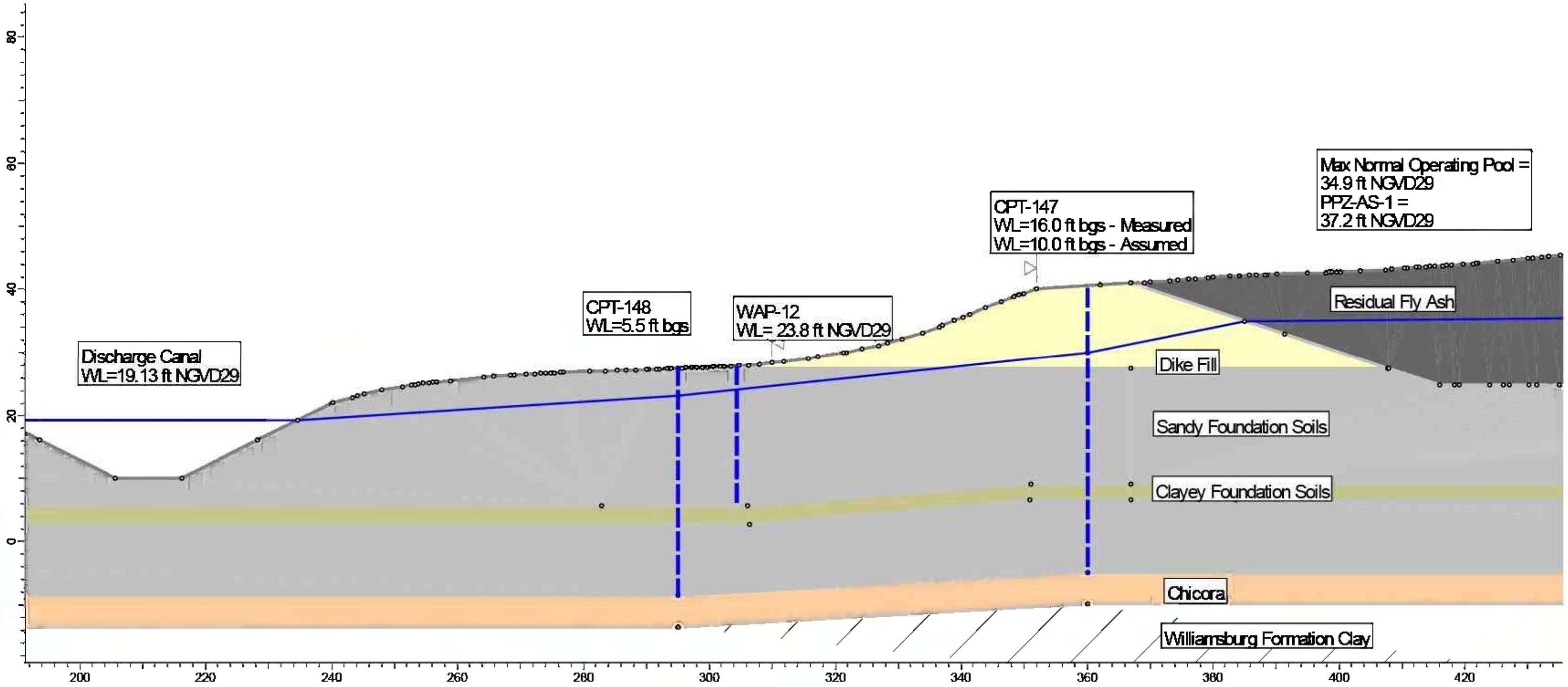


Figure 6. Cross Section E Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE$^{®}$)

Notes:

1. "Maximum Normal Storage Pool" was established as 34.9 ft NGVD29; however, phreatic surface was considered to be perched based on piezometer measurements at PPZ-AS-1 (37.2 ft NGVD29) at the center of the impoundment.
2. Water level at time of CPT-147 was interpreted as 16.0 ft bgs but was conservatively assumed as 10.0 ft bgs for these analyses. The water level at WAP-12 (dike toe) was measured as 23.8 ft NGVD29 on 20 June 2016 (Attachment 5). The phreatic surface within the fly ash may be lower than the normal storage pool.
3. "Maximum Surcharge Pool" (not shown in this Figure) was computed as 38.2 ft NGVD29 within the Ash Pond A interior, as shown in Attachment 1.

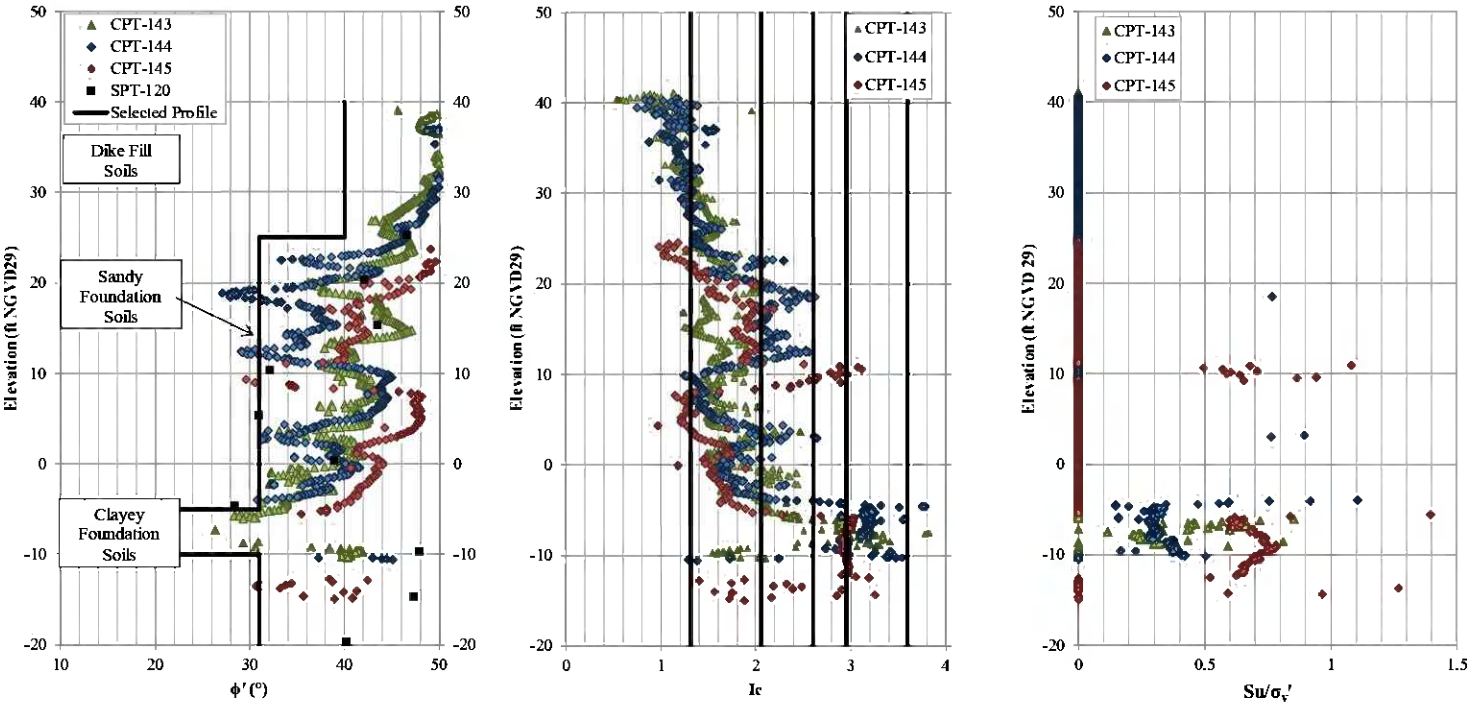


Figure 7. Subsurface Stratigraphy and Shear Strength Model for Cross Section A

Notes:

1. Clayey foundation soils were modeled with φ′ = 18° and c′ = 250 psf during static slope stability and with 80 percent of the $S_u/\sigma'_v = 0.25$ (i.e., $S_u/\sigma'_v = 0.20$) and a $S_{u,min} = 100$ psf during pseudo-static stability analysis (i.e., seismic safety factor).
2. A soil with a soil behavior index (I_c) greater than 2.60 was considered "clay-like" during this evaluation. The "clay-like" seam in CPT-145 near El. 10 ft NGVD29 was not considered in this evaluation. Note that S_u/σ'_v for intervals where $I_c < 2.60$ were plotted as zeros on the rightmost figure above.

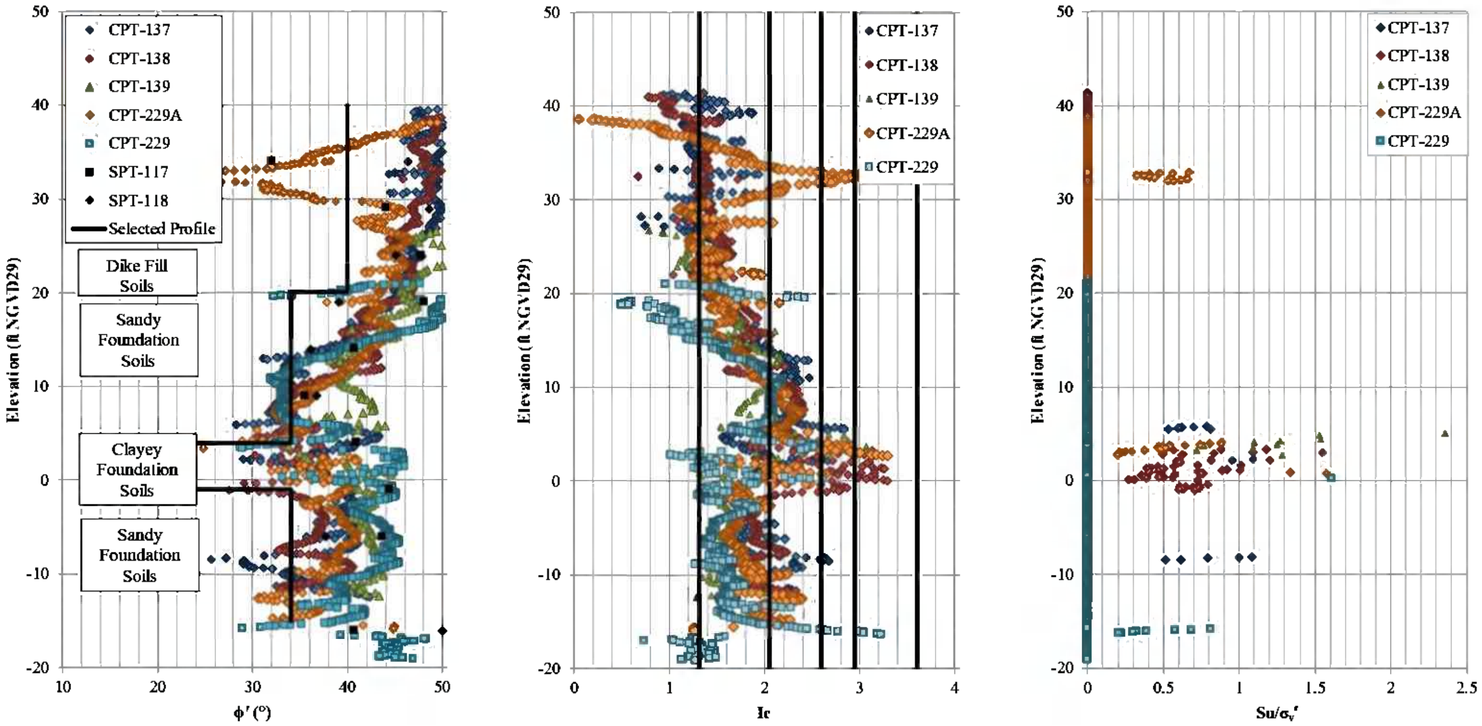


Figure 8. Subsurface Stratigraphy and Shear Strength Model for Cross Section B & C

Note:

1. Clayey foundation soils were modeled with $\phi' = 18°$ and $c' = 250$ psf during static slope stability and with 80 percent of the $S_u/\sigma'_v = 0.40$ (i.e., $S_u/\sigma'_v = 0.32$) and a $S_{u,min} = 100$ psf pseudo-static stability analysis (i.e., seismic safety factor). Note that S_u/σ'_v for intervals where $I_c < 2.60$ were plotted as zeros on the rightmost figure above.

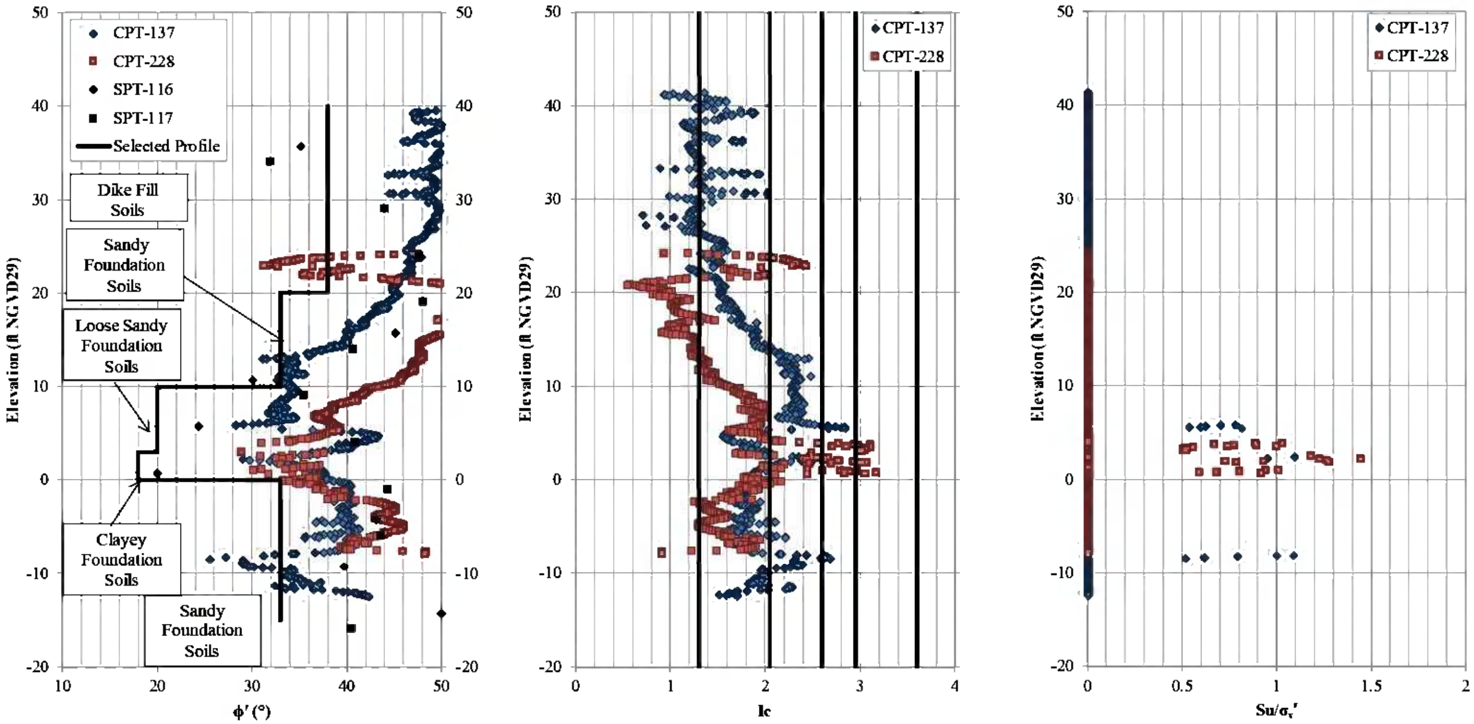


Figure 9. Subsurface Stratigraphy and Shear Strength Model for Cross Section D

Note:

1. Clayey foundation soils were observed in SPT-116 and were modeled from El. -0.5 to -3.0 ft NGVD29 with a $\phi' = 18°$ and a $c' = 250$ psf during static slope stability and with 80 percent of the $S_u/\sigma'_v = 0.40$ (i.e., $S_u/\sigma'_v = 0.32$) and a $S_{u,min} = 100$ psf during pseudo-static stability analysis (i.e., seismic safety factor). Note that S_u/σ'_v for intervals where $I_c < 2.60$ were plotted as zeros on the rightmost figure above.

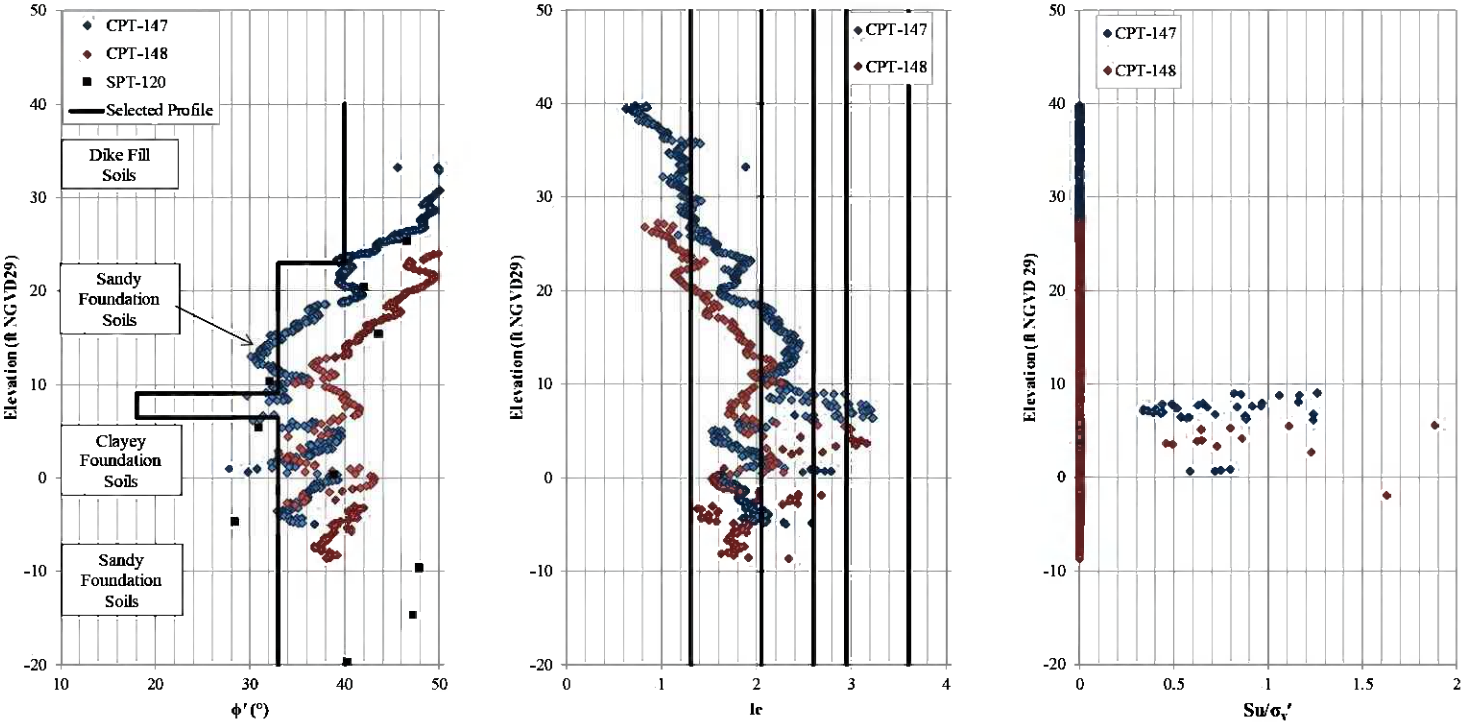


Figure 10. Subsurface Stratigraphy and Shear Strength Model for Cross Section E

Notes:

1. Clayey foundation soils were observed in CPT-147 and were modeled from El. 9.0 to 6.5 ft NGVD29 and from 5.6 to 2.7 ft NGVD29 within CPT-148 with a $\phi' = 18°$ and a $c' = 250$ psf during static slope stability and with 80 percent of the $S_u/\sigma'_v = 0.40$ (i.e., $S_u/\sigma'_v = 0.32$) and a $S_{u,min} = 100$ psf during pseudo-static stability analysis (i.e., seismic safety factor). Note that S_u/σ'_v for intervals where $I_c < 2.60$ were plotted as zeros on the rightmost figure above.

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **M. Viswanath/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**

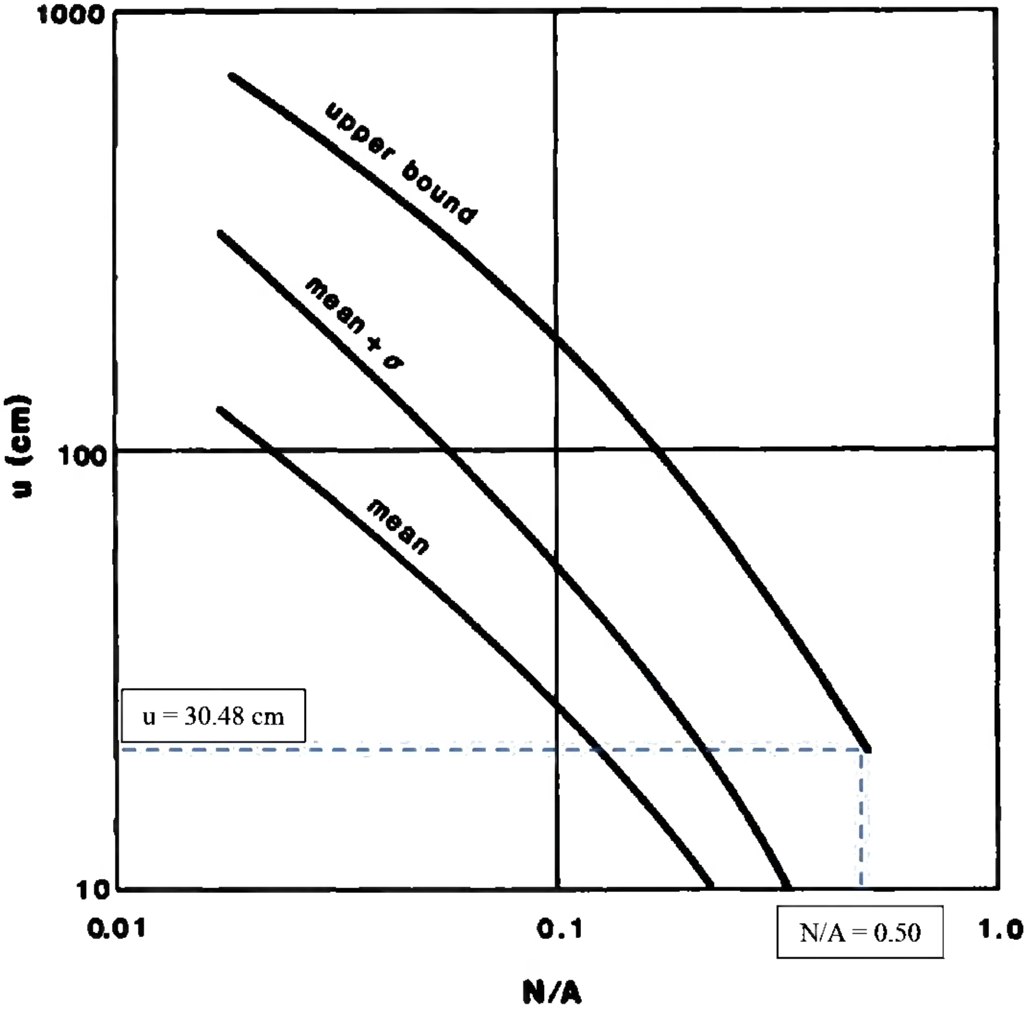
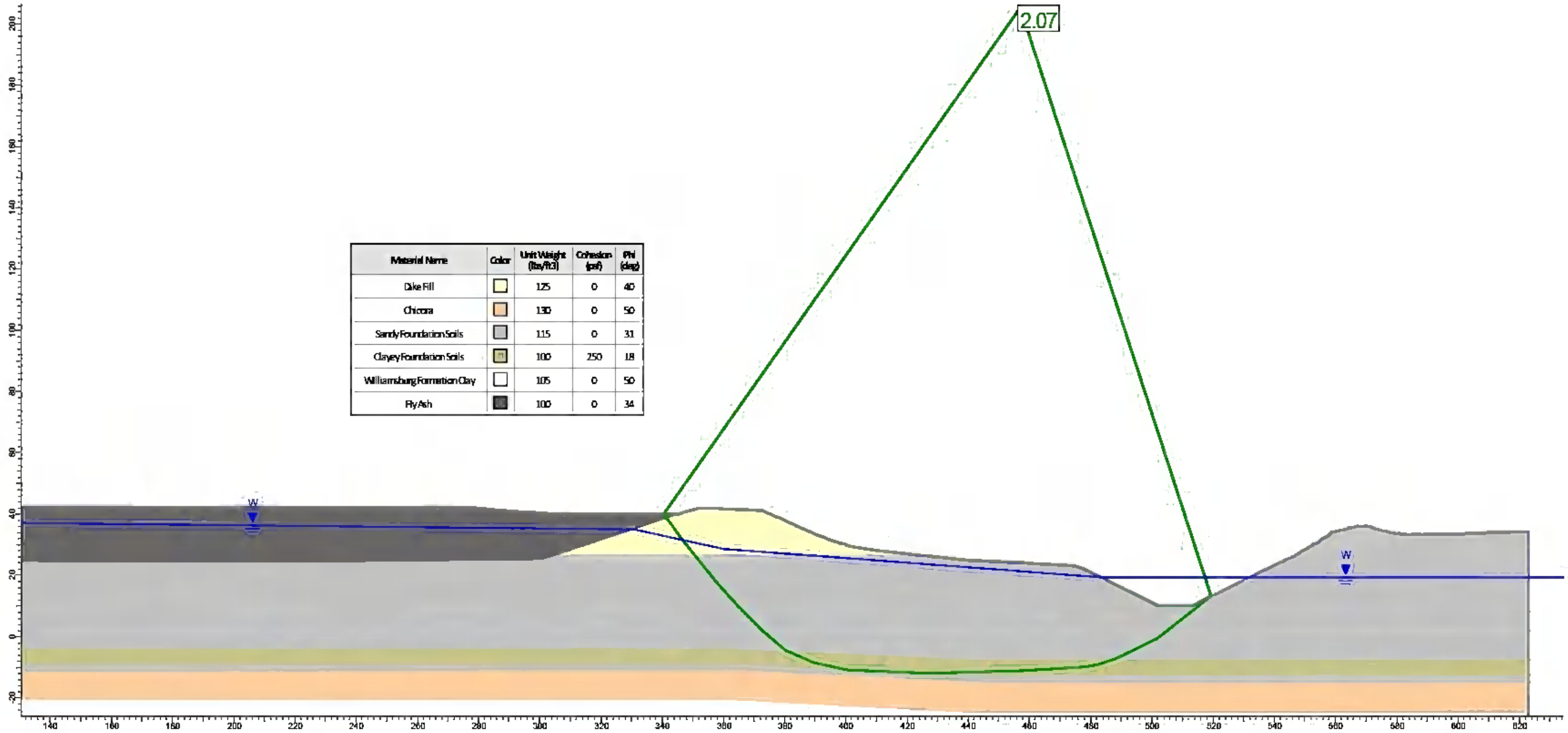


Figure 11. Allowable Displacement (u) vs. N/A (from Figure 7 of Hynes-Griffin and Franklin, 1984)

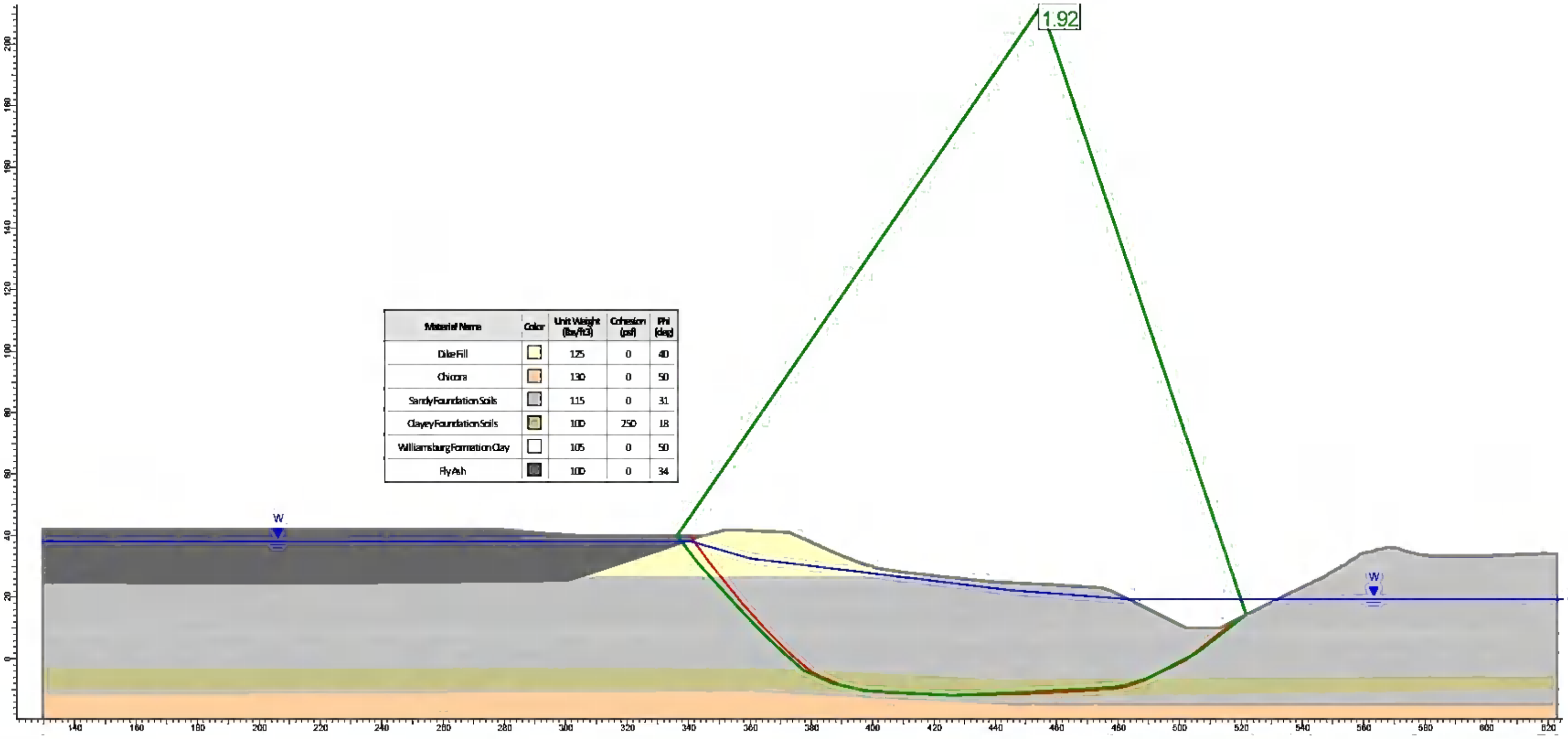
Notes:

1. An allowable deformation (u) of 12 inches (30.48 cm) and the "Upper Bound" curve were selected during these analyses.
2. A ratio of N/A of 0.50 was selected assuming 12 inches of displacement.



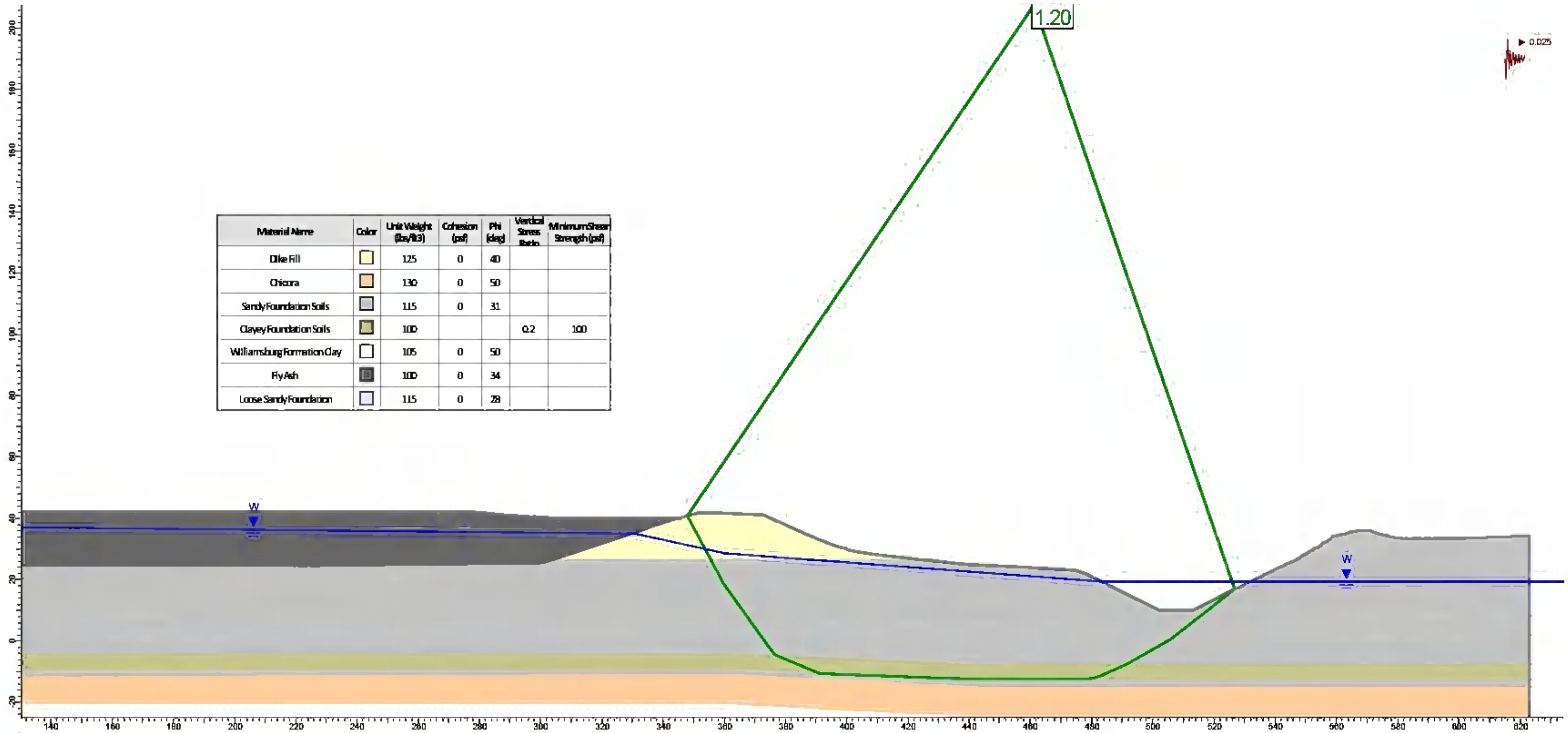
Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	☐	125	0	40
Chicora	☐	130	0	50
Sandy Foundation Soils	☐	115	0	31
Clayey Foundation Soils	☐	100	250	18
Williamsburg Formation Clay	☐	105	0	50
Fly Ash	■	100	0	34

Figure 12. Critical Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool



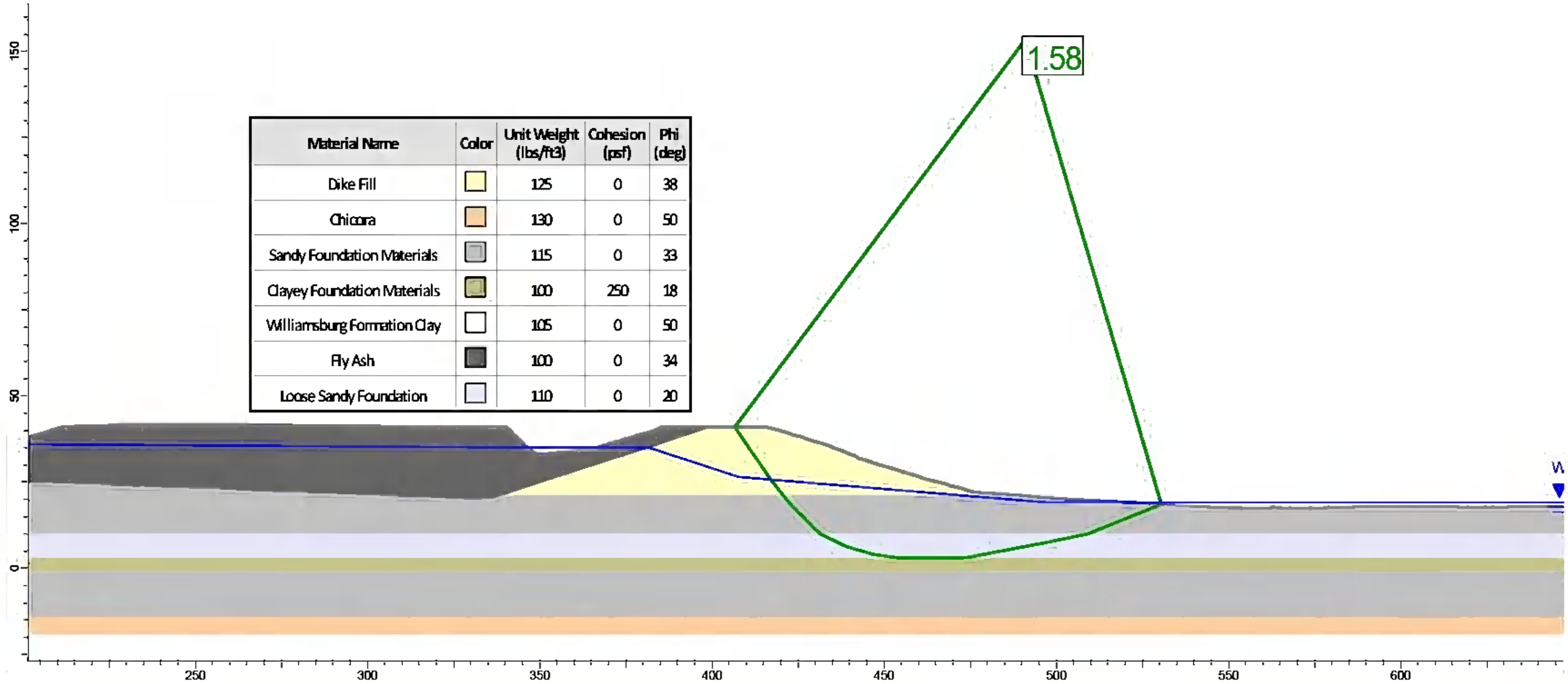
Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	🟨	125	0	40
Chicora	🟧	130	0	50
Sandy Foundation Soils	▨	115	0	31
Clayey Foundation Soils	🟩	100	250	18
Williamsburg Formation Clay	☐	105	0	50
Fly Ash	■	100	0	34

Figure 13. Critical Factor of Safety for Cross Section A: Static Factor of Safety - Maximum Surcharge Pool



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Dike Fill	🟨	125	0	40		
Chicora	🟧	130	0	50		
Sandy Foundation Soils	▨	115	0	31		
Clayey Foundation Soils	🟩	100			0.2	100
Williamsburg Formation Clay	☐	105	0	50		
Fly Ash	■	100	0	34		
Loose Sandy Foundation	☐	115	0	28		

Figure 14. Critical Factor of Safety for Cross Section A: Seismic Factor of Safety – Maximum Normal Storage Pool

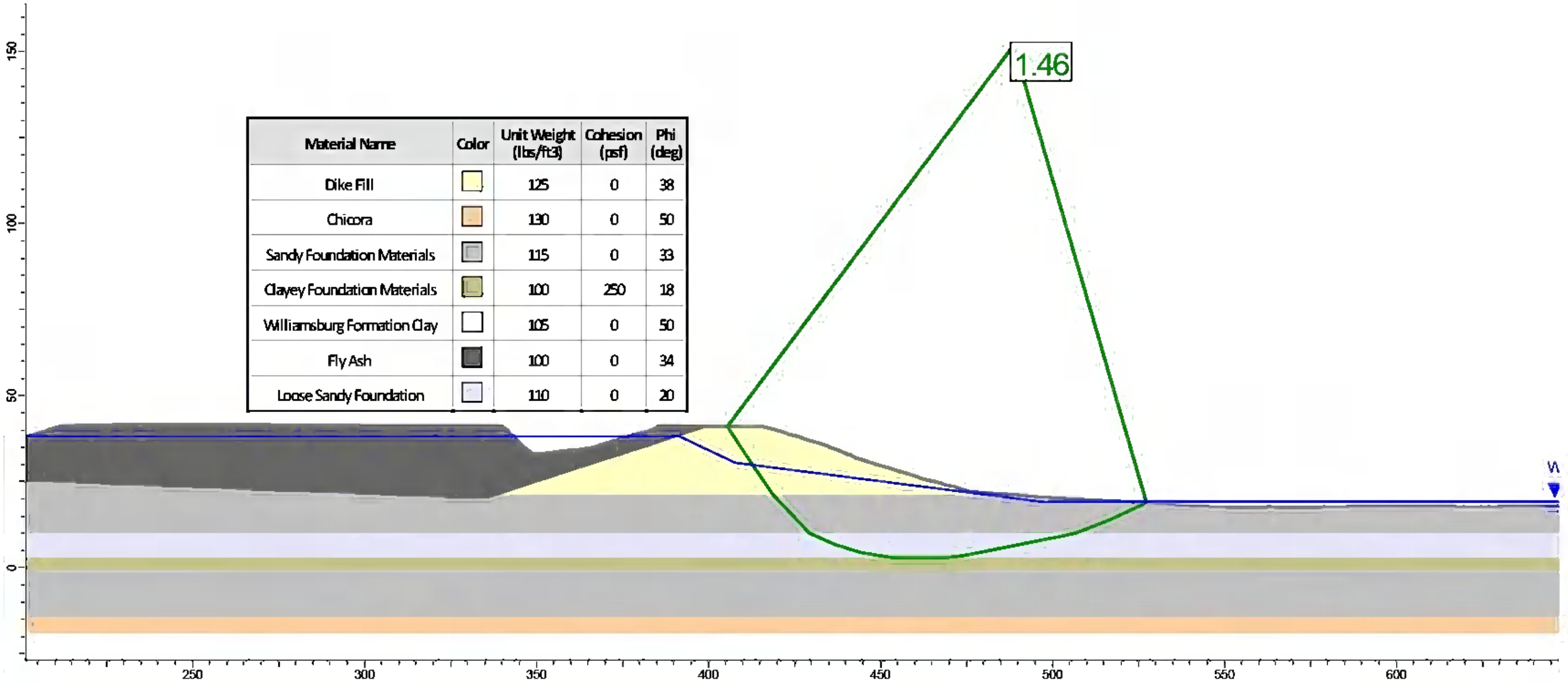


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	🟨	125	0	38
Chicora	🟧	130	0	50
Sandy Foundation Materials	▨	115	0	33
Clayey Foundation Materials	🟩	100	250	18
Williamsburg Formation Clay	☐	105	0	50
Fly Ash	⬛	100	0	34
Loose Sandy Foundation	☐	110	0	20

Figure 15. Critical Factor of Safety for Cross Section D: Static Factor of Safety – Maximum Normal Storage Pool

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **M. Viswanath/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**

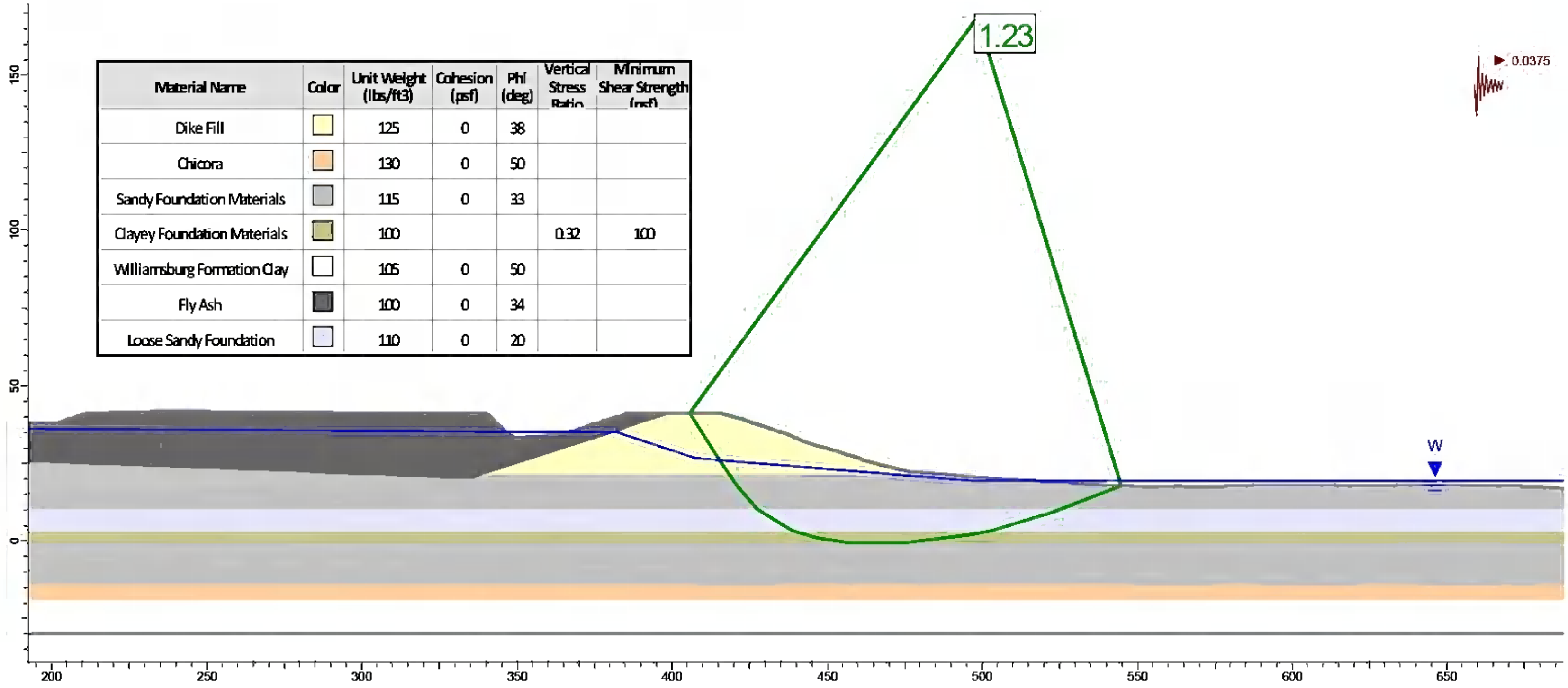


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Dike Fill	🟨	125	0	38
Chicora	🟧	130	0	50
Sandy Foundation Materials	▨	115	0	33
Clayey Foundation Materials	🟩	100	250	18
Williamsburg Formation Clay	⬜	105	0	50
Fly Ash	⬛	100	0	34
Loose Sandy Foundation	▢	110	0	20

Figure 16. Critical Factor of Safety for Cross Section D: Static Factor of Safety - Maximum Surcharge Pool

Written by: **J. McNash** Date: **10/10/2016** Reviewed by: **M. Viswanath/M. Zhu** Date: **10/10/2016**

Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)
Dike Fill	🟨	125	0	38		
Chicora	🟧	130	0	50		
Sandy Foundation Materials	▨	115	0	33		
Clayey Foundation Materials	▨	100			0.32	100
Williamsburg Formation Clay	□	105	0	50		
Fly Ash	■	100	0	34		
Loose Sandy Foundation	□	110	0	20		

Figure 17. Critical Factor of Safety for Cross Section D: Seismic Factor of Safety – Maximum Normal Storage Pool