Prepared for

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2016 SURFACE IMPOUNDMENT PERIODIC SAFETY FACTOR ASSESSMENT REPORT

SOUTH ASH POND
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Prepared by

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TABLE OF CONTENTS

TABLE OF CONTENTS – Continued

LIST OF TABLES

TABLE OF CONTENTS – Continued

LIST OF FIGURES

TABLE OF CONTENTS – Continued

LIST OF ATTACHMENTS

CERTIFICATION STATEMENT

This initial periodic 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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10/12/2016
Date

EXECUTIVE SUMMARY

The Winyah Generating Station (WGS or "Site") is a coal-fired, electric generating facility owned and operated by Santee Cooper and 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 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.

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

A hydrologic and hydraulic (H&H) analysis (Attachment 1) of the South Ash Pond 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. Because the South Ash Pond 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. The free water level in the South Ash Pond is maintained at an elevation of 28.73 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a concrete riser structure which discharges eastward into the Discharge Canal. The peak water level during and after the IDF within the South Ash Pond was computed as 31.8 ft NGVD29, which is below the minimum dike crest elevation of 36.9 ft NGVD29. Thus, the South Ash Pond 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 geotechnical subsurface investigations and laboratory testing programs in 2013 and 2016 to characterize the dike and subsurface soils and supplement historical data for the South

Ash Pond perimeter dikes. Boring logs, cone penetration test (CPT) sounding data, and laboratory testing results are provided in Attachments 2, 3, and 4, respectively, and 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 2% probability of exceedance in 50 years (i.e., 2,475 year return period) as defined in §257.53. A peak ground acceleration of 0.16g was selected for the Site, and the basis for this selection is described in Attachment 6. Site response analyses (Attachment 6) were performed for representative profiles of the South Ash Pond perimeter dikes to compute the cyclic shear stress and maximum equivalent horizontal acceleration profiles supporting subsequent liquefaction potential analysis and safety factor assessments.

The potential of dike fill to liquefy during the design earthquake was evaluated at each soil boring and CPT sounding location situated at the perimeter dike crest (Attachment 7) based on the cyclic shear stresses computed during the site response evaluation, in-situ testing data, and laboratory index testing results. Except for an approximately 1-ft thick zone of dike fill soil spanning from 25.8 ft to 26.8 ft NGVD29 at CPT-205 in the northwest corner of the South Ash Pond, the evaluation results did not show that the dike fill soils or foundation soils directly underlying the perimeter dikes of the South Ash Pond were susceptible to liquefaction during the design earthquake. The liquefaction potential of the foundation soils outside the footprint of the South Ash Pond will be evaluated separately at a later time as a part of an evaluation of "Unstable Areas" conducted in accordance with §257.64.

A safety factor assessment (Attachment 8) was performed on five selected cross sections of the South Ash Pond perimeter dikes 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 "Maximum Normal Storage Pool" level (28.73 ft NGVD29) and the calculated "Maximum Surcharge Pool" level (i.e., 31.8 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 and liquefaction slope stability with minimum required safety factors of 1.00 and 1.20, respectively, were also evaluated during "Maximum Normal Storage Pool" conditions. The liquefaction safety factor was evaluated for the nearest cross section to the computed liquefiable layer within dike fill soils (Attachment 7), and considered post-liquefaction, residual shear strengths for the liquefied layer. The safety factor assessment results indicated that the selected cross sections of the South Ash Pond 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 for each cross section in this Safety Factor Assessment Report, because, except for the location described above, the dike fill or the foundation soils directly underlying of the perimeter dike were not found to be susceptible to liquefaction. However, the post-liquefaction conditions of the foundations soils outside the footprint of South Ash Pond involving the perimeter dikes may be evaluated as a 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 (CCRs): Slurry Pond 3&4 (Slurry Pond), West Ash Pond, Unit 2 Slurry Pond, Ash Pond A, Ash Pond B, and South Ash Pond.

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR Parts 257 and 261 that regulate 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.

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

1.2 Project Site and Construction History

The South Ash Pond, spanning approximately 76 acres, is located immediately south of the Coal Pile and power block and west of the Discharge Canal. This unlined surface impoundment was commissioned in 1980 and is designated for the disposal of fly ash, bottom ash, and boiler slag. The South Ash Pond is bounded by the Coal Pile and power block to the north, Pennyroyal Creek to the west, a forested area to the south, and an access road and the Discharge Canal to the east.

The South Ash Pond was constructed by recompacting excavated soils from the surface impoundment interior to form perimeter dikes. The South Ash Pond perimeter dike has a maximum height of approximately 24 feet (ft). The crest elevation is approximately 38.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) and the toe elevation is approximately 24.0 ft NGVD29. The interior and downstream side slopes of the dikes are approximately 3 horizontal to 1 vertical (3H:1V), except in the western corner where the downstream side slopes are approximately 4H:1V. The dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012). The minimum elevation of the dike crest is 36.9 NGVD29 (Thomas and Hutton, 2012).

The South Ash Pond receives low volume wastewater, hydroveyor water, fly ash sluice water from Units 3 and 4, and stormwater from the SEFA Star Facility. Bottom ash sluice water from Units 3 and 4 and Coal Pile runoff may also be conveyed into this surface impoundment, but are typically directed to Ash Pond A and the Slurry Pond, respectively.

1.3 Report Organization

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

- Descriptions of the performance of the hydraulic structures are presented in Section 2;
- Geotechnical subsurface investigations programs previously by Soil and Materials Engineers, Inc. (S&ME) and recently by Geosyntec are presented in Section 3;
- Subsurface conditions, geology, and geotechnical properties at WGS are discussed in Section 4;
- Selection of the seismic hazard parameters for WGS and the site response analysis of the South Ash Pond perimeter dikes performed by Geosyntec are presented in Section 5;

- Results of the liquefaction potential evaluation conducted by Geosyntec for the South Ash Pond perimeter dikes are presented in Section 6;

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

- The summary and general conclusions from the safety factor assessment are presented in Section 8.

2. HYDROLOGIC AND HYDRAULIC EVALUATION

2.1 Hydrologic and Hydraulic Analysis

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

2.1.1 Regulatory Framework

The CCR Rule (§257.73(d)(1)) requires that a 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."*

The CCR Rule (§257.73(d)(1)(v)(B)(3)) also 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."*

Further, §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 the South Ash Pond computed during the H&H analysis as the "maximum surcharge pool" elevation in the analyses conducted to demonstrate that the requirements of §257.73(e)(1)(ii) are met.

A 4-ft by 4-ft concrete riser structure and 36-inch (in.) diameter reinforced concrete pipe (RCP) located on the east side of the South Ash Pond serve as the spillway for the surface impoundment, manage the free water and process water within the South Ash Pond, and discharge to the east into the Discharge Canal. This spillway also manages discharge during and after the IDF. Because the South Ash Pond 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. The South Ash Pond was assigned a "Low Hazard Potential" classification (Geosyntec, 2016a) since a potential failure would be contained within the property boundary and would not be anticipated to migrate offsite. H&H analyses were performed to demonstrate that the South Ash Pond spillway is 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 H&H analysis results were utilized herein to calculate the maximum surcharge pool elevation in support of the safety factor assessment per 257.73(3)(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: South Ash Pond"*, 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 the South Ash Pond.

The concrete riser structure and RCP manage the discharge from the South Ash Pond. The inlet and outlet inverts for the RCP were 16.93 ft (Lockwood-Greene, 1978).

The South Ash Pond receives contact stormwater from the Coal Pile after rainfall events, which was modeled to have an inflow of 2,450 gallons per minute (gpm) (5.46 ft^3/s) (Santee Cooper, 2014). Units 3 and 4 low volume wastewater, Units 3 and 4 hydroveyor water, and SEFA Star II Scrubber blowdowns were considered to have a combined base inflow to the South Ash Pond totaling 2,740 gpm (6.10 ft^3/s).

The operating level in the South Ash Pond is maintained by the concrete riser structure with a top stop log elevation of 28.73 ft NGVD 29 and associated RCP (Thomas and

Hutton, 2016). The tailwater conditions associated with discharge from the South Ash Pond into the Discharge Canal were modeled using a fixed water surface elevation within the Discharge Canal and Cooling Pond. The tailwater surface elevation was estimated by conservatively assuming 2.5 ft depth of free 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) during the IDF.

HydroCAD® Version 10.0 software (HydroCAD, 2011) 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 the South Ash Pond during the IDF. The 100-yr rainfall event with a 72-hour (hr) duration precipitation event resulted in a rainfall depth of 12.8 in. (NOAA, 2006) and was modeled within HydroCAD® using a SCS Type III rainfall distribution.

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 IDF event (100-yr rainfall with a 72-hr duration) was computed as 31.8 ft NGVD29 occurring 38.1 hours into the rainfall event.

3. GEOTECHNICAL SUBSURFACE INVESTIGATION PROGRAMS

3.1 Overview

This section summarizes the geotechnical subsurface investigation and laboratory testing programs performed in the vicinityof the South Ash Pond perimeter dikes at WGS. In 1977 and 1978, S&ME performed a general subsurface investigation supporting the construction of CCR surface impoundments, including the South Ash Pond, at the WGS (S&ME, 1978). In October 2013, Geosyntec conducted a subsurface investigation in the vicinity of the South Ash Pond to collect geotechnical data supporting the evaluation of closure alternatives for the surface impoundment. Geosyntec remobilized to the site in March 2016 to conduct a focused subsurface investigation of the soft clay foundation layer that underlies the dike on the west corner of the pond. Figure 3 presents the locations of soil test borings performed during the investigations and cone penetration test (CPT) soundings conducted as part of Geosyntec's subsurface investigations.

Soil test borings, CPT sounding data, and laboratory test results for the subsurface investigation programs are included in Attachments 2, 3, and 4, respectively, of this Safety Factor Assessment Report. The interpretation of the subsurface stratigraphy and materials properties used in the geotechnical analyses for the South Ash Pond are presented in Attachment 5 of this Safety Factor Assessment Report.

3.2 Subsurface Investigations

3.2.1 Historical Investigation

The S&ME investigation (S&ME, 1978) was conducted to assess the suitability of on-site materials for construction and to design the perimeter dikes. In the vicinity of the South Ash Pond, the investigation included 18 soil test borings (SC-63, SC-64, SC-66 to SC-78, SC-80, SC-81, and SC-84) advanced before construction of the surface impoundment from 26.5 to 41 feet (ft) below ground surface (bgs) until the refusal was encountered at the Chicora Member (dense cemented shell unit). SPT blow counts (i.e., N-values) were recorded at approximately 2.5 ft intervals up to 10 ft below ground surface and at 5-ft depth intervals thereafter. Representative samples were collected by a standard split spoon sampler or by thin-walled Shelby tubes, which were utilized for index, consolidation, and triaxial shear strength testing. The geotechnical laboratory program consisted of index (grain size distribution and Atterberg limits), unit weight, compaction, consolidation, and shear strength testing of select samples.

3.2.2 Geosyntec Investigations

The October 2013 subsurface investigation conducted by Geosyntec included five soil test borings (SPT-109 to SPT-113) and twelve CPT soundings (CPT-122 to CPT-126, CPT-128 to CPT-133, and CPT-130A). One of the soil borings (SPT-113) and three of the CPT soundings (CPT-131 to CPT-133) were advanced within the interior South Ash Pond and were terminated once native foundation materials were encountered. The remaining borings and soundings were conducted in the dike materials and, except as described below for SPT-110 and SPT-112, were terminated once refusal was encountered. Refusal was defined in the field as an SPT N-value of 50 blows per ft over an advancement of 6 inches (in.) or the inability to further advance the cone; refusal occurred at the top of the Chicora Member. Soil Consultants, Inc. (SCI) of Charleston, SC was the drilling subcontractor, and Mid-Atlantic Drilling, Inc. (MAD) of Wilmington, North Carolina conducted the CPT soundings.

The four soil test borings drilled in the dike materials were advanced to a depth of 51 to 68 ft bgs using a CME-550X drill rig. Drilling was performed using the mud rotary wash method in general accordance with recommendations of Idriss and Boulanger (2008) (Table 1). Split-spoon samples and SPT blow counts (i.e., N-values) were generally collected in 5-ft depth intervals. Several thin-walled Shelby tube samples were also collected in the vicinity of the perimeter dikes. In two soil borings (SPT-110 and SPT-112), SCI replaced the side discharge drill bit with a tri-cone drill bit once the Chicora Member was encountered in order to penetrate the unit. The Chicora Member was slowly drilled through until the underlying Williamsburg Formation Clay was encountered, and then these borings were advanced an additional 5 ft before attempting the collection of a Shelby tube sample and terminating the borings. Boreholes located on the dike centerline were left open for two to three days prior to abandonment, and depth to water levels were recorded before the borings were plugged with a cement-bentonite grout.

Of the nine CPT soundings advanced in the area of the perimeter dike, six were advanced through the perimeter dike centerline, and three CPT soundings were advanced at the dike toe. Shear wave velocity (V_S) testing was conducted at 5-ft depth intervals for three locations along the perimeter dike centerline (CPT-123, CPT-124, CPT-129), two locations at the dike toe (CPT-125, CPT-130A), and two locations within the impoundment interior (CPT-132, CPT-133). Pore pressure dissipation tests were performed as well along the dike centerline (CPT-122, CPT-129), dike toe (CPT-130A), and within the CCR (CPT-131, CPT-133). Results of the V_S and pore pressure dissipation tests are included in Attachment 3.

In March 2016, Geosyntec remobilized to WGS to conduct supplemental soil test borings and CPT soundings on the west corner of the South Ash Pond. Three soil test borings (SPT-302, SPT-303, and SPT-303A) were advanced by the mud rotary wash drilling method, four CPT soundings (CPT-204 to CPT-206 and CPT-208) were advanced through the perimeter dike centerline, and one CPT sounding (CPT-207) was advanced at the dike toe. Two of the CPT soundings were conducted with shear wave velocity (V_S) measurements. The purpose of the subsurface investigation was to: (i) collect physical samples of foundation soils immediately underlying the dike fill for geotechnical laboratory testing; (ii) further characterize the material properties of the observed soft clay foundation soil; and (iii) evaluate the relative density of dike fill soils.

3.3 Laboratory Testing

Geotechnical laboratory testing of soils was conducted during the SM&E and Geosyntec investigations. Laboratory testing results are provided in Attachment 4, and the interpretation of the laboratory testing program is discussed in Attachment 5 of this Safety Factor Assessment Report.

The SM&E laboratory testing program included index testing (percent fines and natural water content), shear strength testing (consolidated undrained (CU) and unconsolidated undrained (UU) triaxial compression), one-dimensional (1-D) consolidation testing, and unit weight testing.

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, foundation soils, and CCR (split-spoon only). The geotechnical laboratory testing program included index testing (20 fines tests, 19 grain size distributions, seven Atterberg limits tests, and 41 natural water content tests), shear strength testing (four CU triaxial compression tests), three 1-D consolidation tests, and nine unit weight tests.

4. SUBSURFACE CONDITIONS AND GEOTECHNICAL PROPERTIES

This section presents regional geology, subsurface conditions, phreatic surface and free water levels, and material properties for the South Ash Pond based on the geotechnical subsurface investigation program discussed in Section 3. A summary of the regional geology is 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 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 consists of light-gray to black shale interlaminated with thin seams of fine-grained sand and mica.
- The Peedee Formation 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 Safety Factor Assessment 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 Perimeter Dike Subsurface Conditions and Water Levels

4.2.1 Subsurface Stratigraphy

The subsurface stratigraphy at the Site was developed from information obtained from the historical and Geosyntec geotechnical subsurface investigations at WGS and is supported by the regional geology. 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 South Ash Pond perimeter dikes were generally observed to be medium dense to very dense, poorly graded to silty sands with uncorrected SPT blow counts typically ranging between 15 and 60 blows per foot and CPT sounding tip resistances typically ranging between 100 and 500 tsf. Grain size testing indicated that dike fill soils typically consist of 60 percent to 91 percent sand-sized material (smaller than No. 4 sieve but greater than No. 200 sieve) and 10 percent to 40 percent silt and clay-sized material (percent fines), with most samples containing 5 percent to 20 percent fines by weight.

- Foundation Soils: Foundation soils were observed to be variable across the South Ash Pond footprint and consist primarily of poorly graded to silty sands with shells and pockets of clayey sand to high plasticity clay. Uncorrected blow counts within foundation soils typically ranged between 2 and 35 blows per foot, and CPT sounding tip resistances typically ranged between 40 and 200 tsf. A 15 to 20-ft thick layer of soft clay, with uncorrected blow counts ranging from 0 to 4 blows per foot and CPT tip resistances below 20 tsf, was observed in the west to southwest corner of the South Ash Pond.

- Chicora Member: A layer of dense to very dense, partially cemented to heavily cemented shells was encountered beneath the foundations soils during the past subsurface investigations at WGS. SPT blow counts in this layer exceeded 50 blows over less than 6 in. of advancement, with minimal sample recovery without rock coring. Based on review of historical (Doar, 2012) and existing data, this layer is the upper portion of the overall Williamsburg Formation and is referred to as the "Chicora Member", "Coquina", or "Shell Hash". Boring and CPT refusal was typically encountered at the top of this stratum. In the two South Ash Pond borings that penetrated the Chicora Member, the layer was found to be between 5 ft and 8 ft thick.

- Williamsburg Formation Clay: The Williamsburg Formation Clay was encountered beneath the Chicora Member and 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 unit was found to be between 30 ft and 90 ft thick in the vicinity of WGS from a review of the regional geology. Based on two SPTs, uncorrected SPT blow counts within this stratum ranged from 10 to 19 blows per foot in the upper 10 ft of the unit. In other areas of the Site, uncorrected SPT blow counts exceeded 20 blows per foot, increasing with depth, in the upper 20 ft of the unit.

4.2.2 Water Levels

As described within Section 2, which describes the H&H analysis for South Ash Pond provided in Attachment 1 of this Safety Factor Assessment Report, the free water level within the South Ash Pond is maintained at an elevation of 28.73 ft NGVD29 by a concrete riser structure.

The phreatic surface through the centerline of the South Ash Pond perimeter dikes to the downstream toe at the time of this Safety Factor Assessment Report was predominantly developed based on water levels collected from CPT sounding u2 signatures and dissipation tests, 24-hour depth to water measurements in soil borings, and observed dike toe drain performance in 2013. In both the "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions, the phreatic surface through the South Ash Pond perimeter dikes was assumed to reach steady-state conditions and be controlled and drawn down by the toe drains and underlying sandy foundation soils.

An operating level of 28.73 ft NGVD29 was selected as the "Maximum Normal Storage Pool" for the South Ash Pond. The maximum free water elevation within the South Ash Pond during and after the IDF was computed as 31.8 ft NGVD29 (Section 2), which was selected as the "Maximum Surcharge Pool" level within this Safety Factor Assessment Report.

4.3 CCR

As noted in Section 3.2, one soil test boring and three CPTs were advanced within the interior of South Ash Pond during geotechnical subsurface investigations. Fly ash was typically described as very soft, wet, black, black, slightly sandy silt-sized material without plasticity or with low plasticity. Uncorrected SPT blow counts typically ranged between 0 (weight of hammer) and 3 blows per foot, and CPT sounding tip resistances typically ranged between 5 tsf and 40 tsf, with most values below 20 tsf.

4.4 Material Parameters

Representative parameters of subsurface materials were selected based on in-situ testing and laboratory testing results, as discussed in Attachment 5 of this Safety Factor Assessment Report. Correlations based on in-situ testing methods were applied to supplement laboratory testing to evaluate the effective stress strength parameters of the dike fill and foundation soils. Representative strength profiles were developed for each cross section evaluated for slope stability described in Section 7. However, a summary of the common material parameters selected for the safety factor assessment are presented in Table 1.

5. SEISMIC HAZARD EVALUATION

This section presents the results of seismic hazard evaluation and site response analysis of the South Ash Pond 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 of this Safety Factor Assessment Report 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 free 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” condition 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 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 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 response analyses of 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 South Ash Pond 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 depths for each profile were calculated (Table 2), and these values were 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 applied 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 South Ash Pond perimeter dikes. The evaluation applies the cyclic shear stress computed as a 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 of this Safety Factor Assessment Report.

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") for slope stability provided 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 South Ash Pond dike fill soils are susceptible to liquefaction. If soils are not found to be liquefiable within the dike fill, 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 soil borings and CPT soundings performed during Geosyntec's 2013 and 2016 subsurface investigations presented in Section 3 of this Safety Factor Assessment Report. 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 South Ash Pond perimeter dikes to the downstream dike toe at the time of liquefaction potential analysis was developed based on water levels collected from CPT sounding u2 signatures and dissipation tests, 24-hour depth to water measurements in soil borings, and observed dike toe drain performance in 2013. Operations of the South Ash Pond (i.e., CCR disposal and sluicing rates) have not changed significantly since the 2013 and 2016 subsurface investigations (Section 4.2.2), and these 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 6, an age correction factor (K_{dr}) of 1.3 was applied for the Pleistocene-aged soils at the WGS site (typically foundation soils below the base of the dike), and an age correction factor of 1.0 was applied to the dike fill soils. The location of the interface between dike fill soil and foundation soils was estimated as 1 ft below the toe drains elevations shown on the Lockwood-Greene (1978) design drawings.

6.3 Evaluation Results

The factor of safety against liquefaction (FS_{liq}) was computed at every depth interval where data was collected for soil test borings (2-ft or 5-ft intervals) and CPT sounding (0.16-ft intervals) advanced in the vicinity of the South Ash Pond perimeter dikes. Analysis results for each soil boring and CPT sounding analyzed are provided as figures within Attachment 7 of this Safety Factor Assessment Report. Except for an approximately 1-ft thick zone of dike fill soil spanning from 25.8 ft to 26.8 ft NGVD29 at CPT-205 in the northwest corner of the South Ash Pond, FS_{liq} values computed for dike fill soils were found to exceed 1.0 for the conditions described within this Safety Factor Assessment Report. Therefore, the liquefaction safety factor for the perimeter dikes is required to be evaluated during the periodic safety factor assessment for a critical cross section through the zone of liquefiable soil. It is noted that the post-liquefaction conditions of the foundations soils outside the footprint of the South Ash Pond perimeter dikes may be evaluated as a part of the assessment of "Unstable Areas" performed at a later time, depending on the liquefaction 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 South Ash Pond perimeter dikes. This evaluation is presented in detail in Attachment 8 of this Slope Stability Assessment Report and summarized herein.

7.1 Regulatory Framework

Slope stability analyses were conducted to assess whether the South Ash Pond perimeter dikes achieve the safety factor (also referred to as "factor of safety") criteria 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 the South Ash Pond based on the information obtained from several sources: (i) recent topographic surveys (Thomas and Hutton, 2012; Thomas and Hutton, 2016); (ii) design grading for the South Ash Pond cover drainage plan; (iii) available engineering reports and drawings for WGS; (iv) subsurface stratigraphy developed from subsurface investigations (Section 4); and (v) water level measurements (Section 4.2.2). Five selected representative cross sections (Cross Sections A through E) were evaluated, and their locations are depicted in Figure 5.

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, subsurface soil stratigraphy, phreatic surface elevation, external loading conditions, and properties of subsurface materials.

7.3.2 Seismic Slope Stability

Pseudo-static slope stability analyses were performed utilizing Spencer's method to evaluate the seismic performance of the perimeter dike structures using 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) to compute an additional horizontal force ($F = k_h \times W$) for each slice, based on slice weight (W), during a seismic event. 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 South Ash Pond perimeter dike structures was selected as 12 inches (in.) (30.5 centimeters (cm)). Based on this allowable displacement and the upper bound relation, the Hynes-Griffin and Franklin (1984) procedure was used to adjust the MHEA at the target depth by 0.5 to compute the k_h applied in 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 the South Ash Pond is maintained at 28.73 ft NGVD29 by a concrete riser structure.

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 South Ash Pond is maintained at 31.8 ft NGVD29 and steady-state conditions have been established within the perimeter dikes. The maximum surcharge pool elevation of 31.8 ft NGVD29 was computed as the peak free water level within South Ash Pond 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 computed seismic horizontal force coefficients of 0.092, 0.062, 0.066, 0.066, and 0.114, respectively, to each slice within SLIDE®. The seismic safety factor was evaluated for free water and phreatic surface levels considering the Maximum Normal Storage Pool Conditions as described in Section 7.4. During the evaluation of the seismic safety factor, soil shear strengths were 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, a layer of perimeter dike fill in the northwest corner of the South Ash Pond was found to be liquefiable near Cross Section A. Thus, a liquefaction safety factor assessment was evaluated for Cross Section A assigning post-liquefaction residual strengths for the soil layer that was computed to liquefy during the design earthquake. Under these assumptions, the South Ash Pond perimeter dikes were found to meet the minimum liquefaction factor of safety of 1.20.

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. Analysis cases for the maximum normal storage pool condition are shown for Cross Sections A through E in Figures 6 through 10, respectively. Cross Section A was found to contain the lowest safety factor for the static safety factor cases for both the maximum normal storage pool condition (Figure 11) and the maximum surcharge pool condition (Figure 12), while Cross Section B was calculated to contain the lowest safety factor for the seismic safety factor case (Figure 13). Results of the liquefaction maximum normal storage pool condition are shown in Figure 14. These results indicate that the perimeter dikes of the South Ash Pond at WGS meet the periodic safety factor assessment criteria given in §257.73(e)(1) of the CCR Rule. Further details of the safety factor assessment for the South Ash Pond 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 the South Ash Pond during the 100-yr rainfall event (IDF) was evaluated, and the calculated maximum surcharge pool within the surface impoundment was used for the safety factor assessment.

- A desktop review of site history and engineering reports, 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 the South Ash Pond.

- 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 2 percent probability of exceedance 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 South Ash Pond perimeter dikes.

- Except for an approximately 1-ft thick zone of dike fill soil located in the northwest corner of the South Ash Pond, the evaluation of liquefaction potential indicated that the dike fill soil and foundation soils directly underlying the South Ash Pond perimeter dikes were not liquefiable. An evaluation of the liquefaction safety factor for a critical cross section through the liquefiable zone (Cross Section A) was conducted 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" for the South Ash Pond at a later time.

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

Based on the evaluations presented within this Safety Factor Assessment Report, the South Ash Pond meets or exceeds 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. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters		Undrained Parameters[1]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill	120[2]	27 to 36[3]	0	-	-
Dike Fill (Post-Liquefaction)	120	0	30		
Clayey Foundation Soils	94[2]	15	300	Varies[4]	300
Sandy Foundation Soils	123[2]	30 to 32[3]	0	-	-
Chicora	130[2]	50[2]	0	-	-
Williamsburg Formation Clay	105[2]	50[2]	0	-	-
Fly Ash	100[2]	34[2]	0	-	-
Riprap Buttress	150	45	0	-	-

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only.
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 explained in Attachment 5.
3. These drained shear strengths (ϕ') vary by location. Interpretation of in-situ results applied in the selection is provided in Attachment 8.
4. The selected undrained strength ratio (S_u/σ'_{vo}) varies between locations and ranges from 0.25 to 0.70 for the selected cross sections. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in Attachment 5 and Attachment 8.

Table 2. Calculated Maximum Shear Stress Envelopes

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	41	2.5	36
7.5	96	7.5	80
12.5	124	12.5	110
16.5	146	16.5	135
19.5	171	20.5	160
23.5	192	25.5	182
28.5	204	30.5	195
33.5	213	35.5	205
38.0	272	40.5	214
42.0	306	45.5	223
46.0	331	50.5	283
50.5	364	58.0	383
58.0	457	68.0	488
68.0	572	78.0	555
78.0	682	88.0	679
88.0	789	98.0	758
98.0	929	108.0	892
108.0	1064		

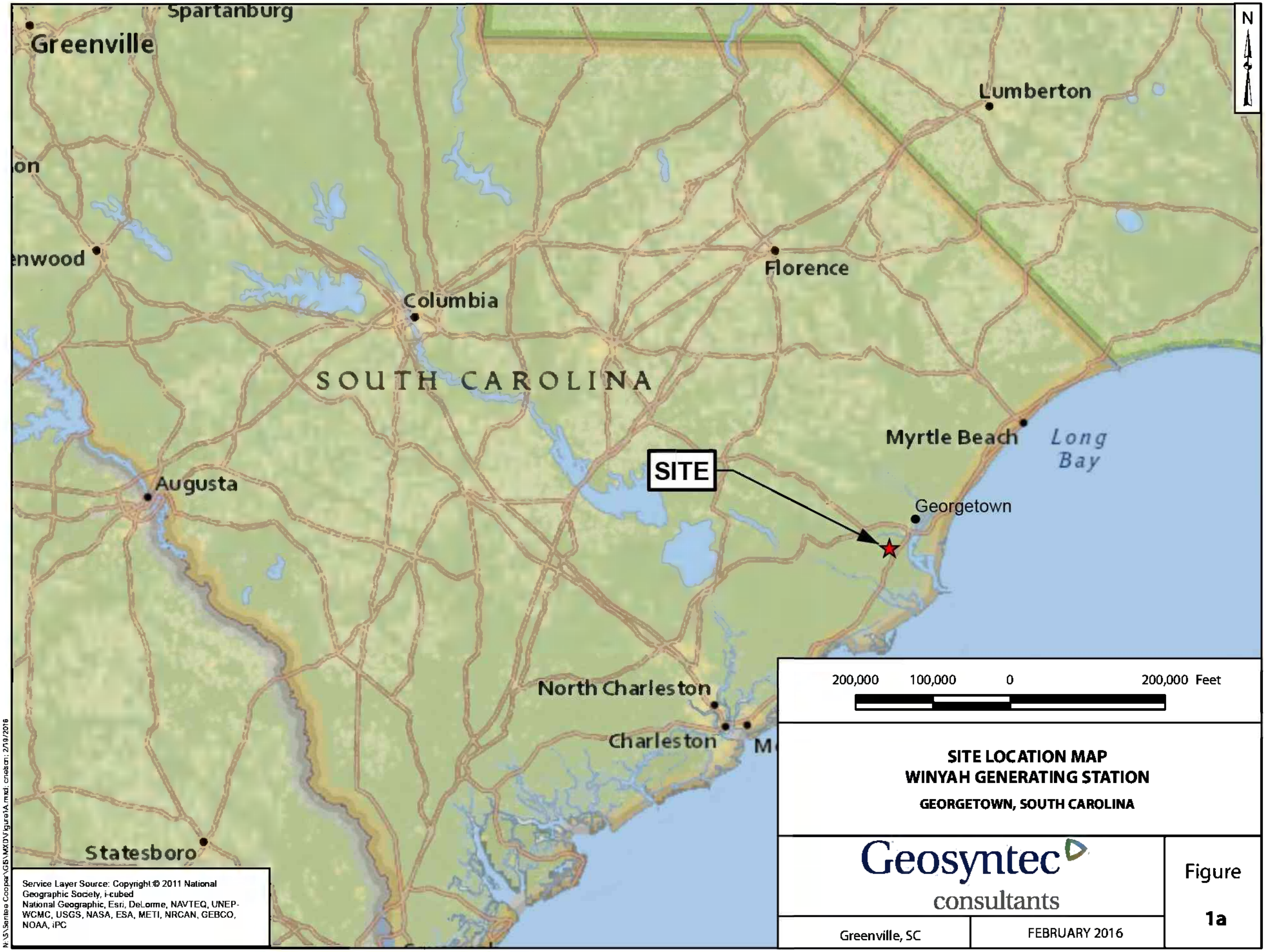
Table 3. Summary of Safety Factor Analysis Results

Safety Factor Case	**Targe t FS**	**Cross Section A**	**Cross Section B**	**Cross Section C**	**Cross Section D**	**Cross Section E**
Static - Maximum Normal Storage Pool	1.50	*1.69*	1.81	1.96	2.05	1.90
Static FS- Maximum Surcharge Pool	1.40	*1.69*	1.71	1.82	2.04	1.90
Seismic - Maximum Normal Storage Pool	1.00	1.09	*1.04*	1.12	1.28	1.26
Liquefaction Slope Stability[1]	1.20	*1.32*	N/A	N/A	N/A	N/A

Notes:

1. The liquefaction potential analysis for Cross Section A is presented in the Liquefaction Package. The liquefaction safety factors for Cross Sections B, C, D, and E were not evaluated as embankment soils were not found to be liquefiable (Liquefaction Package).
2. The lowest computed safety factor for each analysis case is *italicized.* Critical FS's for Cross Sections A and B were found to contain the lowest computed FS's and are shown in Figures 11 through 14.
3. Only critical failure surfaces passing through the perimeter dikes were considered.

FIGURES



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SITE LOCATION MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Greenville, SC

FEBRUARY 2016

Figure
1a

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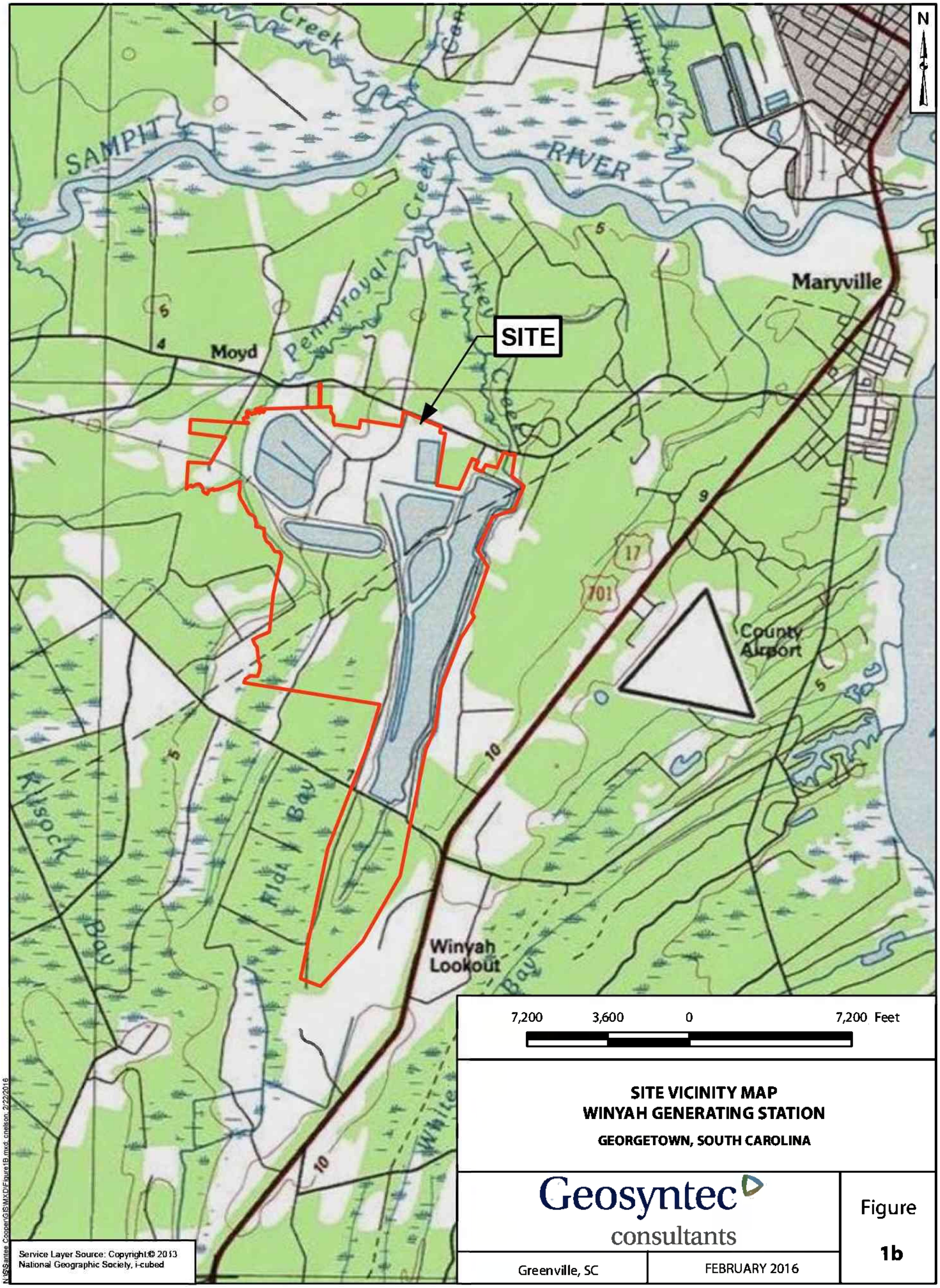

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SITE VICINITY MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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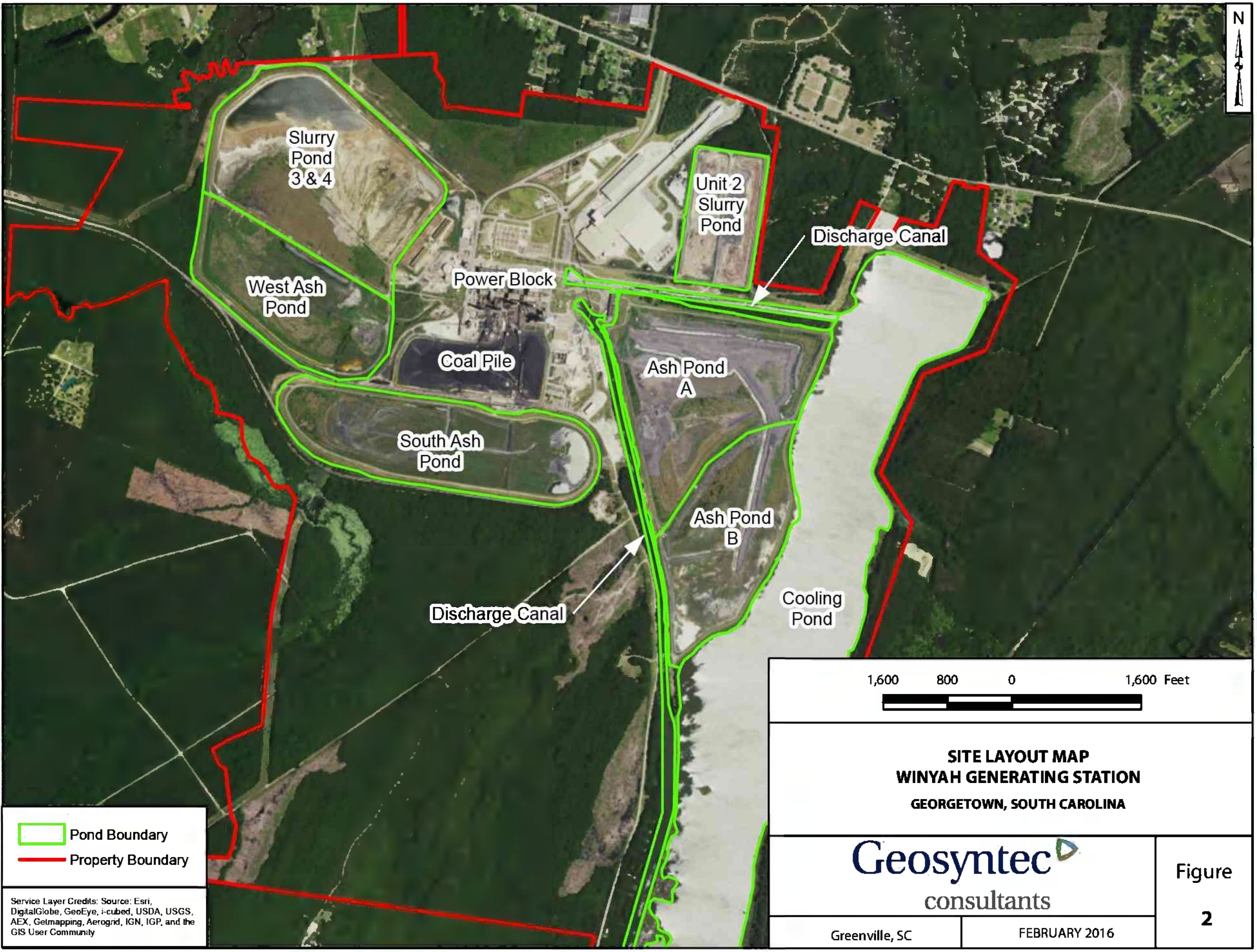
Figure
1b



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SITE LAYOUT MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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Greenville, SC

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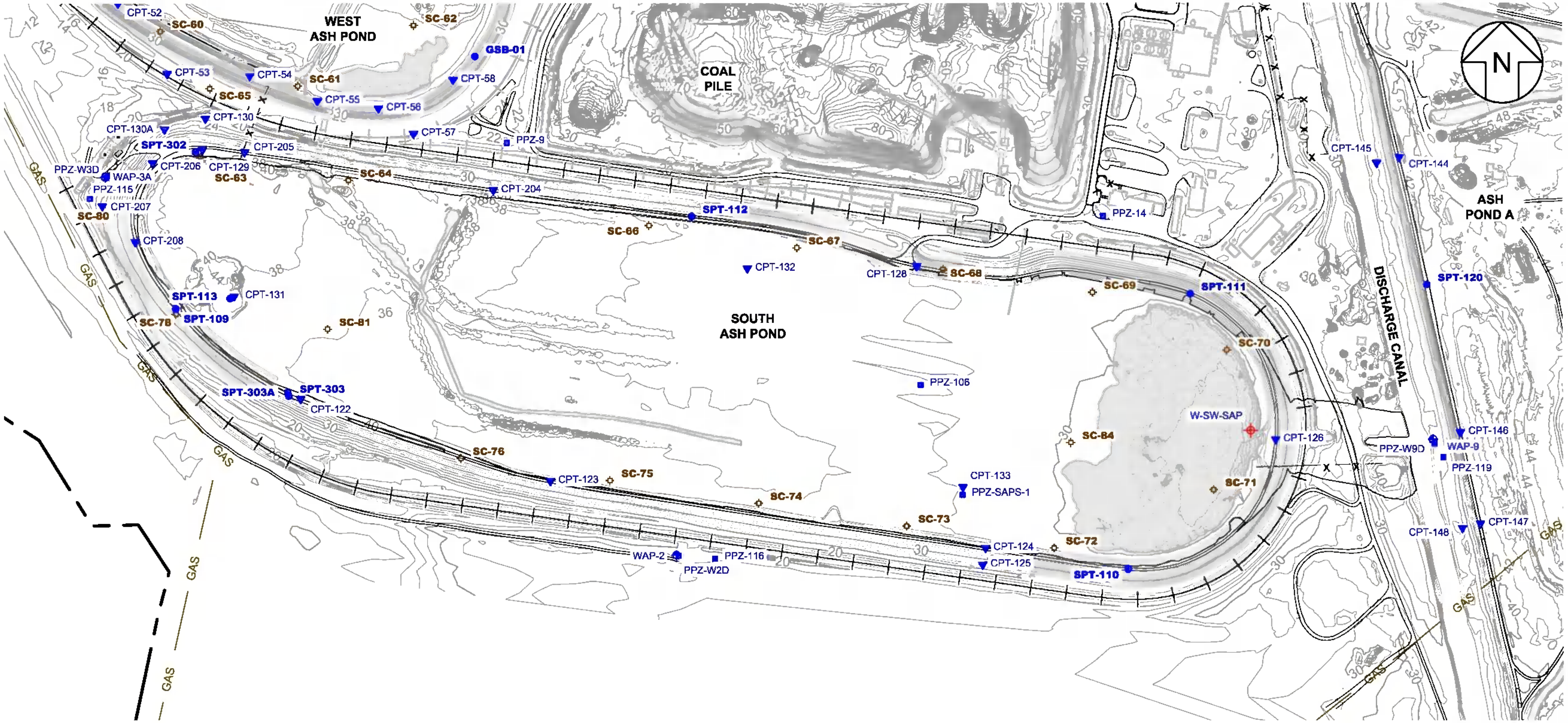
Figure
2



LEGEND

—— GAS ——— GAS ——	EXISTING GAS LINE
——— 10 ———	EXISTING MAJOR GRADE CONTOUR
┼──┼──┼──┼	EXISTING RAILROAD
▨	EXISTING PONDED WATER (28.73 NGVD) (2016)
⌖ W-SW-SAP	EXISTING STAFF GAUGE
▼ CPT-52	GEOSYNTEC CONE PENETRATION TEST
● **SPT-110**	GEOSYNTEC SOIL BORING
⌖ **SC-60**	HISTORICAL BORING
◆ WAP-9	MONITORING WELL
■ PPZ-9, PPZ-SAPS-1, PPZ-W3D	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.
4. HISTORICAL BORING SC-77 IS NOT SHOWN ON THIS MAP AS ITS LOCATION IS NOT KNOWN. IT IS ASSUMED TO BE A SOUTH ASH POND BORING BASED ON ITS NUMERICAL SEQUENCE RELATED TO OTHER SOUTH ASH POND BORINGS.

0 400' 800'

SCALE IN FEET

SOUTH ASH POND
BORING LOCATION MAP

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FIGURE
3

PROJECT NO: GSC5242

FEBRUARY 2016

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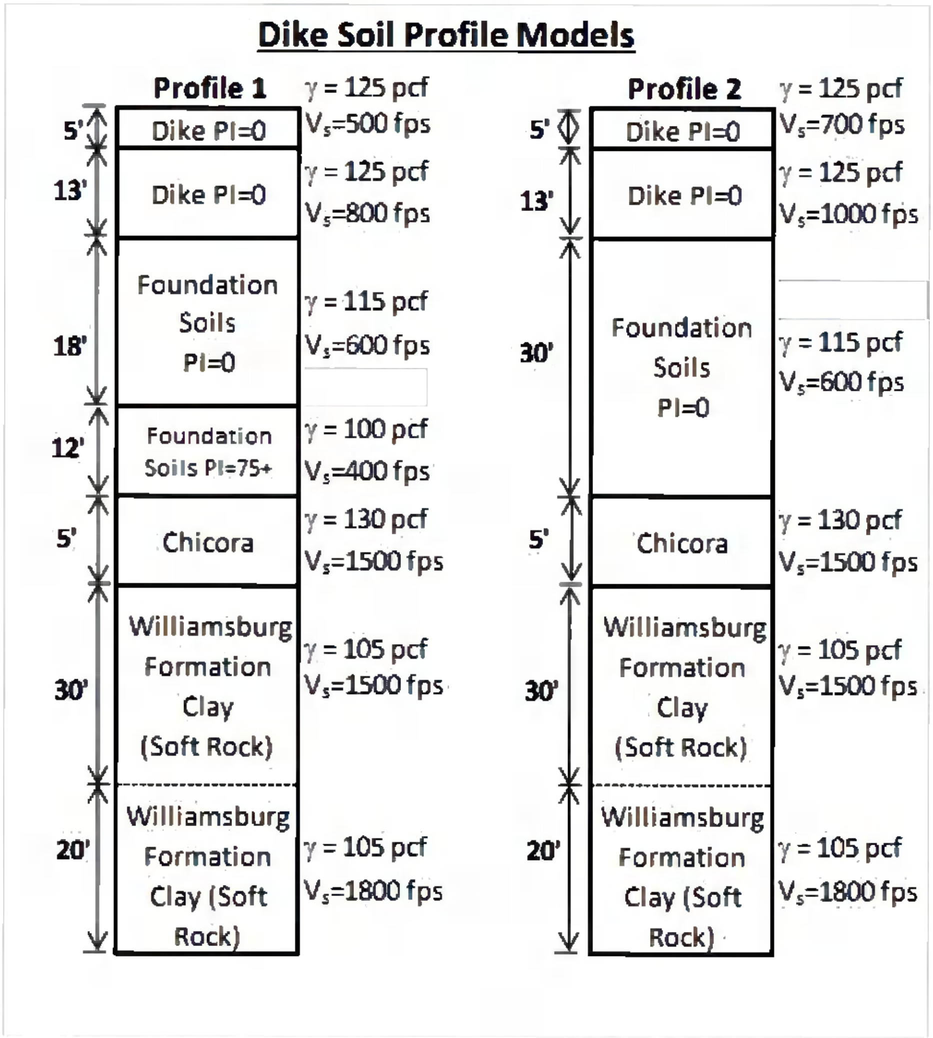
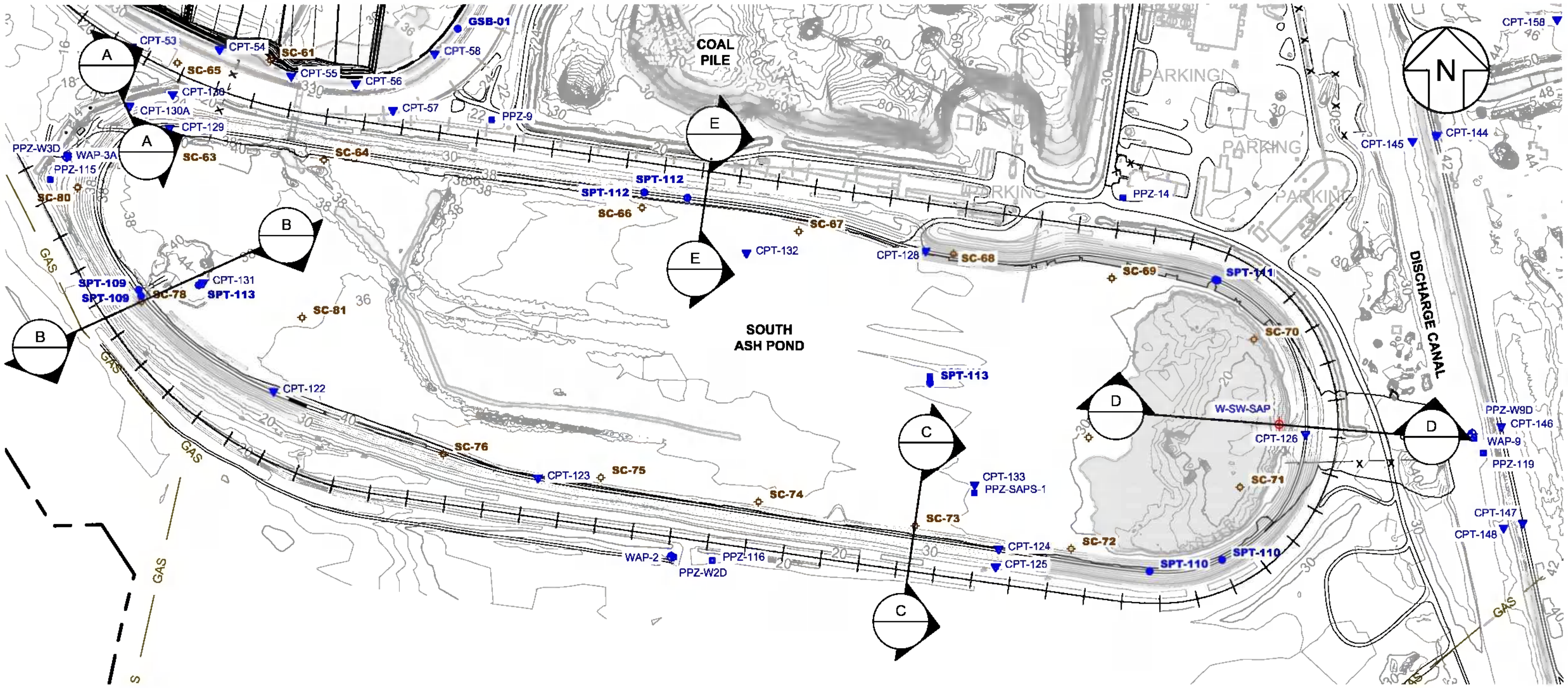


Figure 4. Representative Subsurface Profiles for Site Response Analysis



LEGEND

── 10 ──	EXISTING MAJOR GRADE CONTOUR
┼─┼─┼─┼	EXISTING RAILROAD
A ─── A	PROPOSED SECTION
▨	EXISTING PONDED WATER (28.73 FT NGVD)
⌖ W-SW-SAP	EXISTING STAFF GAUGE
✧ SC-64	BORING BY OTHERS
▼ CPT-05	GEOSYNTEC CONE PENETRATION TEST
● GSB-01 ● SPT-109	GEOSYNTEC SOIL BORING
◈ WAP-2	MONITORING WELL
■ PPZ-9, PPZ-W2D	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

WGS - SOUTH ASH POND
CROSS SECTION LOCATION MAP

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FIGURE
5

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FEBRUARY 2016

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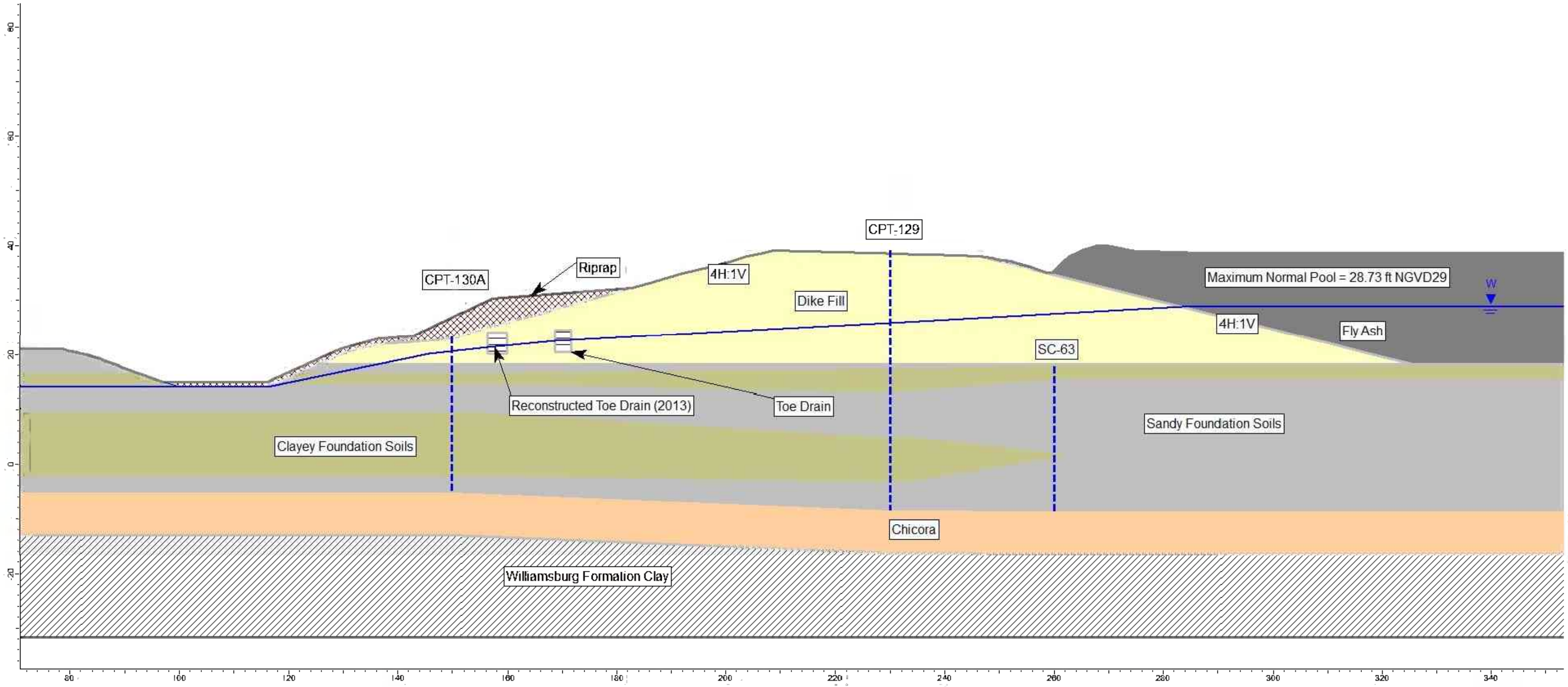


Figure 6. Cross Section A Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE$^{®}$)

Note:

1. A riprap buttress was constructed against the downgradient dike slope and across the adjacent drainage swale in the vicinity of the computed liquefiable zone (Attachment 7).

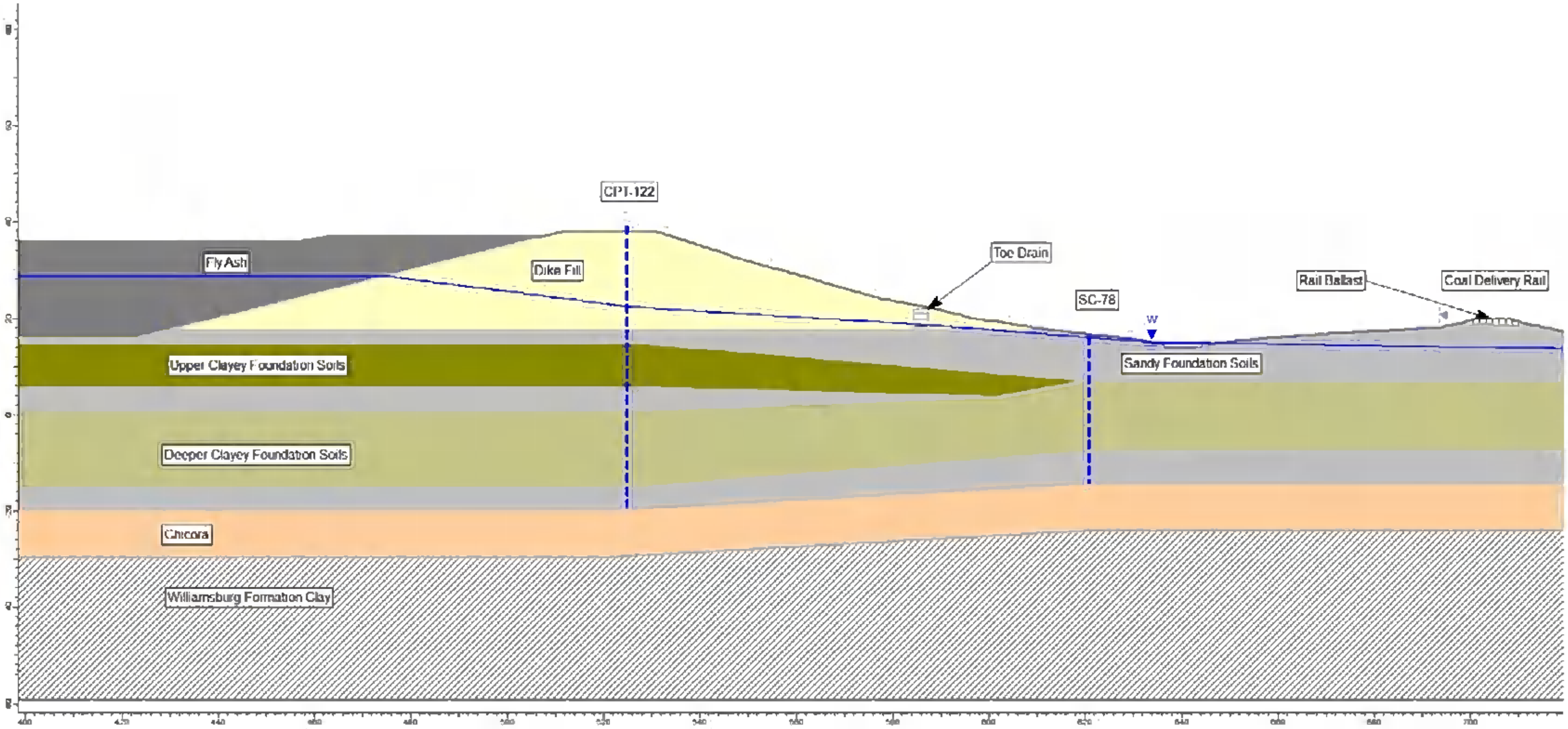


Figure 7. Cross Section B Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

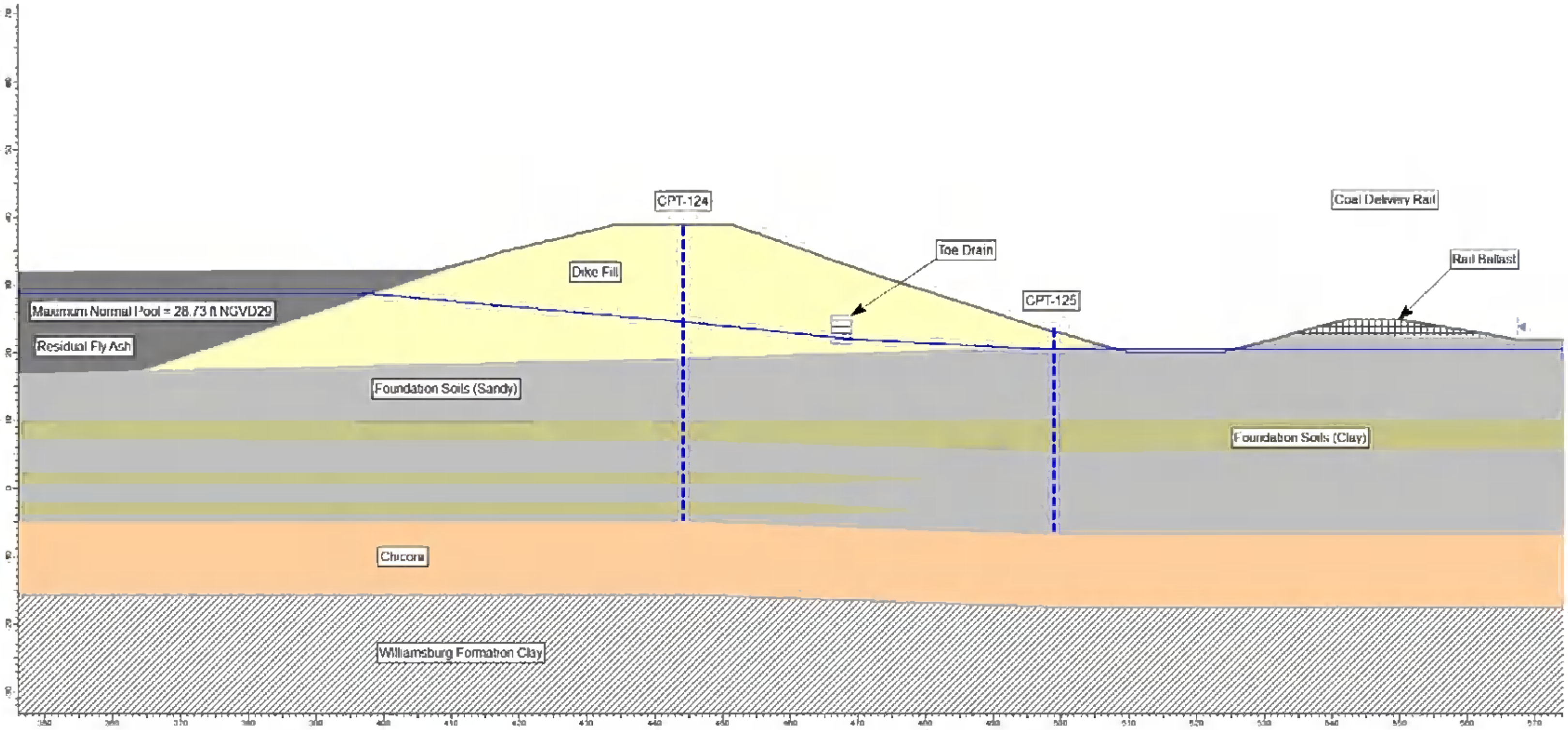


Figure 8. Cross Section C Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

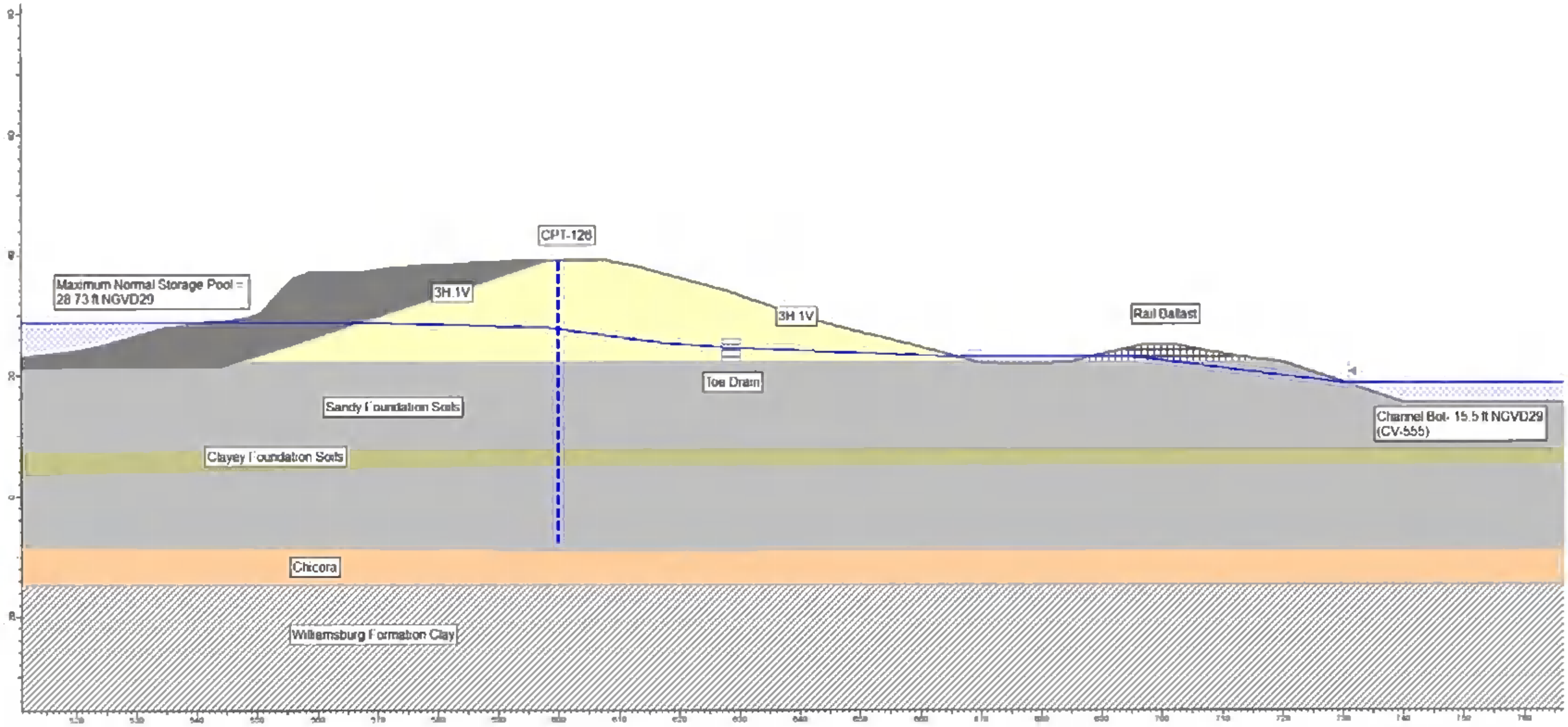


Figure 9. Cross Section D Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)

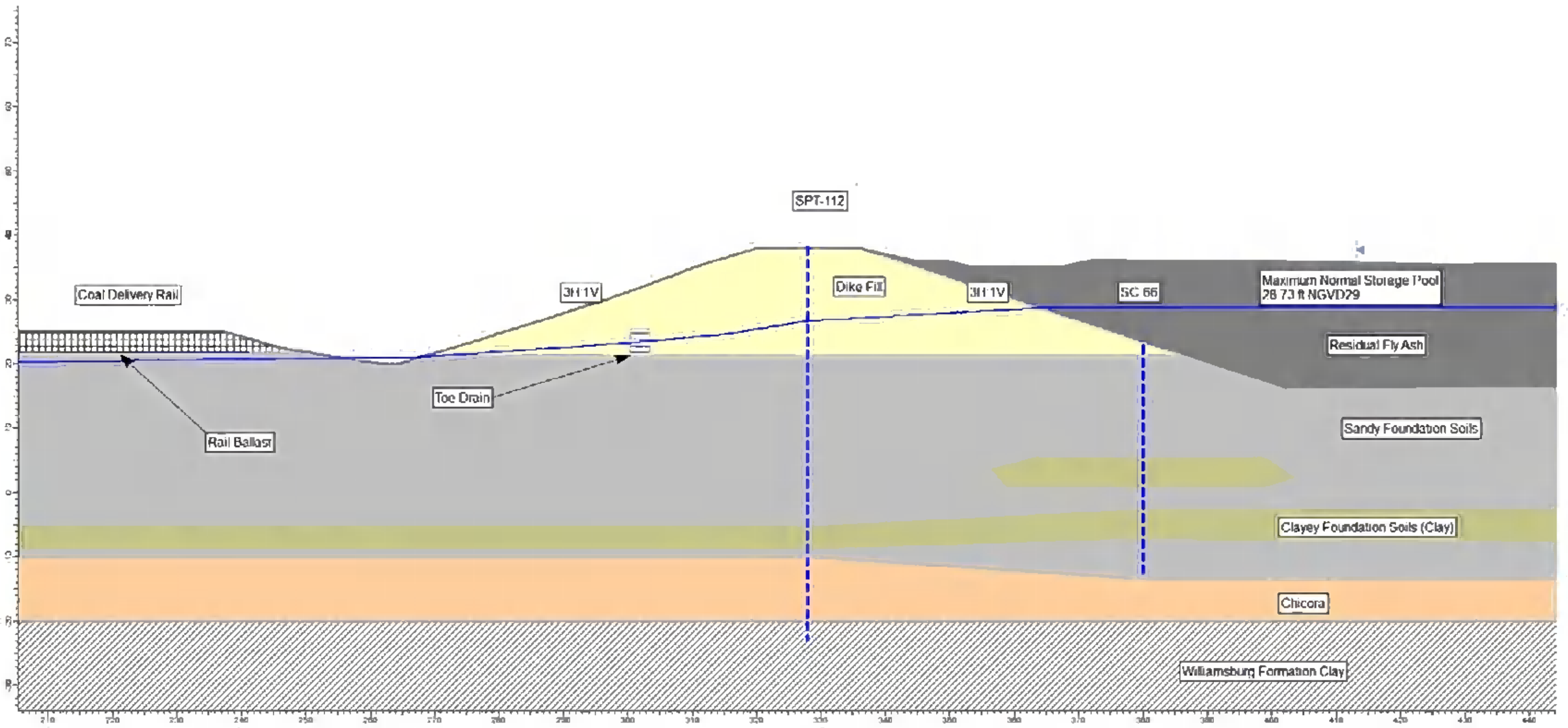
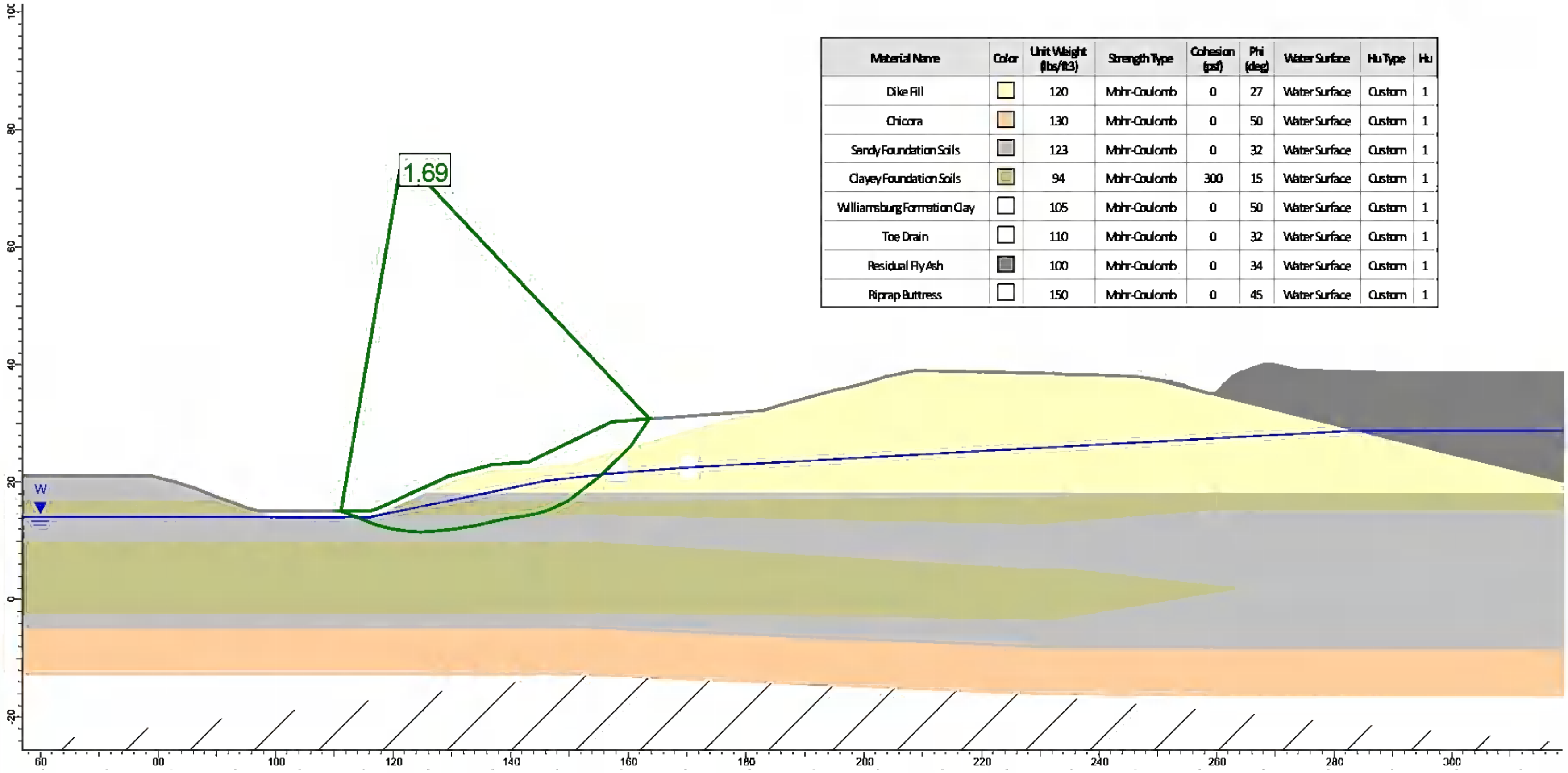
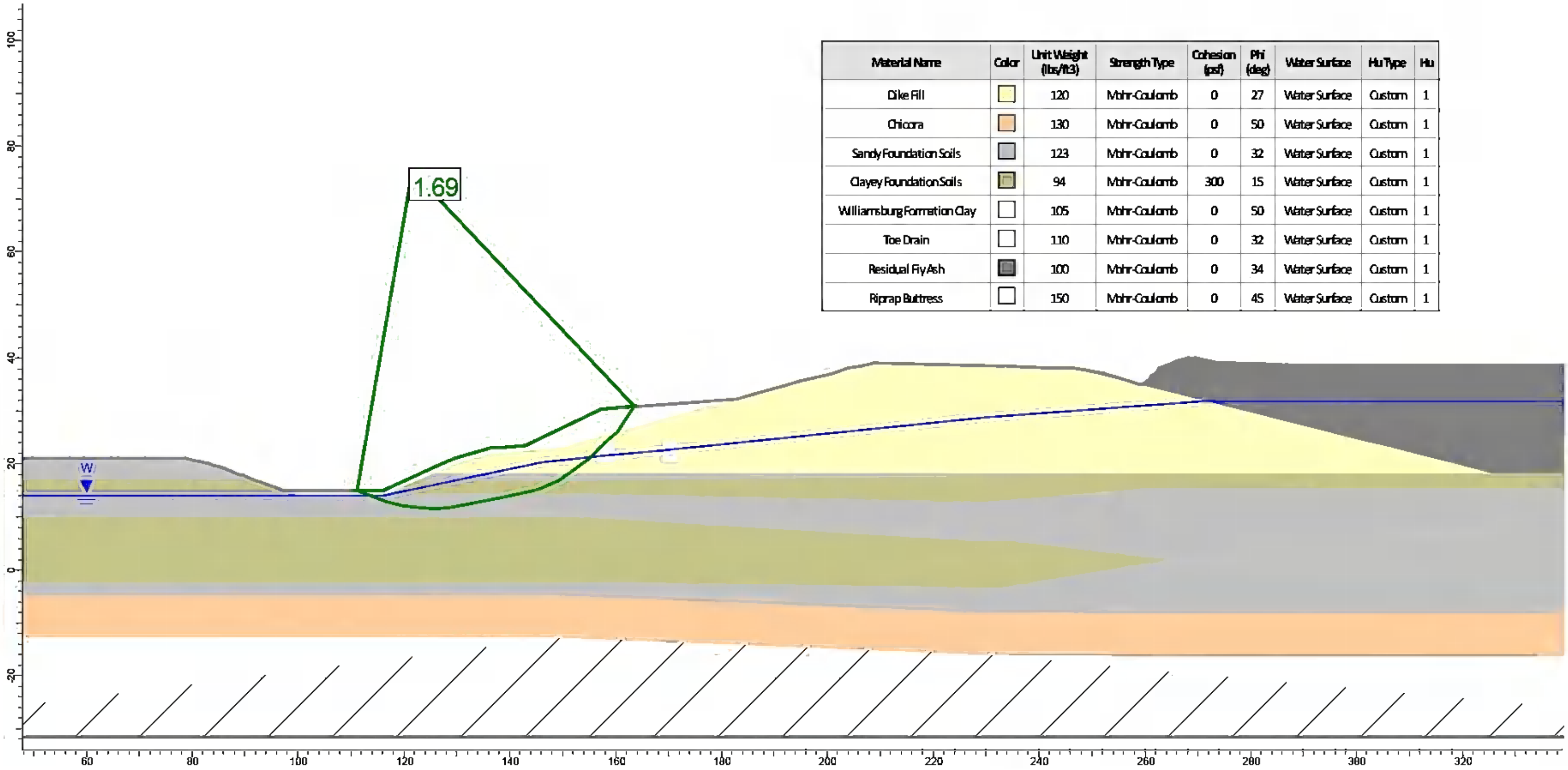


Figure 10. Cross Section E Geometry during Maximum Normal Operating Pool Conditions (as implemented within SLIDE®)



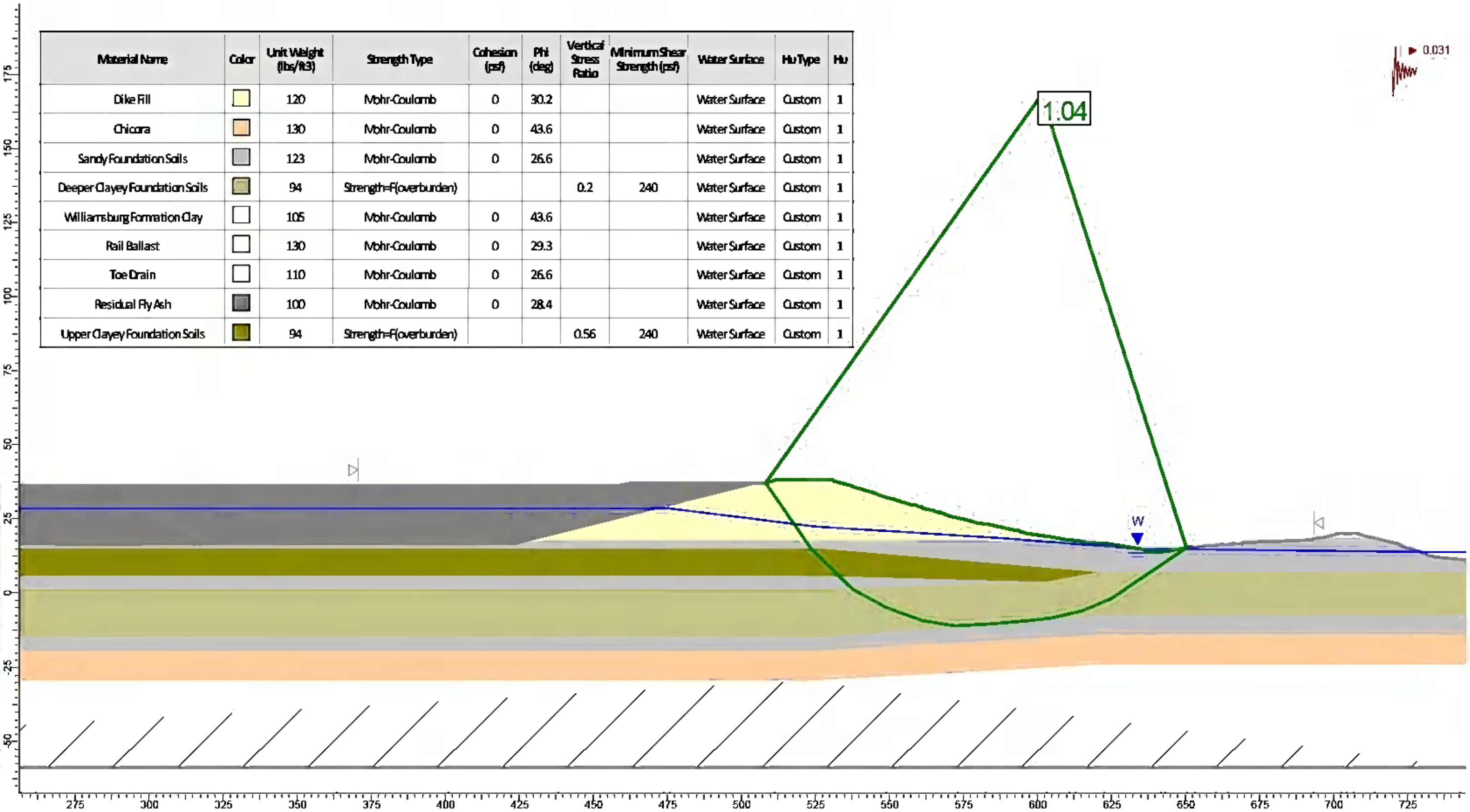
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	🟩	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	☐	150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 11. Critical Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool



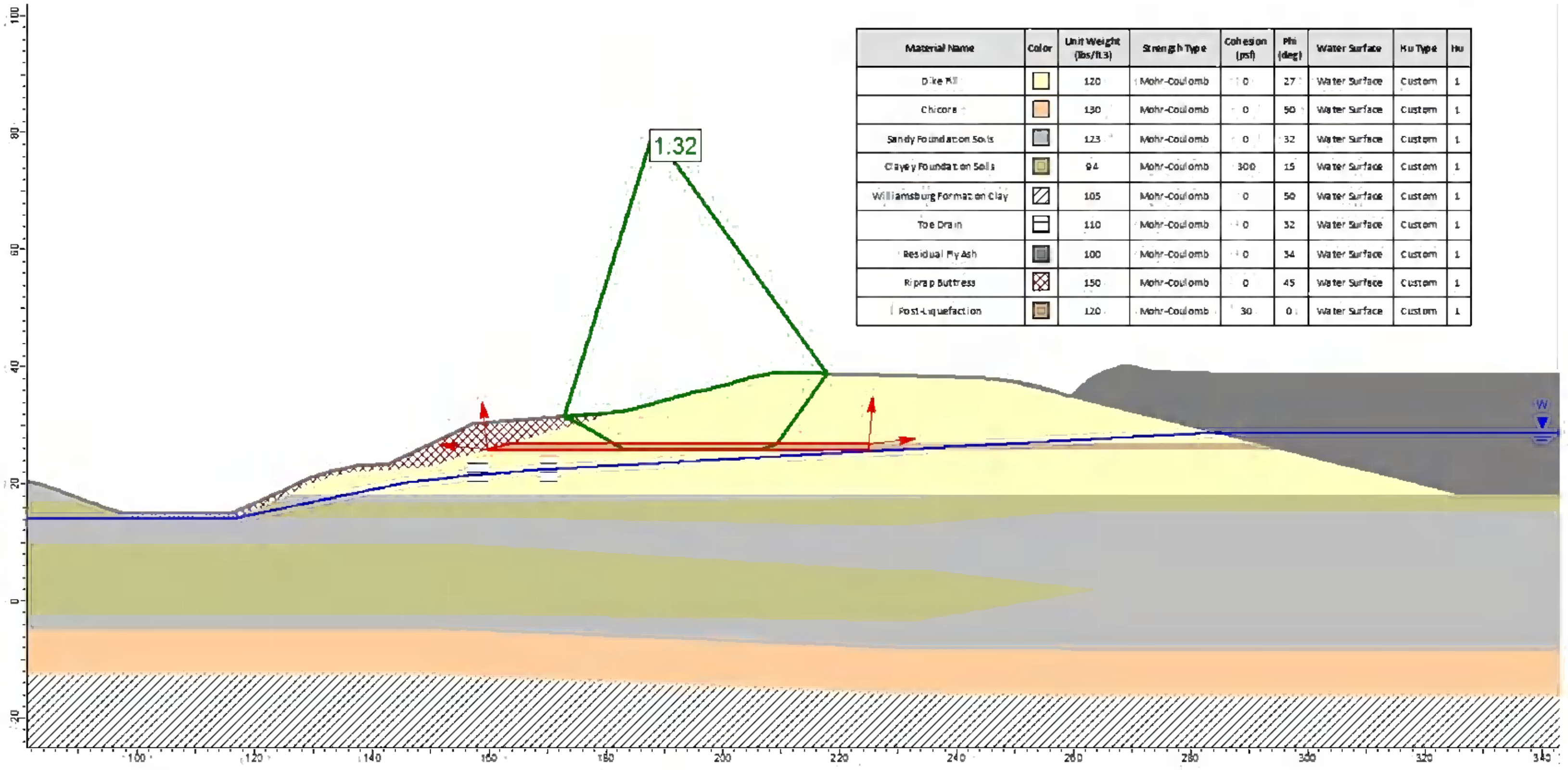
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	▧	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	☐	150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 12. Critical Factor of Safety for Cross Section A: Static Factor of Safety - Maximum Surcharge Pool



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	30.2			Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Deeper Clayey Foundation Soils	▨	94	Strength=F(overburden)			0.2	240	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Rail Ballast	☐	130	Mohr-Coulomb	0	29.3			Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	28.4			Water Surface	Custom	1
Upper Clayey Foundation Soils	■	94	Strength=F(overburden)			0.56	240	Water Surface	Custom	1

Figure 13. Critical Factor of Safety for Cross Section B: Seismic Factor of Safety – Maximum Normal Storage Pool



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	⬜	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	🟩	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	▨	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	⊟	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	⬛	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	▩	150	Mohr-Coulomb	0	45	Water Surface	Custom	1
Post-Liquefaction	🟫	120	Mohr-Coulomb	30	0	Water Surface	Custom	1

Figure 14. Critical Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool – Liquefaction

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: South Ash Pond**

Computations by: Signature Sarah M. Herr 2/9/16
Printed Name Sarah Herr Date
Title Senior Staff Engineer

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

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

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

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

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	**S. Herr**	Date:	**10/7/16**	Reviewed by:	**B. Wallace**	Date:	**10/7/16**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GSC5242**	Task No.:	**01**

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

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 the South Ash Pond 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 Residuals (CCR) Rule. The South Ash Pond 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 the South Ash Pond is a low hazard surface impoundment, the 100 yr storm frequency is analyzed herein.

The South Ash Pond, encompassing approximately 76 acres (ac), is situated immediately south of the Coal Pile and power block and is encircled by a railroad that loops around the pond (Thomas and Hutton, 2012). (Note that 76 ac is the area contained within the dike crest boundary. The area of the limits of CCR is slightly less at approximately 75 ac.) The northern extent of the South Ash Pond is bounded by the rail line and Coal Pile, while the southern extent is bounded by a forested area. To the west, the South Ash Pond is bounded by Pennyroyal Creek and is bordered to the east by an access road and the Discharge Channel. The maximum height of the South Ash Pond perimeter dike is 22 feet (ft) (Thomas and Hutton, 2012). The minimum crest elevation of the South Ash Pond perimeter dikes is 36.9 ft National Geodetic Vertical Datum of 1929 (NGVD 29) (Thomas and Hutton, 2016). A Site Map including the surface impoundment and hydraulic features associated with the South Ash Pond is provided in **Figure 1**.

The South Ash Pond impounds CCRs in the form of fly ash, boiler slag, and bottom ash. The South Ash Pond also receives low volume wastewater and other process water inflows described herein. Additionally, the South Ash Pond receives contact stormwater from the Coal Pile. Decanted water is discharged through a riser structure and outlet pipe approximately 350 ft in length to the Discharge Canal (Lockwood Greene, 1978).

METHODOLOGY

Stormwater runoff volumes and associated discharges to the South Ash Pond 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 Service (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 to the South Ash Pond.

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 inches (in.) (NOAA, 2006). The design storm hyetograph was developed using SCS Type III rainfall distribution and was directly input to the *HydroCAD* model.

Drainage Areas and Curve Numbers

The contributing watershed area for the South Ash Pond is 75.6 ac (Thomas and Hutton, 2012). The area was delineated using the dike crests to correspond to the pond's direct drainage area. The 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. The South Ash Pond was assumed to be approximately 82% ash (CN = 87), 7% sparse vegetation (CN = 68), and 11% water (CN = 100) (Weighted CN = 87). The contributing watershed area and CN is summarized in **Table 1** and was directly input into the *HydroCAD* model.

Times of Concentration 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 the South Ash Pond is characterized by sheet flow and shallow concentrated 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 overland 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 the South Ash Pond. The sheet flow length was limited to 100 ft, because sheet flow beyond 100 ft typically transitions to shallow concentrated flow. 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 the South Ash Pond are shown in **Table 2**.

Shallow concentrated flow travel time was computed using the Upland Method (NRCS, 2010).

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

where: T_t = travel time (seconds [s]);

L = flow length (ft); and

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

The average velocity was computed using the following equation (NRCS, 2010).

$$V = K_v S^{0.5}$$

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

K_v = velocity factor (ft/s); and

S = slope of hydraulic grade line (or land slope) (ft/ft).

A velocity factor of 16.1 ft/s, representing flow across an unpaved surface, was used to calculate shallow concentrated flow travel time within the South Ash Pond. The parameters used to describe shallow concentrated flow within the South Ash Pond are presented in **Table 2**. The computed times of concentration for the South Ash Pond are summarized in **Table 3**.

Inflows

In the *HydroCAD* model, stormwater inflow associated with the South Ash Pond is represented by Sub-Catchment 2S. Pond 1P represents the South Ash Pond. In addition to stormwater inflow, process water is discharged to the South Ash Pond. The process water flows corresponding to Units 3 and 4 hydroveyor water, Units 3 and 4 low volume wastewater, and SEFA Star II Scrubber blowdowns are represented by Nodes 3L, 4L, and 5L, respectively. The base inflows are modeled as 2,180 gallons per minute (gpm) (4.86 cubic feet per second [cfs]) from the Units 3 and 4 hydroveyor water, 540 gpm (1.20 cfs) from the Units 3 and 4 low volume wastewater, and 20 gpm (4.46E-02 cfs) from the SEFA Star II Scrubber blowdowns (Santee Cooper, 2015). During storm events, the South Ash Pond receives contact stormwater from the Coal Pile, represented by Node 6L in the *HydroCAD* model. This base inflow is modeled as 2,450 gpm (5.46 cfs) (Santee Cooper, 2014). The *HydroCAD* model routing diagram is provided in **Appendix A**.

Storage Capacities

The available stormwater storage volume of the South Ash Pond between elevations 12.0 ft and 36.9 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). The lowest contour within the South Ash Pond is 12.0 ft NGVD 29. The minimum crest elevation of the South Ash Pond perimeter dikes is 36.9 ft NGVD 29. The surface area of each contour was measured and tabulated at each elevation. The available surface water volume in each 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 the South Ash Pond between these elevations is 295.6 acre-feet (ac-ft). The South Ash Pond is maintained at a normal operational pool elevation of 28.73 ft NGVD 29 (Thomas and Hutton, 2016). As a result, the starting elevation of Pond 1P is set to 28.73 ft NGVD 29. The area-volume data are presented in **Table 4**.

Outlet Structures

The normal operating level in the South Ash Pond is maintained by a rectangular concrete riser structure with 4 ft long stop logs on a single face. The top stop log elevation is 28.73 ft NGVD 29 (Thomas and Hutton, 2016). A 36 in. diameter reinforced concrete pipe with an upstream invert elevation of 16.93 ft NGVD 29 conveys water from the riser structure to the Discharge Canal (Lockwood Greene, 1978; Thomas and Hutton, 2016).

The tailwater effects associated with discharge from the South Ash Pond 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 are represented by Node 7L in the *HydroCAD* model.

RESULTS

The resulting peak water surface elevation and storage volume for the 100 yr storm event is shown in **Table 5**. The South Ash Pond will effectively contain the 100 yr storm event. This hydrologic and hydraulic analysis demonstrates that the South Ash Pond contains the 72 hr duration precipitation event for the 100 yr frequency return period assuming the South Ash Pond is maintained at a normal operating elevation of 28.73 ft NGVD 29.

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Thomas and Hutton. (2016). *Topographic Survey of the Dike Crests at Santee Cooper Winyah Generating Station.*

TABLES

Table 1 – Watershed Area and Curve Number

Drainage Basin	Area (ac)	Weighted Curve Number (--)
South Ash Pond	75.614	87

Table 2 – Input Parameters Describing Sheet Flow and Shallow Concentrated Flow

Flow Path	**Sheet Flow**				**Shallow Concentrated Flow**		
	Land Slope (ft/ft)	*Manning's Roughness Coefficient (--)*	*Flow Length (ft)*	*2 Yr, 24 Hr Rainfall (in.)*	*Flow Length (ft)*	*Land Slope (ft/ft)*	*Velocity Factor (ft/s)*
South Ash Pond							
Sheet	0.0525	0.020	100	4.38	--	--	--
Shallow Concentrated	--	--	--	--	2,200	0.0036	16.1

Table 3 – Times of Concentration

Flow Path	Time of Concentration (minutes [min])
South Ash Pond	
Sheet	1.1
Shallow Concentrated	38.0

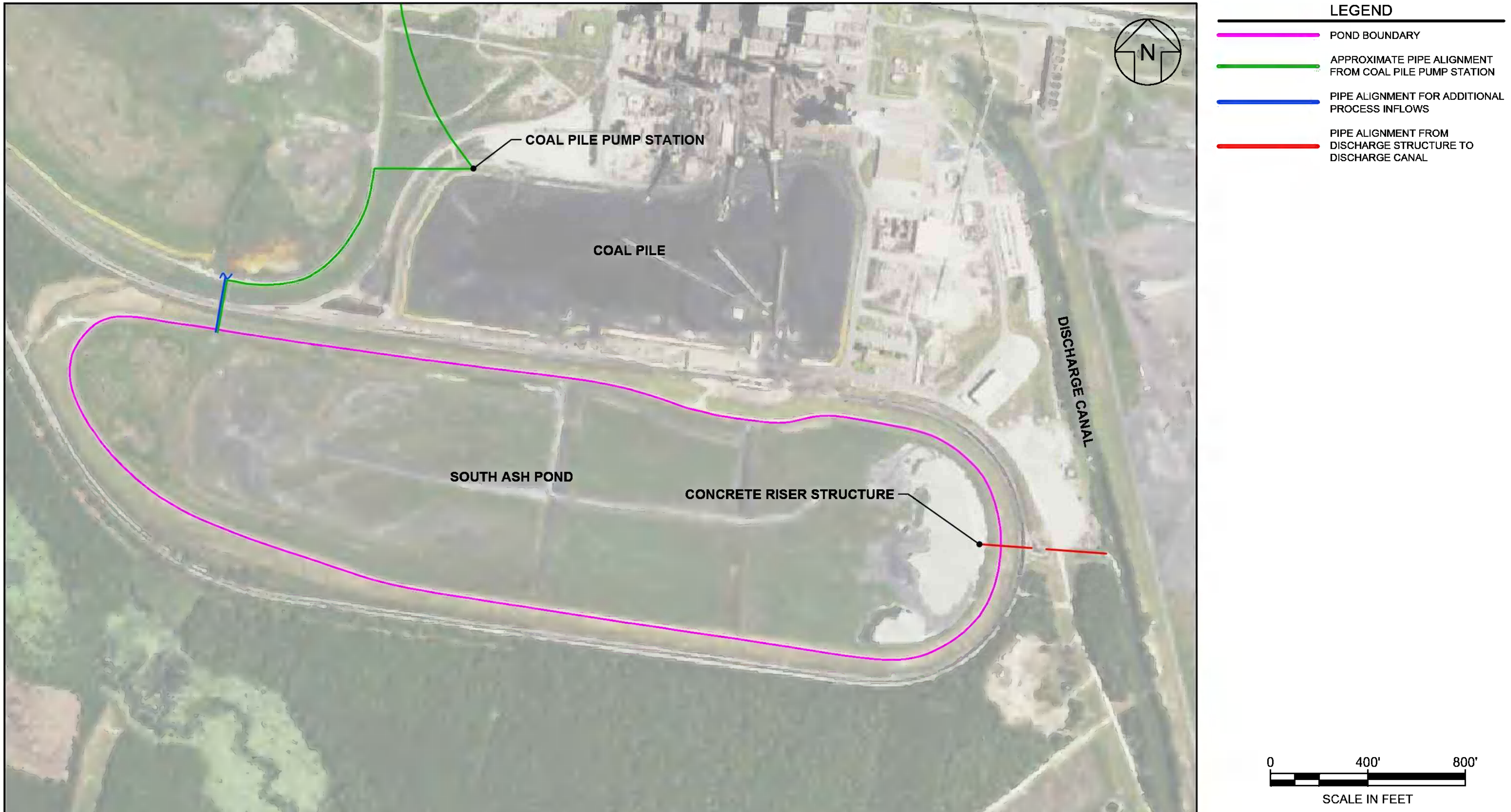
Table 4 – Stage Storage Table (Thomas and Hutton, 2012; Thomas and Hutton, 2016)

South Ash Pond			
Elevation (NGVD 29) (ft)	*Area (ac)*	*Volume (ac-ft)*	*Cumulative Volume (ac-ft)*
36.9	60.2	52.4	295.6
36	56.3	97.3	243.2
34	41.0	61.8	145.9
32	20.8	31.1	84.0
30	10.3	16.7	53.0
28	6.5	11.7	36.2
26	5.2	9.4	24.5
24	4.2	7.2	15.2
22	3.1	5.2	7.9
20	2.1	2.4	2.8
18	0.31	0.35	0.40
16	0.033	0.040	0.048
14	0.007	0.008	0.008
12	0.001	0.000	0.000

Table 5 – Peak Elevations and Volumes

Storm Event	**South Ash Pond**		
	Elevation (NGVD 29) (ft)	*Volume (ac-ft)*	*Time (hr)*
100-Yr, 72-Hr	31.81	80.191	38.08

FIGURES



WINYAH GENERATING STATION
SITE MAP

Geosyntec consultants

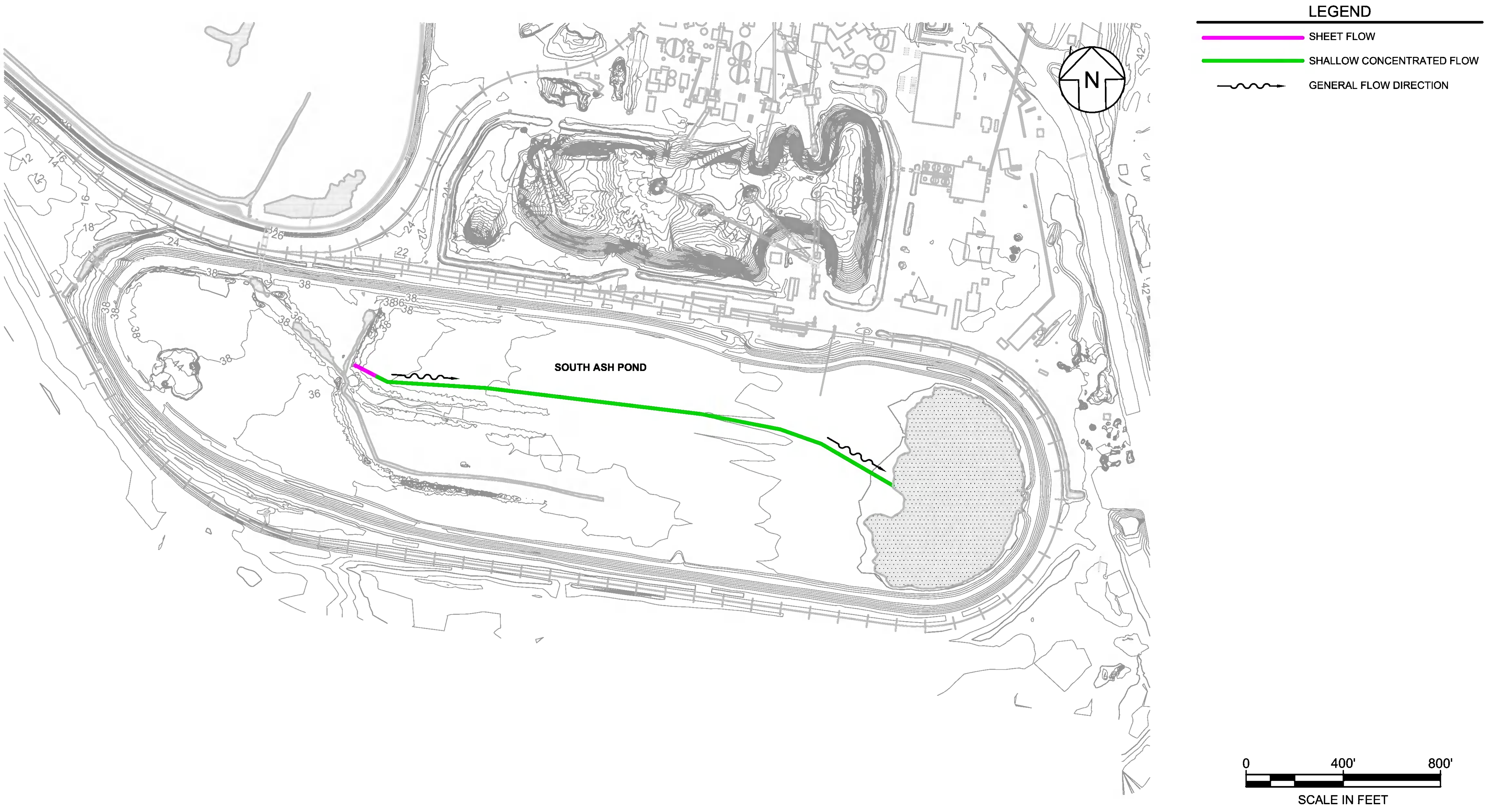
FIGURE

1

PROJECT NO: GSC5242

FEBRUARY 2016

M:\S\SANTEE COOPER\SANTEE COOPER-WINYAH\0028-WINYAH H&H ANALYSES\FIGURES\F-0-SC-585-00-F0028-020



WINYAH GENERATING STATION
SOUTH ASH POND FLOW PATH

Geosyntec
consultants

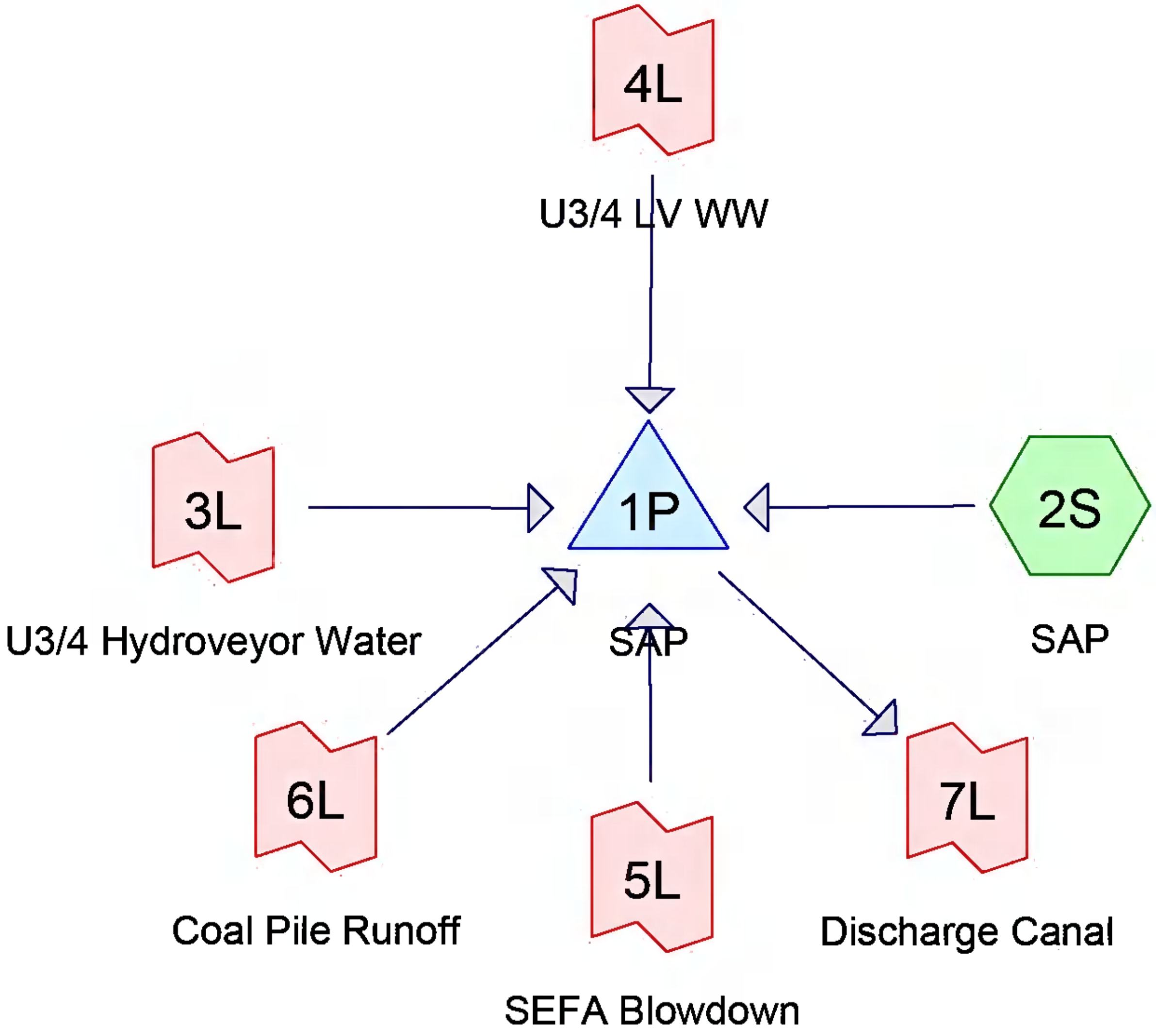
FIGURE
2

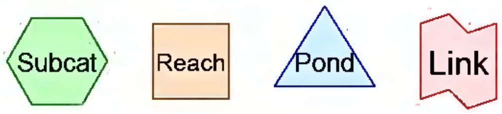
PROJECT NO: GSC5242
FEBRUARY 2016

M:\S\SANTEE COOPER\SANTEE COOPER-WINYAH\0028-WINYAH H&H ANALYSES\FIGURES\F-0-SC-585-00-F0028-021

APPENDICES

APPENDIX A





Routing Diagram for South Ash Pond
Prepared by Geosyntec Consultants, Printed 10/11/2016
HydroCAD® 10.00-15 s/n 00939 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
75.614	87	11% water, 82% CCR, 7% shrubs (2S)

Time span=0.00-999.00 hrs, dt=0.01 hrs, 99901 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: SAP	Runoff Area=75.614 ac 0.00% Impervious Runoff Depth=11.17" Flow Length=2,300' Tc=39.1 min CN=87 Runoff=245.16 cfs 70.362 af
Pond 1P: SAP	Peak Elev=31.81' Storage=80.191 af Inflow=256.73 cfs 930.950 af Outflow=59.83 cfs 930.936 af
Link 3L: U3/4 Hydroveyor Water	Manual Hydrograph Inflow=4.86 cfs 361.492 af Primary=4.86 cfs 361.492 af
Link 4L: U3/4 LV WW	Manual Hydrograph Inflow=1.20 cfs 89.257 af Primary=1.20 cfs 89.257 af
Link 5L: SEFA Blowdown	Manual Hydrograph Inflow=0.05 cfs 3.719 af Primary=0.05 cfs 3.719 af
Link 6L: Coal Pile Runoff	Manual Hydrograph Inflow=5.46 cfs 406.120 af Primary=5.46 cfs 406.120 af
Link 7L: Discharge Canal	Inflow=59.83 cfs 930.936 af Primary=59.83 cfs 930.936 af

Summary for Subcatchment 2S: SAP

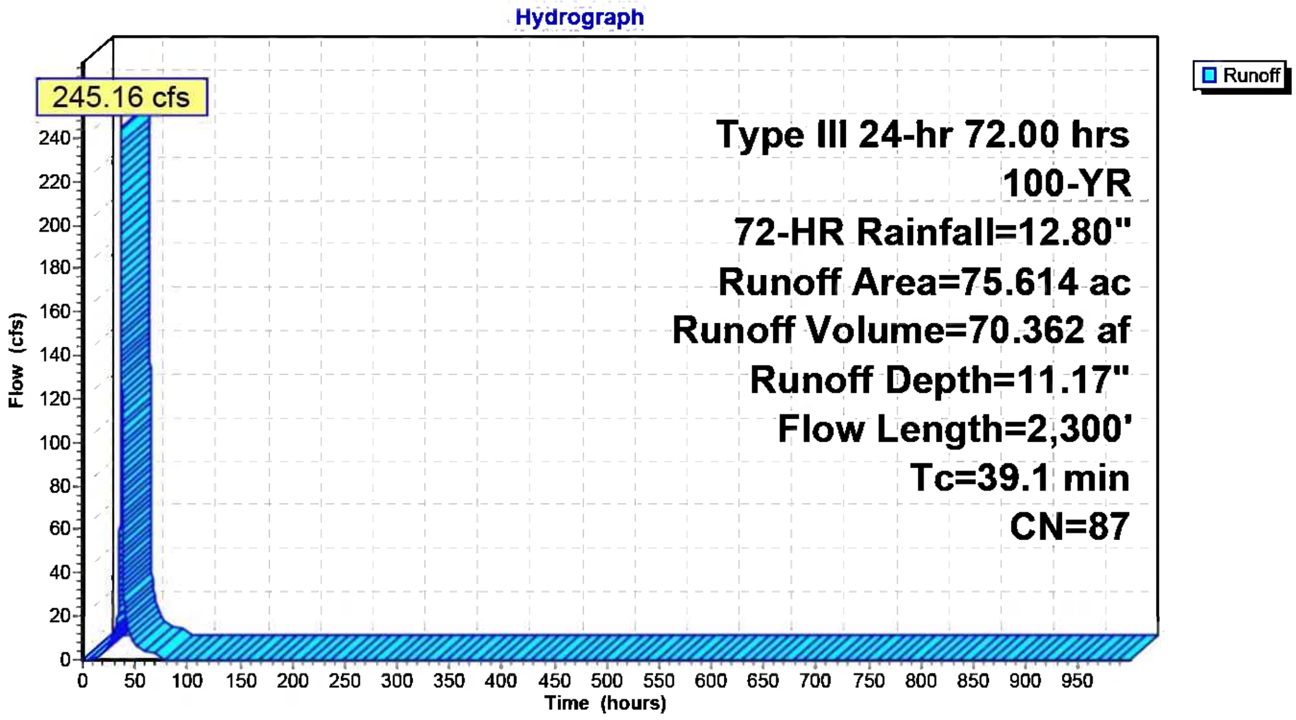
Runoff = 245.16 cfs @ 36.53 hrs, Volume= 70.362 af, Depth=11.17"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs
Type III 24-hr 72.00 hrs 100-YR, 72-HR Rainfall=12.80"

	Area (ac)	CN	Description
*	75.614	87	11% water, 82% CCR, 7% shrubs
	75.614		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1.1	100	0.0525	1.47		**Sheet Flow, Sheet Flow** n= 0.020 P2= 4.38"
38.0	2,200	0.0036	0.97		**Shallow Concentrated Flow, Shallow Concentrated** Unpaved Kv= 16.1 fps
39.1	2,300	Total			

Subcatchment 2S: SAP



Summary for Pond 1P: SAP

Inflow Area = 75.614 ac, 0.00% Impervious, Inflow Depth =147.74" for 100-YR, 72-HR event
Inflow = 256.73 cfs @ 36.53 hrs, Volume= 930.950 af
Outflow = 59.83 cfs @ 38.08 hrs, Volume= 930.936 af, Atten= 77%, Lag= 93.3 min
Primary = 59.83 cfs @ 38.08 hrs, Volume= 930.936 af

Routing by Stor-Ind method, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs
Starting Elev= 28.73' Surf.Area= 7.854 ac Storage= 41.444 af
Peak Elev= 31.81' @ 38.08 hrs Surf.Area= 19.797 ac Storage= 80.191 af (38.747 af above start)

Plug-Flow detention time= 2,926.1 min calculated for 889.478 af (96% of inflow)
Center-of-Mass det. time= 517.5 min (25,653.1 - 25,135.6)

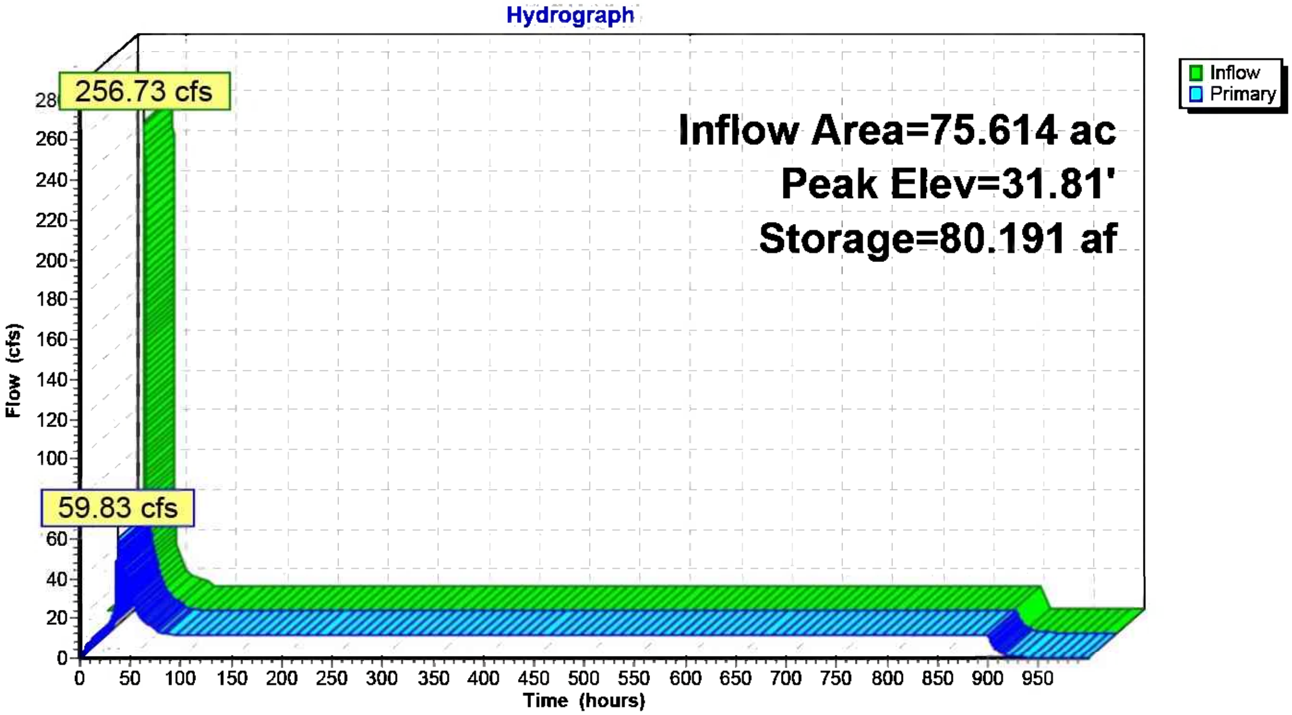
Volume	Invert	Avail.Storage	Storage Description
#1	12.00'	295.584 af	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
12.00	0.001	0.000	0.000
14.00	0.007	0.008	0.008
16.00	0.033	0.040	0.048
18.00	0.314	0.347	0.395
20.00	2.071	2.385	2.780
22.00	3.080	5.151	7.931
24.00	4.152	7.232	15.163
26.00	5.225	9.377	24.540
28.00	6.456	11.681	36.221
30.00	10.286	16.742	52.963
32.00	20.794	31.080	84.043
34.00	41.028	61.822	145.865
36.00	56.283	97.311	243.176
36.90	60.180	52.408	295.584

Device	Routing	Invert	Outlet Devices
#1	Primary	16.93'	**36.0" Round Culvert** L= 350.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 16.93' / 16.93' S= 0.0000 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 7.07 sf
#2	Device 1	28.73'	**4.0' long Sharp-Crested Rectangular Weir** 2 End Contraction(s)

Primary OutFlow Max=59.82 cfs @ 38.08 hrs HW=31.81' (Free Discharge)
↑—1=**Culvert** (Passes 59.82 cfs of 101.09 cfs potential flow)
↑—2=**Sharp-Crested Rectangular Weir** (Weir Controls 59.82 cfs @ 5.74 fps)

Pond 1P: SAP


Hydrograph
256.73 cfs
59.83 cfs
Inflow Area=75.614 ac
Peak Elev=31.81'
Storage=80.191 af
Inflow
Primary
Flow (cfs)
0
20
40
60
100
120
140
160
180
200
220
240
260
28
Time (hours)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950

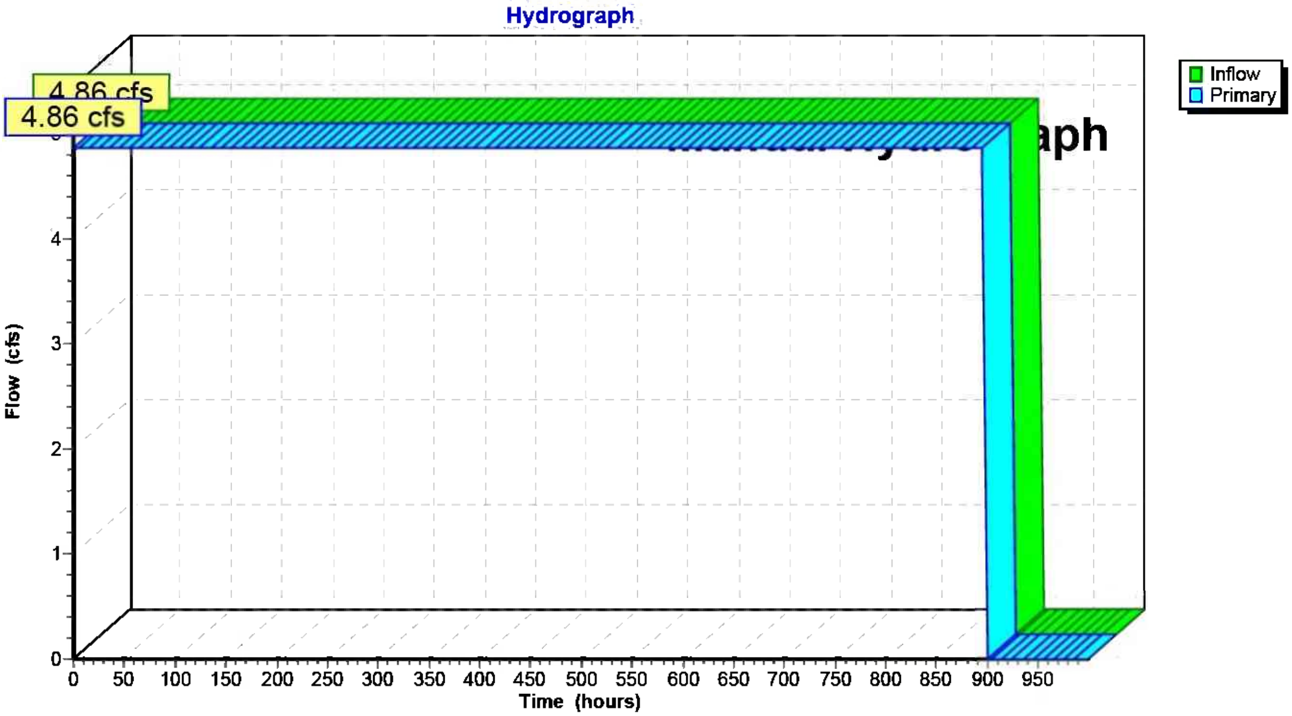
Summary for Link 3L: U3/4 Hydroveyor Water

Inflow = 4.86 cfs @ 0.00 hrs, Volume= 361.492 af
Primary = 4.86 cfs @ 0.00 hrs, Volume= 361.492 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs

19 Point manual hydrograph, To= 0.00 hrs, dt= 50.00 hrs, cfs =
4.86 4.86 4.86 4.86 4.86 4.86 4.86 4.86 4.86 4.86
4.86 4.86 4.86 4.86 4.86 4.86 4.86 4.86 4.86

Link 3L: U3/4 Hydroveyor Water



Summary for Link 4L: U3/4 LV WW

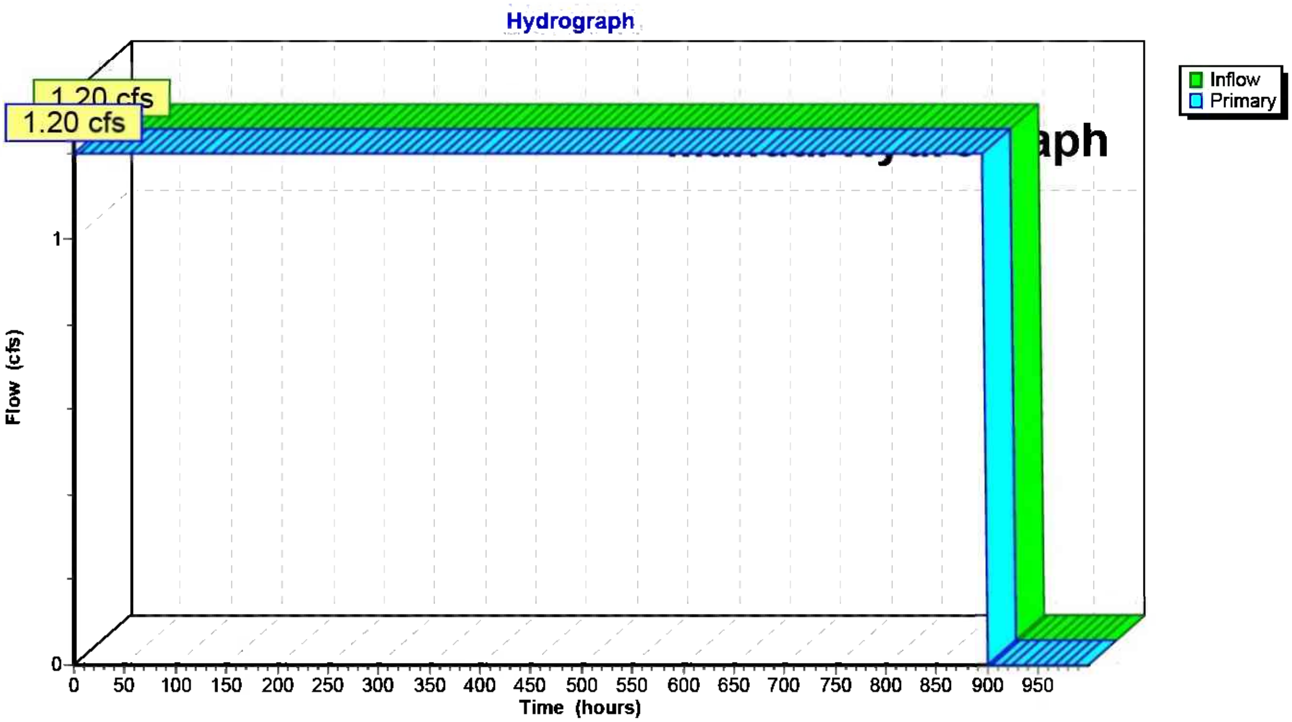
Inflow	=	1.20 cfs @	0.00 hrs, Volume=	89.257 af	
Primary	=	1.20 cfs @	0.00 hrs, Volume=	89.257 af,	Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs

19 Point manual hydrograph, To= 0.00 hrs, dt= 50.00 hrs, cfs =

1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20
1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20 1.20

Link 4L: U3/4 LV WW



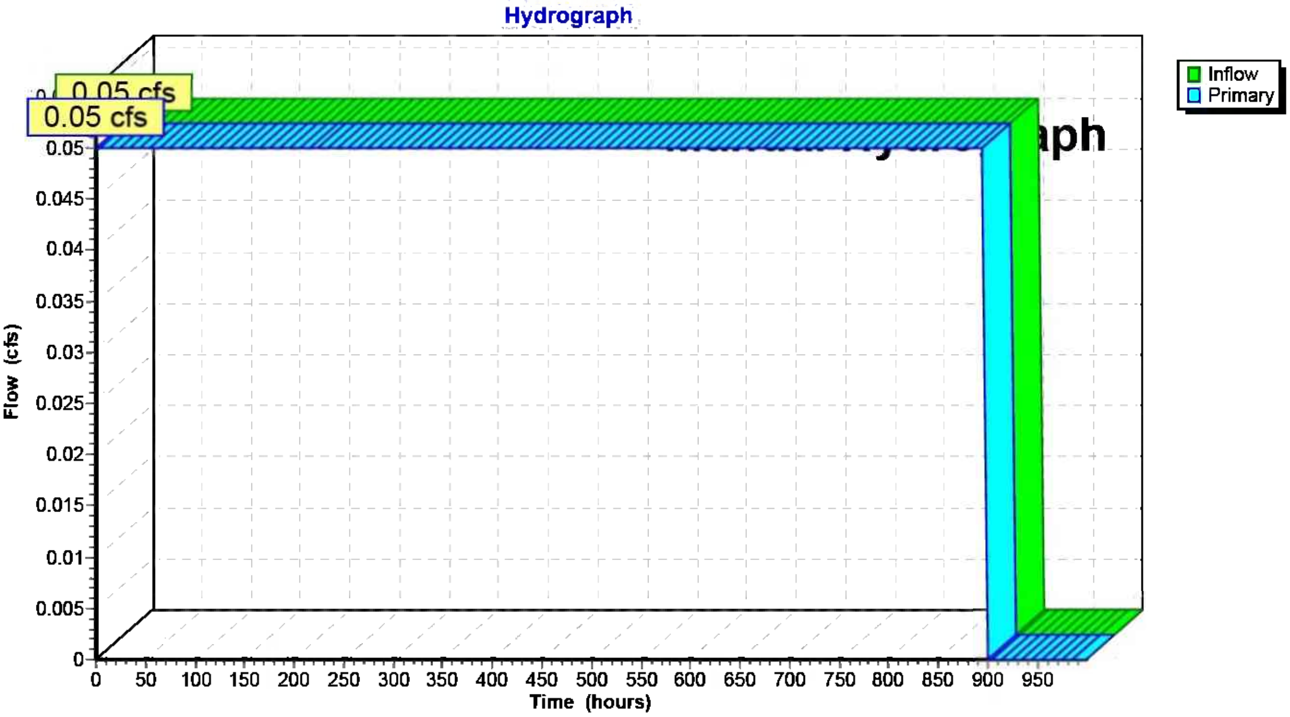
Summary for Link 5L: SEFA Blowdown

Inflow = 0.05 cfs @ 0.00 hrs, Volume= 3.719 af
Primary = 0.05 cfs @ 0.00 hrs, Volume= 3.719 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs

19 Point manual hydrograph, To= 0.00 hrs, dt= 50.00 hrs, cfs =
0.05 0.05 0.05 0.05 0.05 0.05 0.05 0.05 0.05 0.05
0.05 0.05 0.05 0.05 0.05 0.05 0.05 0.05 0.05

Link 5L: SEFA Blowdown



Summary for Link 6L: Coal Pile Runoff

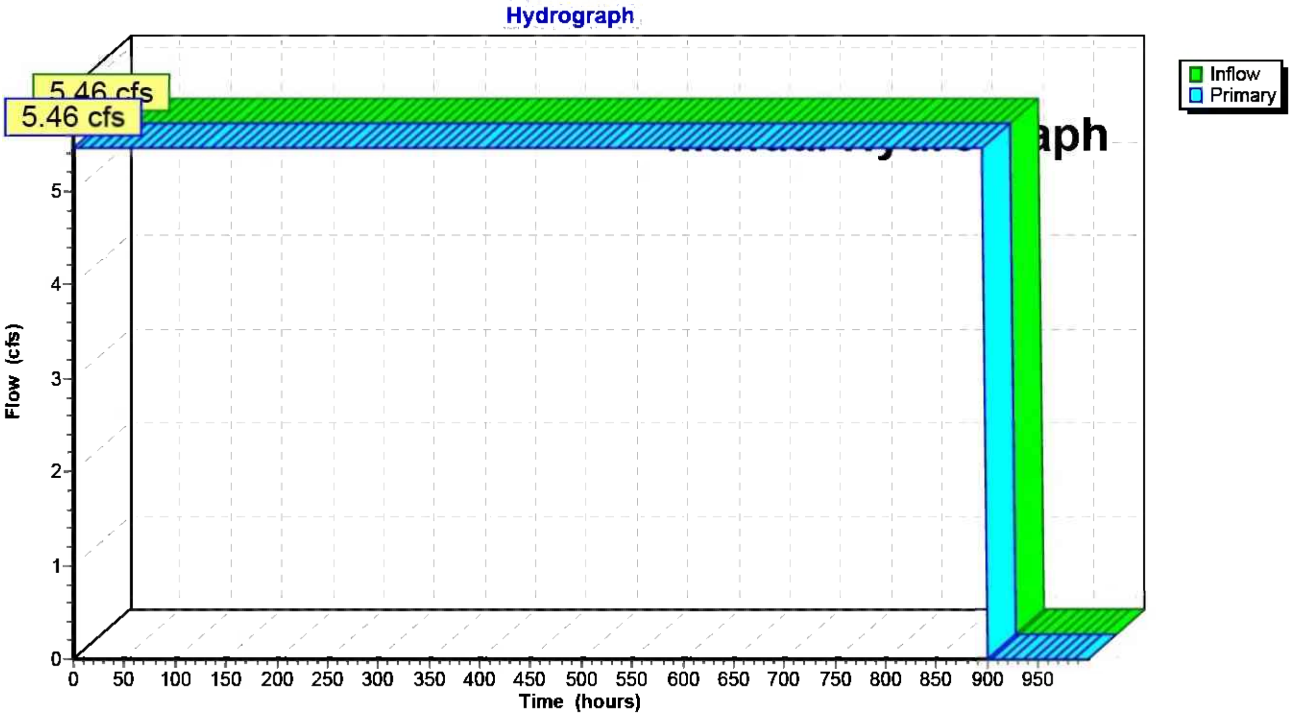
Inflow = 5.46 cfs @ 0.00 hrs, Volume= 406.120 af
Primary = 5.46 cfs @ 0.00 hrs, Volume= 406.120 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs

19 Point manual hydrograph, To= 0.00 hrs, dt= 50.00 hrs, cfs =

5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46
5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	5.46	

Link 6L: Coal Pile Runoff



Summary for Link 7L: Discharge Canal

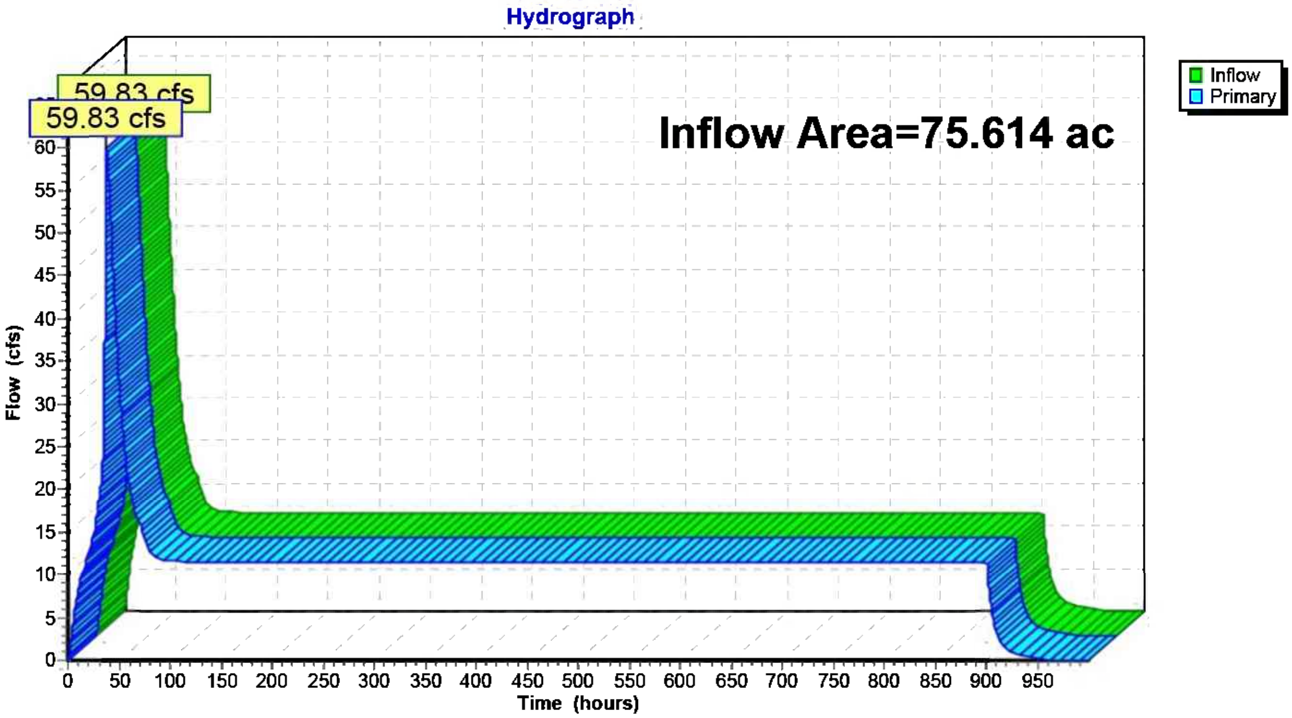
[79] Warning: Submerged Pond 1P Primary device # 1 by 7.22'

Inflow Area = 75.614 ac, 0.00% Impervious, Inflow Depth >147.74" for 100-YR, 72-HR event
Inflow = 59.83 cfs @ 38.08 hrs, Volume= 930.936 af
Primary = 59.83 cfs @ 38.08 hrs, Volume= 930.936 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-999.00 hrs, dt= 0.01 hrs

Fixed water surface Elevation= 24.15'

Link 7L: Discharge Canal



ATTACHMENT 2

Boring Logs

Attachment 2-A

Geosyntec Boring Logs

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

BORING LOG

BOREHOLE ID: *SPT-109*

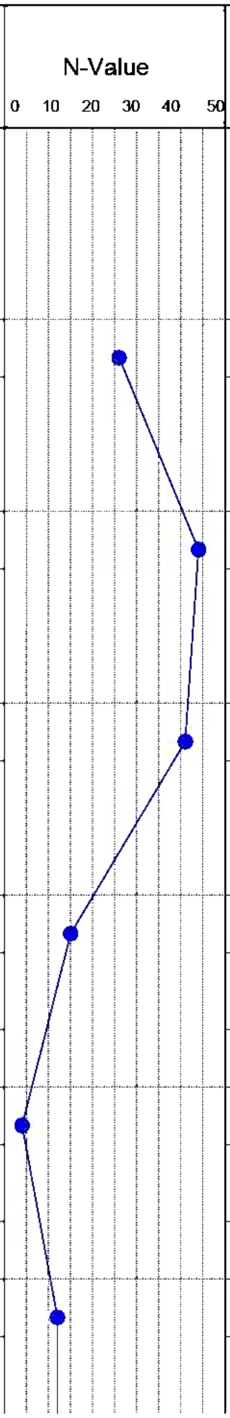
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/24/2013*
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: *546898.7338*
EASTING: *2498876.8972*
GROUND ELEVATION: *37.39 ft msl*

Elev. (ft msl)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	0					
35						
	▼	Medium dense, black to light brown, fine SAND (SP), slightly silty.		7-11-15	12"	Water level measured as 5.10 ft bgs on 10/5/2013.
30						
	-10	Dense, gray to dark gray, fine SAND (SP), slightly silty.		11-20-24	8"	MC = 22.4%; Fines = 14.5%.
25						
	-15	Dense, gray to black, medium to fine SAND (SP), slightly silty.		9-19-22	14"	MC = 19.6%; Gravel = 0.0%; Sand = 78.7%; Fines = 21.3%.
20						
	-20	Medium dense, gray to dark brown, sandy clay to clayey SAND (SC).		4-6-9	13"	MC = 21.2%; Fines = 38.1%.
				ST-1	24"	Shelby Tube advanced from 21.50 to 23.50 ft bgs. MC = 21.2%; Gravel = 0.1%; Sand = 62.5%; Silt = 12.9%; Clay = 24.5%. LL = 48%; PL = 19%; PI = 29%.
15						
	-25	Loose, black to light brown, sandy clay to clayey SAND (SC).		1-2-2	18"	
				ST-2	24"	Shelby Tube advanced from 26.50 to 28.50 ft bgs.
10						
	-30	Medium dense, gray clayey fine SAND (SC).		2-4-8	NR	No recovery on blow counts. Split spoon sampler pushed to collect sample.
5						Borehole collapsed prior to abandonment at 30.00 ft bgs.



All depths referenced to ground surface.

Total Depth: 51.00 ft bgs

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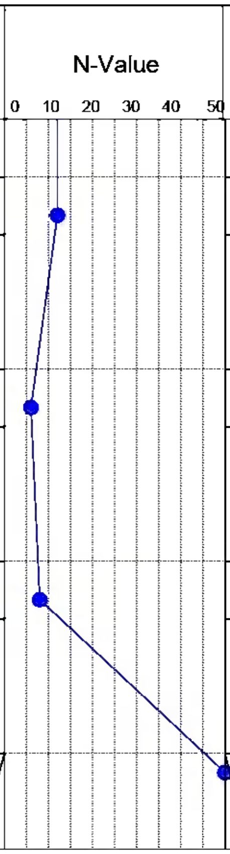
104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

Borehole ID: *SPT-109*
Project No: *GSC5242*

Elev. (ft msl)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
						Tremie grouted from 30.00 ft bgs.
	-35	Medium dense, gray, clean, fine SAND (SP).		7-4-8	10"	MC =28.7%; Fines = 8.3%.
0						
	-40	Firm, gray CLAY (CH), high plasticity.		3-3-3	14"	MC = 71.7%; Gravel = 0.0%; Sand = 23.7%; Silt = 22.9%; Clay = 53.4%, LL = 110%; PL = 33%; PI = 77%.
-5						
	-45	Loose, gray, clayey SAND (SC) with SHELLS.		3-5-3	7"	MC = 37.2%; Fines = 29.1%.
-10						
	-50	Very dense, gray, clayey SAND (SC). Boring terminated at 51.00 ft bgs.		48-50/1"	3"	N-value = 98 blows/ft.
-15						



All depths referenced to ground surface.

Total Depth: 51.00 ft bgs

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

BORING LOG

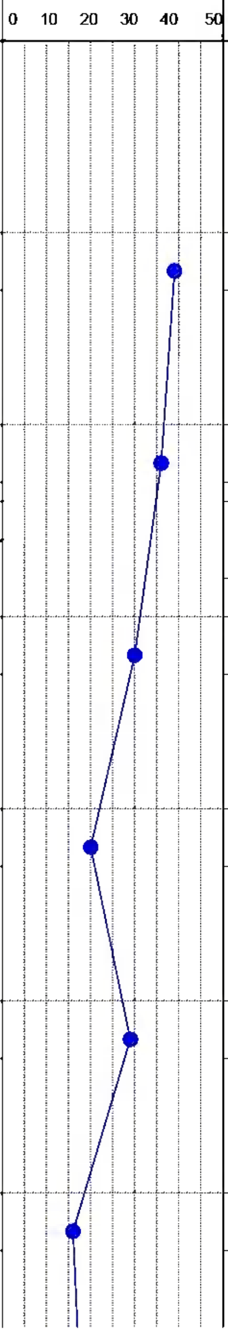
BOREHOLE ID: *SPT-110*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/24/2013*
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: *546029.9130*
EASTING: *2502059.5414*
GROUND ELEVATION: *38.72 ft msl*



Elev. (ft msl)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				
35					
	-5	Dense, dark brown, fine to medium SAND (SP), slightly silty.	10-17-22	15"	
30					
	-10	Dense, dark brown, fine SAND (SP), slightly silty.	11-16-20	15"	MC = 16.3%; Fines = 15.9%. Water level measured as 11.00 ft bgs on 9/25/2013.
			ST-1	10"	Shelby Tube advanced 12" from 12.00 to 13.00 ft bgs. Rig lifted after pushing tube 12".
25					
	-15	Medium dense, dark brown to dark gray, fine SAND (SP), slightly silty.	7-13-17	16"	MC = 18.0%; Fines = 15.9%.
20					
	-20	Medium dense, gray, fine to medium, clean SAND (SP).	5-8-12	11"	Borehole collapsed prior to abandonment at 20.00 ft bgs. Tremie grouted from 20.00 ft bgs.
15					
	-25	Medium dense, gray, fine SAND (SP), slightly silty.	8-15-14	11"	MC = 24.9%; Fines = 12.8%.
10					
	-30	Top 6": Medium dense, gray, silty SAND (SP). Bot 6": Very stiff, gray, sandy CLAY with SHELLS.	7-3-13	12"	

All depths referenced to ground surface.

Total Depth: 68.00 ft bgs

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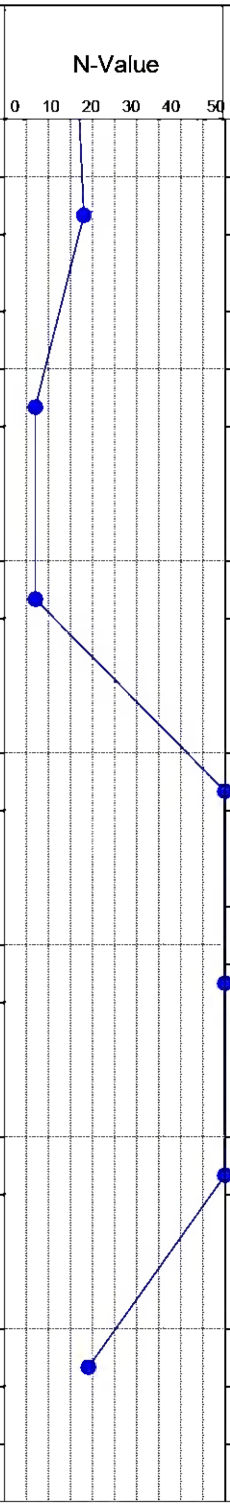
Suite 115
Greenville, SC 29601

BORING LOG

Borehole ID: *SPT-110*
Project No: *GSC5242*



Elev. (ft msl)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
5					
	-35	Medium dense, gray, fine to medium SAND (SM) with SHELLS.	7-7-11	18"	
			ST-2	22"	Shelby Tube advanced 24" from 36.50 to 38.50 ft bgs. Gravel = 2.6%; Sand = 81.9%; Silt = 4.8%; Clay = 10.7%. LL = NP; PL = NP; PI = NP.
0					
	-40	Loose, gray, fine SAND (SM) with many SHELLS, slightly silty.	3-4-3	10"	MC = 25.2%; Gravel = 17.8%; Sand = 65.7%; Fines = 16.5%.
-5					
	-45	Loose, gray, silty fine SAND (SM) with many SHELLS.	2-3-4	8"	MC = 21.7%
-10					
	-50	Very dense, gray, clayey fine SAND (SC) with SHELLS.	4-50/5"	8"	
					Hard drilling between 52.00 and 53.00 ft bgs. Wood fragments float to surface.
-15				4"	
	-55	Very dense, gray, clayey fine SAND (SC) and SHELLS.	50/5"		MC = 13.1%; Gravel = 35.9%; Sand = 45.3%; Fines = 18.8%. Hard drilling between 56.80 and 57.20 ft bgs.
-20					
	-60	Very dense, clayey GRAVEL (GC) and SHELLS.	10-50/5"	8"	
-25					
	-65	Stiff, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Black Mingo Formation).	5-7-12	18"	Shelby Tube advanced 18" from 66.50 to 68.00 ft bgs. Rig lifted at 68.00 ft bgs. Bottom of Shelby Tube was crushed.
			ST-3	15"	MC = 50.0%; Gravel = 0.0%; Sand = 13.0%; Silt = 38.4%; Clay = 48.6%.
-30		Boring terminated at 68.00 ft bgs.			

All depths referenced to ground surface.

Total Depth: 68.00 ft bgs

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104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-111*

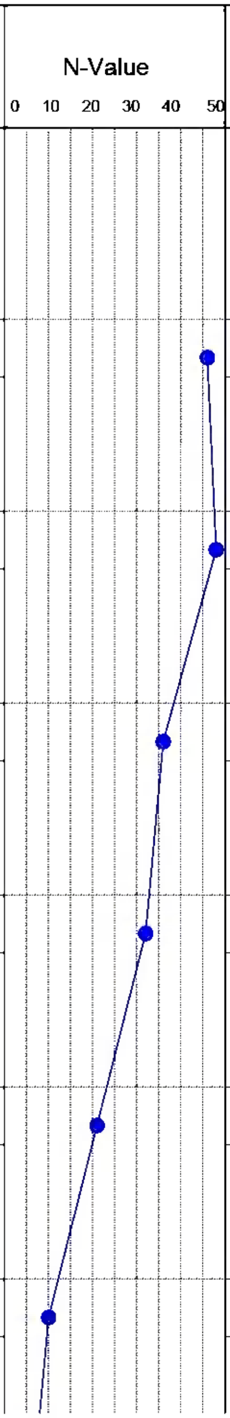
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/24/2013*
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: *546949.6779*
EASTING: *2502267.1691*
GROUND ELEVATION: *39.41 ft msl*

Elev. (ft msl)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				
35	-5	Dense, dark brown, fine to medium SAND (SP), slightly silty.	7-17-29	12"	MC = 13.7%; Gravel = 2.9%; Sand = 79.8%; Silt = 8.3%; Clay = 9.0%.
30	-10	Dense, dark brown, fine to medium SAND (SP).	20-21-27	12"	Water level measured as 10.10 ft bgs on 9/25/2013.
25	-15	Dense, brown to dark gray, silty, fine to medium SAND (SM), wet.	12-18-18	14"	
20	-20	Dense, dark gray, silty, fine to medium SAND (SM).	12-15-17	13"	
15	-25	Medium dense, gray, fine to medium SAND (SP), some shells, slightly silty.	9-14-7	10"	MC = 19.1%; Gravel = 0.2%; Sand = 85.6%; Fines = 14.2%.
10	-30	Loose, gray, fine to medium, clean SAND (SP) and SHELLS.	3-4-6	11"	MC = 21.7%; Fines = 9.5%.

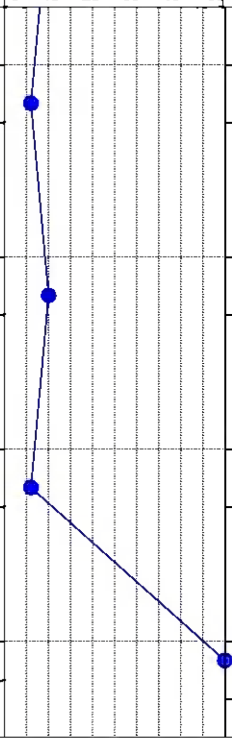


All depths referenced to ground surface.

Total Depth: 51.50 ft bgs

BORING LOG

Borehole ID: *SPT-111*
Project No: *GSC5242*



Elev. (ft msl)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
5	-35	Loose, gray, fine to medium SAND (SP) with SHELLS (1" clay seam 11" from top of sample).		2-2-4		15"	MC = 34.6%; Gravel = 2.3%; Sand = 74.0%; Fines = 23.7%.
0	-40	Loose, gray, fine to medium, clean SAND (SP) with SHELLS (1" clay seam).		5-4-6		5"	MC = 20.8%; Fines = 8.1%. Borehole collapsed prior to abandonment at 40.00 ft bgs. Tremie grouted from 40.00 ft bgs.
-5	-45	Firm, gray, fine, sandy CLAY (CL) with many SHELLS.		1-3-3		18"	
-10	-50	Very hard, gray, clayey SAND (SC) with many SHELLS. Boring terminated at 51.50 ft bgs.		3-5-50/5"		15"	

All depths referenced to ground surface.

Total Depth: 51.50 ft bgs

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

BORING LOG

BOREHOLE ID: *SPT-112*

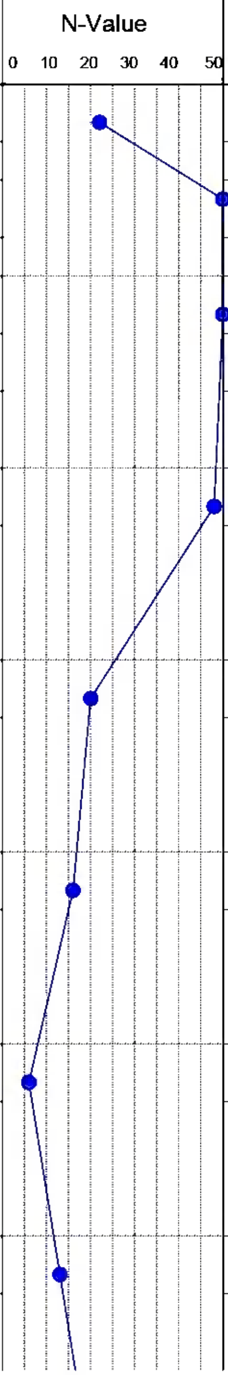
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *9/23/2013*
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: *547207.9843*
EASTING: *2500600.5880*
GROUND ELEVATION: *37.66 ft msl*

Elev. (ft msl)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0	Medium dense, light brown to white, fine, clean SAND (SP), dry.	6-9-13	12"	Water level measured as 5.30 ft bgs on 9/25/2013.
35		Very dense, light brown to white, fine SAND (SP) to black, silty SAND, dry.	9-30-30	14"	N-value = 60 blows/ft; MC = 17.6%; Fines = 10.9%.
	-5	Very dense, light gray to brown, fine to medium SAND (SP), wet, slightly silty.	13-29-30	15"	N-value = 59 blows/ft. Shelby Tube attempted at 8.00 ft bgs. Screw sheared and sampler was lost in borehole.
30			ST-1	20"	Boring offset 2.0 ft west. Shelby Tube advanced 24" from 8.00 to 10.00 ft bgs. MC=21.1%; Gravel=0.0%; Sand=78.7%; Fines=21.3%.
	-10	Dense, light gray to white, fine SAND (SP).	13-21-27	15"	MC = 13.6%; Fines = 13.3%.
25					
	-15	Medium dense, black, silty, fine SAND (SM), some organics (twigs).	5-9-11	12"	
20					
	-20	Medium dense, dark brown, fine SAND (SP), slightly silty.	5-8-8	13"	
15					
	-25	Loose, light gray to brown, fine, clean SAND (SP).	2-3-3	9"	MC = 29.2%; Gravel = 0.1%; Sand = 91.1%; Fines = 8.8%.
10					
	-30	Medium dense, gray to light brown, clean SAND (SP).	4-6-7	10"	MC = 26.8%; Fines = 5.4%.
5					



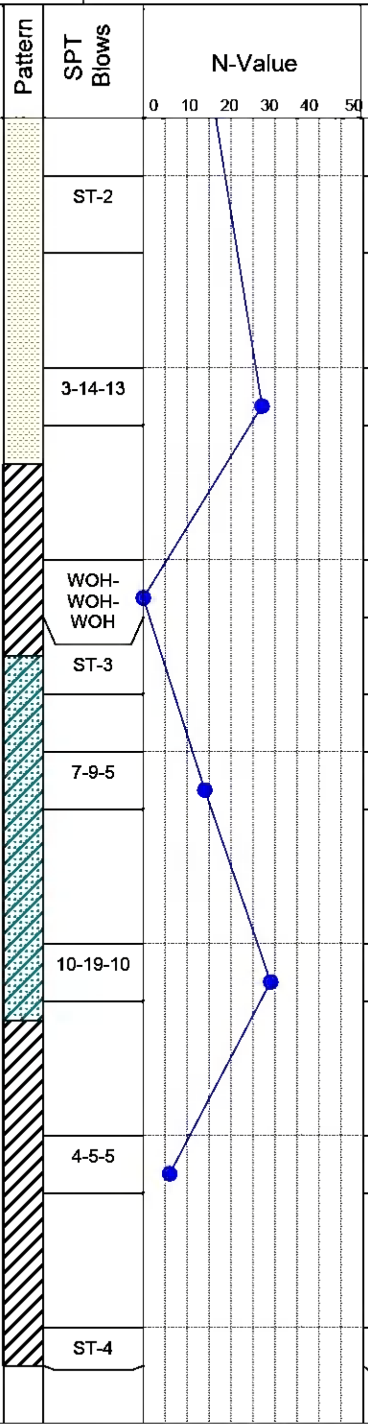
All depths referenced to ground surface.

Total Depth: 66.00 ft bgs

BORING LOG

Borehole ID: *SPT-112*
Project No: *GSC5242*

Elev. (ft msl)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
0	-35	Medium dense, gray to light brown, clean SAND (SP).	ST-2	NR	Shelby Tube advanced 24" from 35.00 to 37.00 ft bgs. No Recovery.
-5	-40	Medium dense, light gray, fine, clean SAND (SP).	3-14-13	9"	
-10	-45	Very soft, gray (blue tint) CLAY (CH), medium to high plasticity.	WOH-WOH-WOH ST-3	18" 24"	Shelby Tube advanced 24" from 46.50 to 48.50 ft bgs. Gravel=0.3%; Sand=2.3%; Silt=21.5%; Clay=75.9%. LL=151%; PL=55%; PI=96%.
-15	-50	Medium dense, gray, clayey SAND (SC), some fine gravel, some shell fragments.	7-9-5	14"	Borehole collapsed at 50.00 ft bgs.Tremie grouted from 50.00 ft bgs. Hard drilling at 53.00 ft bgs.
-20	-55	Medium dense, gray, clayey SAND (SC), some fine gravel, some shell fragments.	10-19-10	13"	MC = 15.3%; Fines = 22.7%.
-25	-60	Stiff, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Black Mingo Formation).	4-5-5	18"	MC = 48.9%; Gravel = 0.0%; Sand = 9.3%; Silt = 42.0%; Clay = 48.7%.
	-65	Stiff, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Black Mingo Formation). Boring terminated at 66.00 ft bgs.	ST-4	12"	Shelby Tube advanced 12" from 65.00 to 66.00 ft bgs. Stiff material encountered. Unable to advance tube past 66.00 ft bgs. k=1.8E-8 cm/s



All depths referenced to ground surface.

Total Depth: 66.00 ft bgs

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

BORING LOG

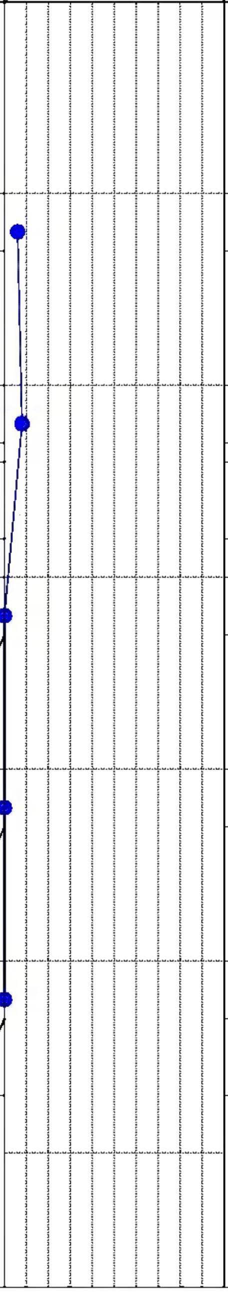
BOREHOLE ID: *SPT-113*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *10/3/2013*
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: *546933.0676*
EASTING: *2499058.1034*
GROUND ELEVATION: *42.27 ft NGVD29*



Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0						
40							
	-5	Soft, black SILT (ML) (Fly Ash), some organics and wood fragments, slightly sandy.		2-1-2		6"	
35							
	▼						Water level measured as 8.40 ft bgs on 10/9/2013.
	-10	Very loose, black, silty, medium to coarse SAND (SW) (Bottom Ash), some wood fragments.		2-2-2		12"	MC = 31.8%; Gravel = 7.7%; Sand = 61.8%; Clay = 30.5%.
30		Black, silty, medium to coarse SAND (SW) (Bottom Ash).		ST-1		8"	Shelby Tube advanced by means of a Piston Sampler 24" from 12.00 to 14.00 ft bgs. Sample is collected in a bag. Wood chips observed in drilling fluid.
	-15	Very loose, black, silty, coarse SAND (SW) (Bottom Ash), some fine gravel.		WOH-WOH-WOH		1"	MC = 43.8%; Gravel = 0.0%; Sand = 81.1%; Clay = 18.9%.
25							
	-20	No Recovery (Fly Ash).		WOR-WOR-WOR		NR	
20							
	-25	No Recovery (Fly Ash).		WOR-WOR-WOR		NR	Borehole did not collapse prior to abandonment. Tremie grouted from 28.50 ft bgs.
15		Boring terminated at 28.50 ft bgs.		ST-2		5"	Shelby Tube advanced by means of a Piston Sampler 24" from 26.50 to 28.50 ft bgs. MC=40.5%; Gravel= 10.4%; Sand=23.6%; Silt=43.8%; Clay=22.2%; pH = 5.7.
	-30						
10							

All depths referenced to ground surface.
Total Depth: 28.50 ft bgs

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

BORING LOG

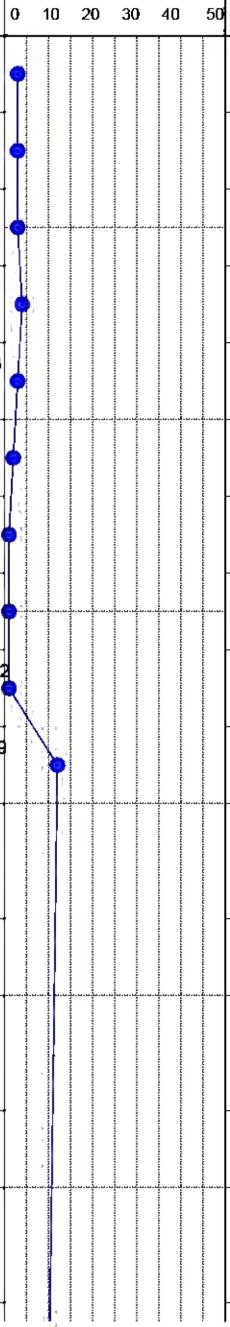
BOREHOLE ID: *SPT-302*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/01/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Mid Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *547422.19*
EASTING: *2498943.21*
GROUND ELEVATION: *NS*



Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0			2-2-1-2		24"	
35				2-1-2-2		22"	
	-5			1-1-2-3		15"	Water level was measured 4.2 ft bgs on 03/02/2016 at 7.15 am.
				2-2-2-1		16"	Water level was measured 5.66 ft bgs on 03/03/2016 at 7.50 am.
30				WOH-1-2-3		14"	
	-10			2-1-1-1		17"	
25				1/24"		22"	
	-15			1/24"		11"	
				WOH/12"-1-2		11"	
20				WOH-2-10-9		12"	
	-20						
15				ST-1		24"	
	-25						
10				ST-1		24"	
	-30						
5							

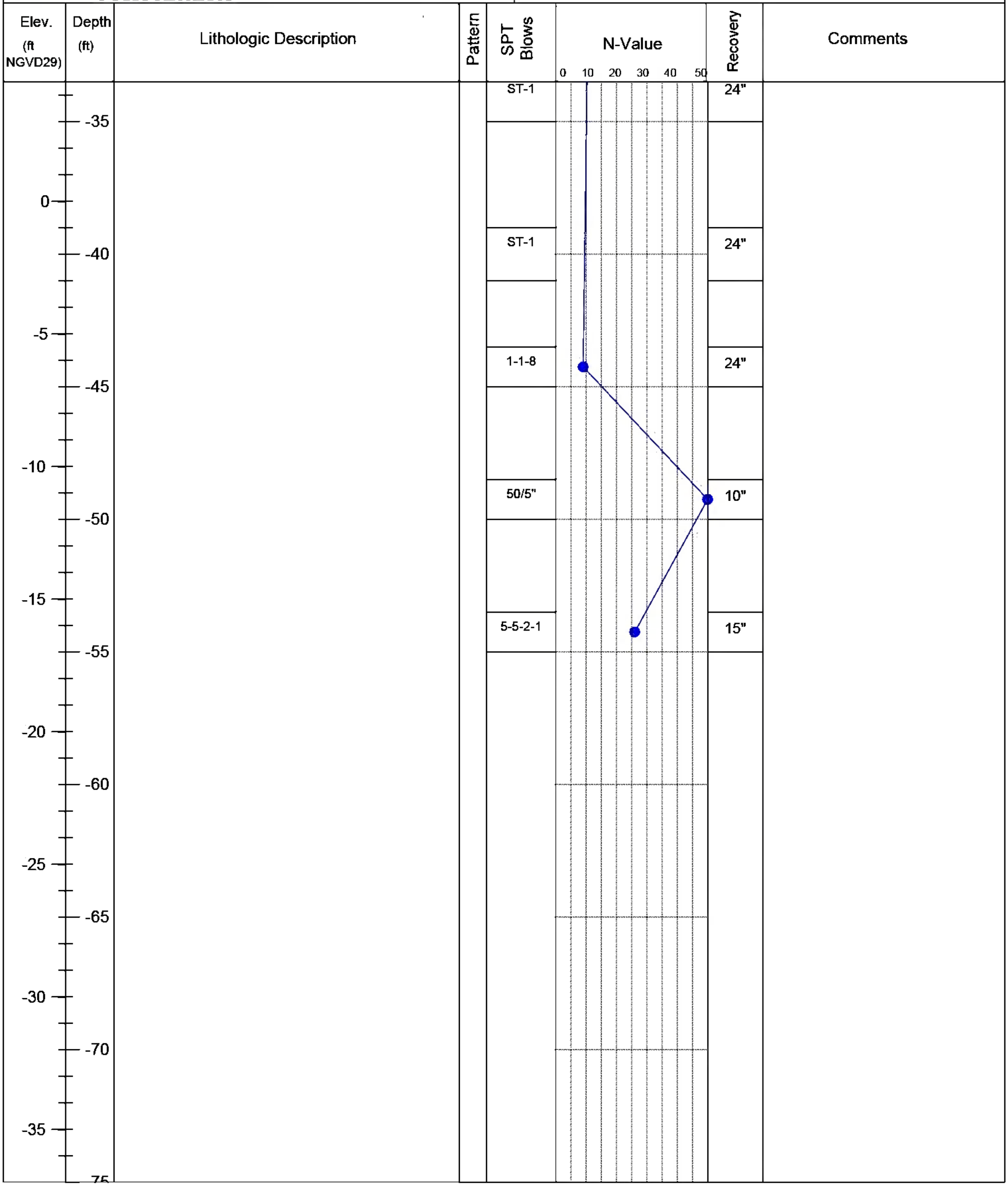
All depths referenced to ground surface.

Total Depth: 55 ft bgs

BORING LOG

Borehole ID: *SPT-302*

Project No: *GSC5242*



SPT Blows	Recovery
ST-1	24"
ST-1	24"
1-1-8	24"
50/5"	10"
5-5-2-1	15"

All depths referenced to ground surface.

Total Depth: 55 ft bgs

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Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *SPT-303*

GENERAL INFORMATION

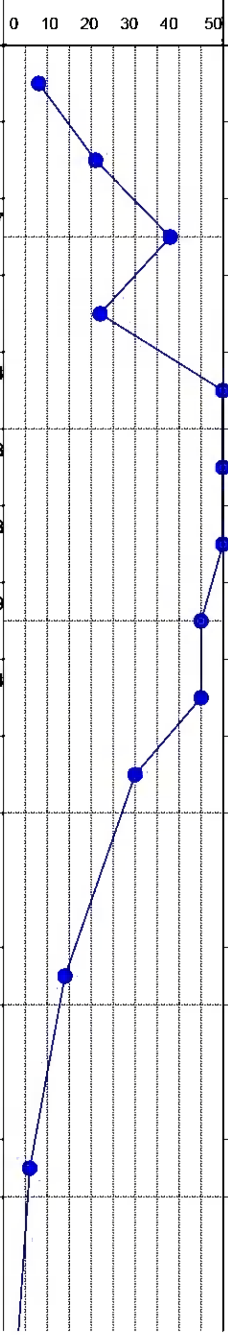
PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/29/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Mid Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *546607.80*
EASTING: *2499254.34*
GROUND ELEVATION: *NS*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
0	0			1-3-5-7	14"	
				5-9-12-19	15"	
-5	-5			10-18-20-17	24"	
				3-8-14-21	16"	
				10-25-33-44	15"	
-10	-10			12-26-36-48	13"	
				12-26-38-48	17"	
-15	-15			15-22-23-19	10"	
				13-20-25-34	14"	
				9-11-19-20	15"	
-20	-20					
				5-5-9	13"	
-25	-25					
				5-3-3	14"	
-30	-30					

N-Value



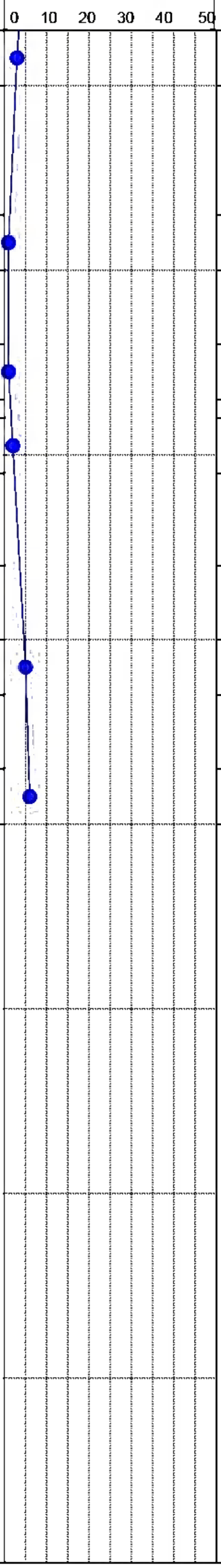
All depths referenced to ground surface.

Total Depth: 55 ft bgs

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

BORING LOG

Borehole ID: *SPT-303*
Project No: *GSC5242*



Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
				1-1-2	14"	
-35	-35					
				WOH/12"-1	24"	
-40	-40					
				ST-1	24"	
				WOH/12"-1	24"	
				1-1-1	18"	
-45	-45					
				ST-2	24"	
-50	-50					
				3-2-3	12"	
				13-3-3	6"	
-55	-55					Boring Terminated at 55 feet. Water level was not measured due to cave in.
-60	-60					
-65	-65					
-70	-70					
-75	-75					

All depths referenced to ground surface.

Total Depth: 55 ft bgs

Geosyntec consultants
1300 South Mint Street
Suite 410
Charlotte, NC 28203

BORING LOG

BOREHOLE ID: *SPT-303A*

GENERAL INFORMATION

PROJECT NAME: L.V. Sutton Sheet Pile Investigation
PROJECT NO: *GC6005*
SITE LOCATION: *Wilmington, North Carolina*
BORING DATE: *03/02/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Mid-Atlantic Drilling*
DRILLER NAME: *William Wiggins*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C Track Rig (SN: 273964)*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT with Split Spoon*
LATITUDE:
LONGITUDE:
ELEVATION:

Elev. (ft)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value 0 10 20 30 40 50	Recovery	Comments
	-30						
5							
				ST-1			
	-35						Water level was measured at 5.43 ft bgs at 7.20 am on 03/03/2016.
0							
				ST-2			
	-40						
-5							
				ST-3			
	-45						Shelby Tube advanced 24" from 43 to 45 ft bgs. Sand = 71.5%; Fines=97.4%; Silt = 30.6%; Clay = 66.8%. LL = 108.0; PL = 38.0; PI = 70.
-10							
				ST-4			
	-50			ST-5			No sample recovery
-15							
	55						

All depths referenced to ground surface.

Total Depth:

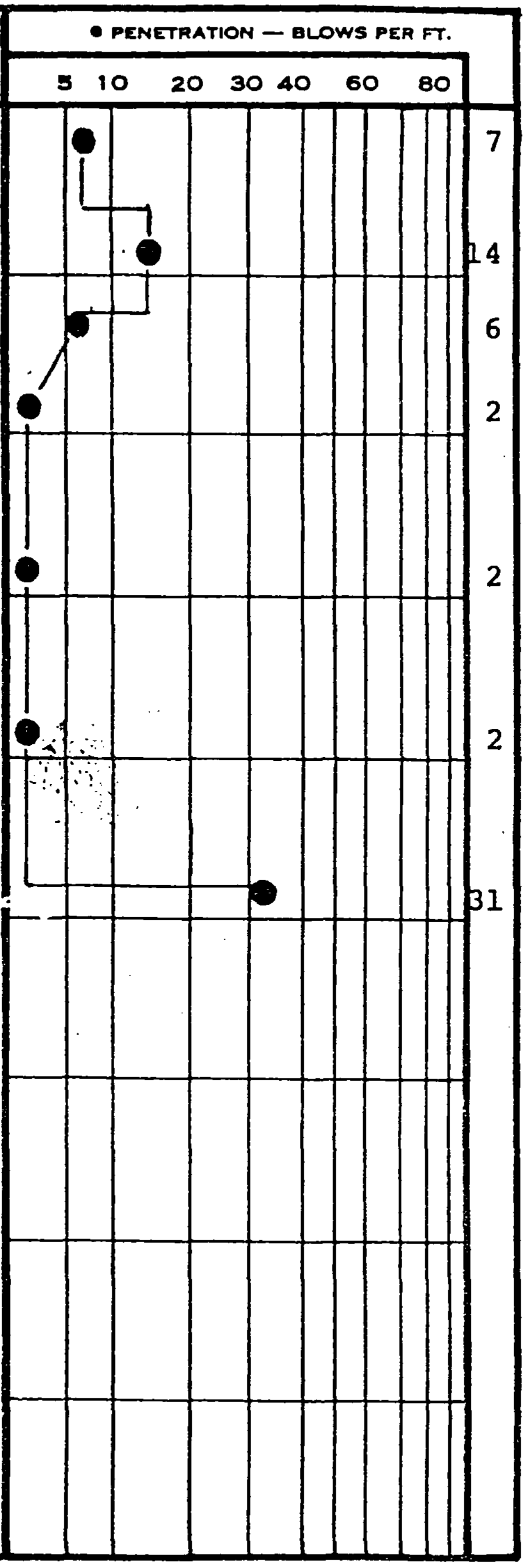
Attachment 2-B

S&ME (1978) Boring Logs

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.
		18.2
	Black Dark Brown Peaty*	
	Firm Brown Clayey Fine Sandy Silty CLAY	
		15.2
5	Firm Gray Brown Silty Fine to Medium SAND	
		11.7
10	Loose Gray Firm to Medium SAND with Silty Clayey Seams and Shells	
15		
20		
		-5.8
25	Firm Gray Silty Fine **	
		-8.3
	Boring Terminated Encountered Shell Limestone	
30	Water Level 2' @ TOB	
	*Silty SAND with Organic Roots **SAND with Shells	
35		
40		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

▬ UNDISTURBED SAMPLE

═ WATER TABLE — 24 HR.

⌶ WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY

◀ LOSS OF DRILLING WATER

BORING NO. SC-63

DATE DRILLED 1-26-78

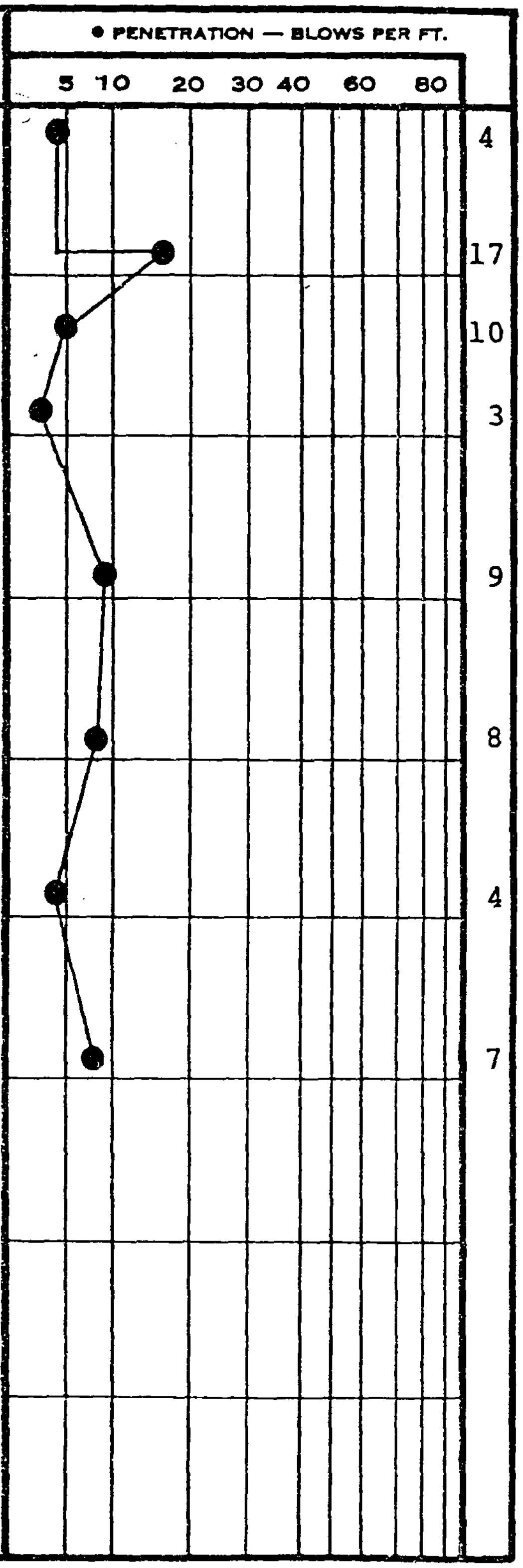
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV.
	Black Silty Fine SAND*		18.9
	Soft Gray Yellow Fine to Medium Sandy Silty CLAY With Organic Material	≡	
5	Firm Gray Fine to Medium SAND With Shells		14.4
10 15 20	Loose Gray Fine to Medium SAND With Some Soft Clay Seams and Shells		9.9 5.9
25 30	Very Loose to Loose Gray Silty Fine SAND With Some Shells		-3.1
35 40	Boring Terminated Encountered Shell Limestone Water Level @ 2'6" @ TOB *Organically Stained		-14.6



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

≡ WATER TABLE — 24 HR.

▬ UNDISTURBED SAMPLE ≡ WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY ◀ LOSS OF DRILLING WATER

BORING NO. SC-64

DATE DRILLED 12-28-77

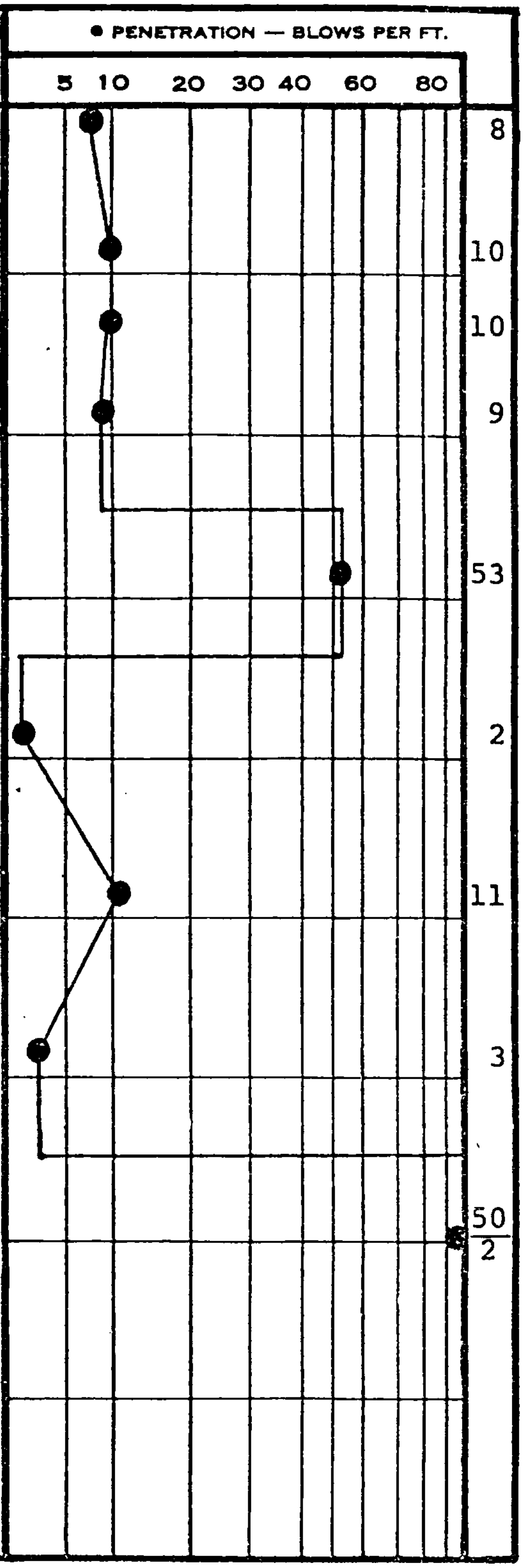
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 22.6
	Loose Gray Black Silty Fine to Medium SAND*	
5 10	Loose Dark Brown Gray Slightly Silty Medium Fine SAND* *Organically Stained	10.1
15	Dense Gray Fine to Medium SAND With Shells	5.6
20 25 30	Very Soft to Stiff Dark Gray Silty CLAY With Fine to Medium Sand Seams	
35	Very Dense Light Gray Silty Fine to Medium SAND With Shells and Shell Limestone Fragments	-13.4
40	Boring Terminated Encountered Shell Limestone Water Level @ 3'6" @ TOB	



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

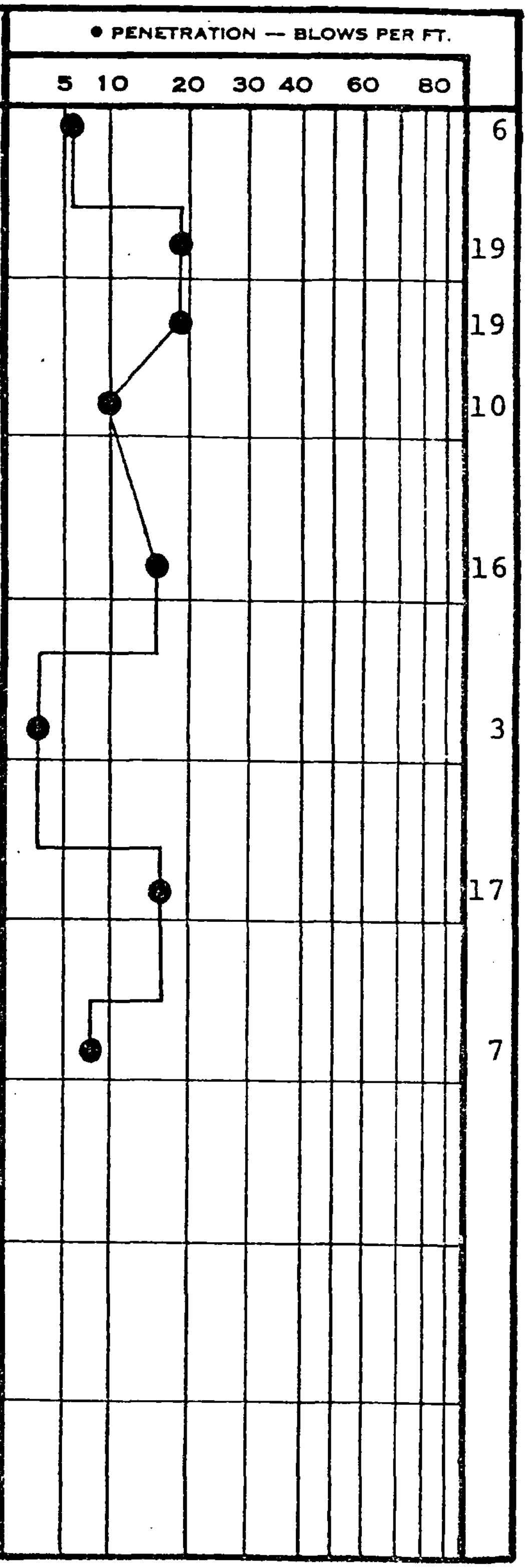
WATER TABLE — 24 HR.
▬ UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER

BORING NO. SC-66
DATE DRILLED 12-20-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 24.4	● PENETRATION — BLOWS PER FT.
	Tan Silty Fine SAND		6
	Loose Brown Clayey Silty Fine SAND	21.4	
5 10	Loose to Firm Brown Tan Medium Fine SAND	11.4	19 19 10
15	Firm Gray Medium Fine SAND With Shells	7.4	16
20	Soft Gray Very Fine Slightly Sandy Silty CLAY With Shells and Fine Sand Seams	1.4	3
25	Very Stiff Fine Sandy Silty CLAY With Some Very Coarse Sand	-3.6	17
30	SHELLS With Gray Silty Fine to Medium SAND	-9.1	7
35 40	Boring Terminated Encountered Shell Limestone Water Level 2.5' @ TOB		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

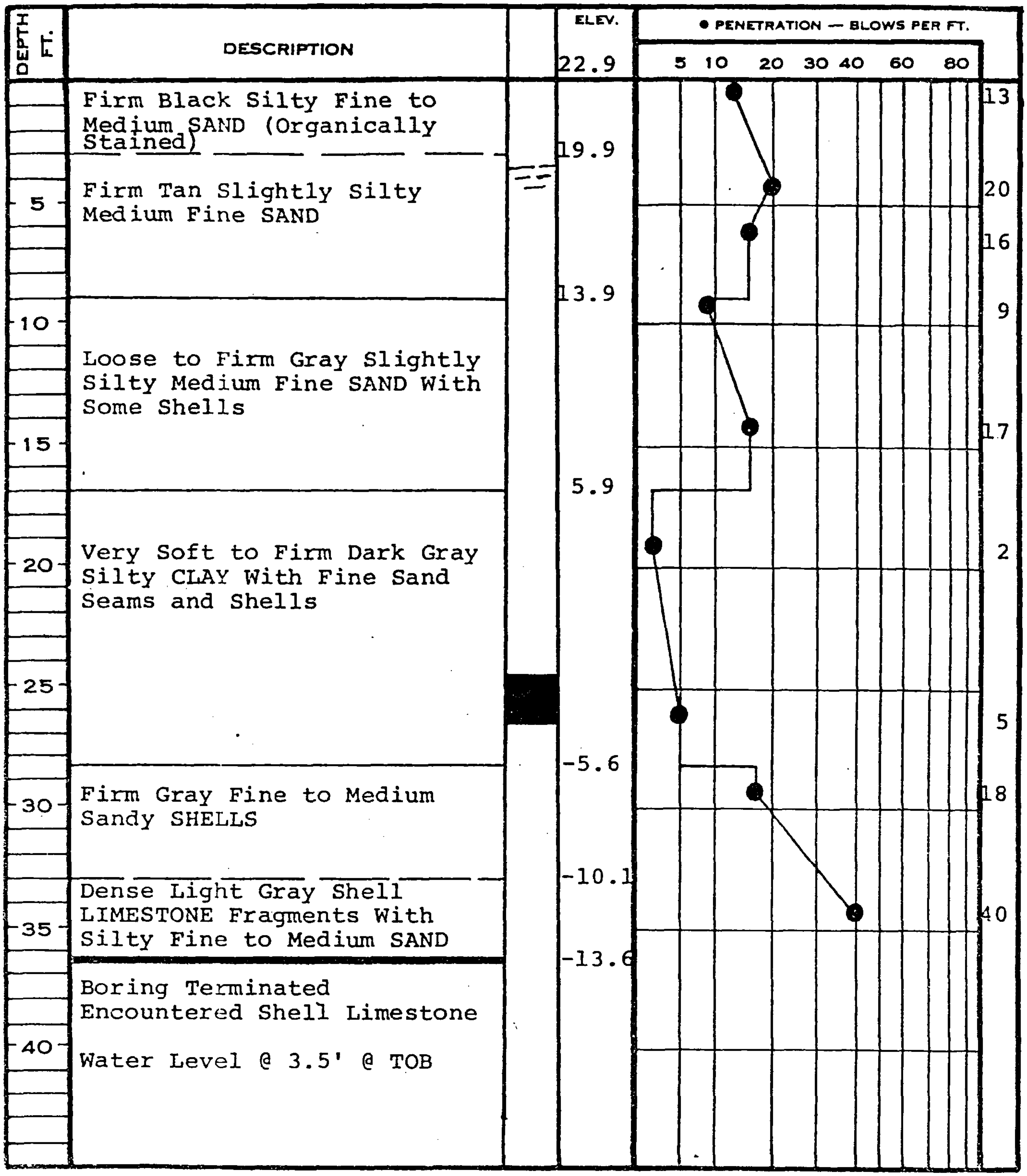
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-67
DATE DRILLED 1-12-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA



DESCRIPTION	ELEV.
	22.9
Firm Black Silty Fine to Medium SAND (Organically Stained)	19.9
Firm Tan Slightly Silty Medium Fine SAND	13.9
Loose to Firm Gray Slightly Silty Medium Fine SAND With Some Shells	5.9
Very Soft to Firm Dark Gray Silty CLAY With Fine Sand Seams and Shells	-5.6
Firm Gray Fine to Medium Sandy SHELLS	-10.1
Dense Light Gray Shell LIMESTONE Fragments With Silty Fine to Medium SAND	-13.6
Boring Terminated Encountered Shell Limestone Water Level @ 3.5' @ TOB	

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

═ WATER TABLE — 24 HR.

▬ UNDISTURBED SAMPLE

-⊥- WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY

◀ LOSS OF DRILLING WATER

BORING NO. SC-68

DATE DRILLED 12-22-77

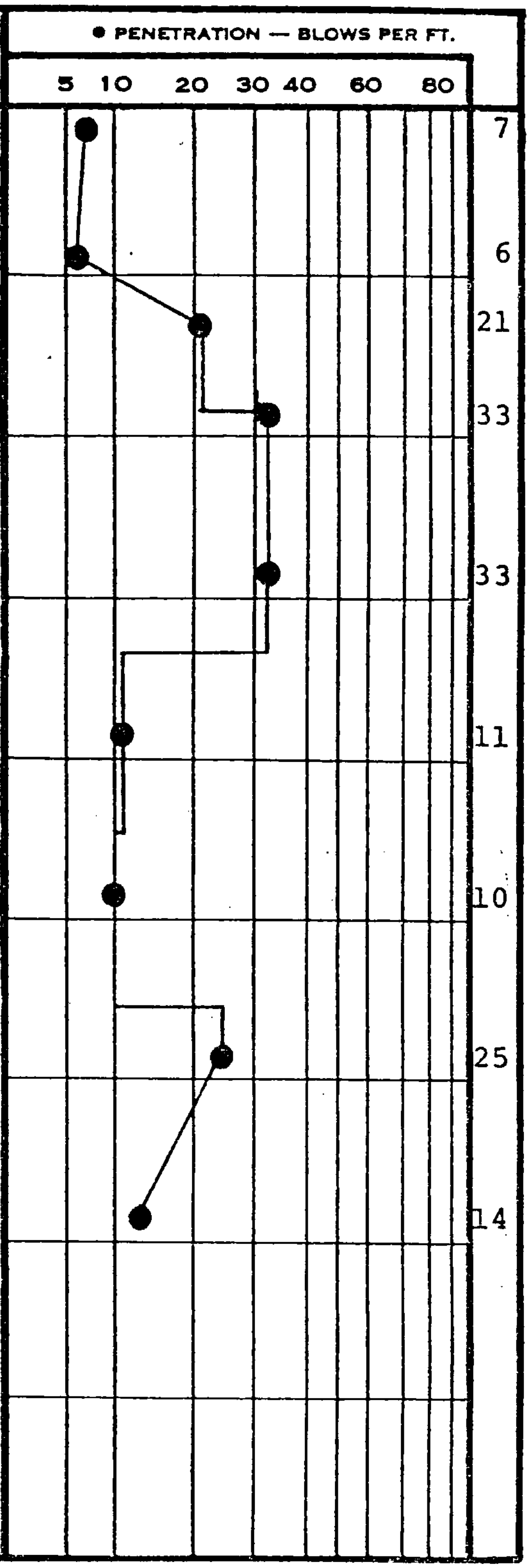
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DESCRIPTION	ELEV.
	26.0
Tan Gray Clayey Silty Fine SAND	
Loose to Firm Brown Gray Slightly Silty Fine to Medium SAND With Wood and Shells	17
Dense Gray Fine SAND	13
Loose Gray Slightly Silty Fine SAND With Some Shells	9.0
SHELLS With Gray Fine to Medium Sand	3.5
Stiff Gray Fine Slightly Sandy Silty CLAY With Some Shells	-2.0
Firm Gray Fine SAND With Many Shells	-10.5
Boring Terminated Encountered Shell Limestone	



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

▬ UNDISTURBED SAMPLE

WATER TABLE — 24 HR.

WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY

◀ LOSS OF DRILLING WATER

BORING NO. SC-69

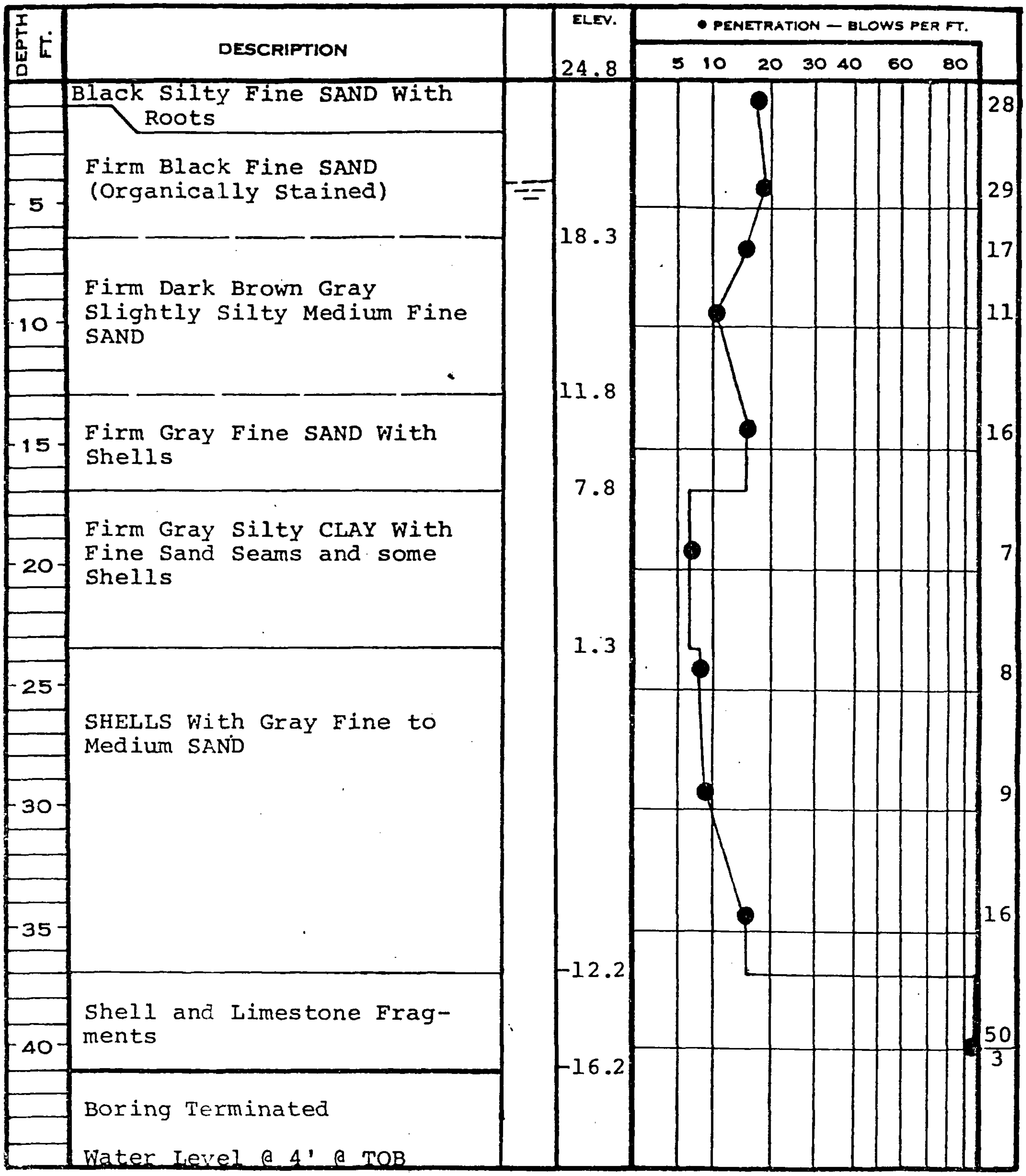
DATE DRILLED 1-12-78

JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA



DESCRIPTION		ELEV.	● PENETRATION — BLOWS PER FT.
		24.8	
Black Silty Fine SAND With Roots			28
Firm Black Fine SAND (Organically Stained)	═		29
		18.3	
Firm Dark Brown Gray Slightly Silty Medium Fine SAND			17 11
		11.8	
Firm Gray Fine SAND With Shells			16
		7.8	
Firm Gray Silty CLAY With Fine Sand Seams and some Shells			7
		1.3	
SHELLS With Gray Fine to Medium SAND			8 9 16
		-12.2	
Shell and Limestone Fragments			50/3
		-16.2	
Boring Terminated Water Level @ 4' @ TOB			

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

═ WATER TABLE — 24 HR.

▨ UNDISTURBED SAMPLE

═ WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY

◀ LOSS OF DRILLING WATER

BORING NO. SC-70

DATE DRILLED 12-22-77

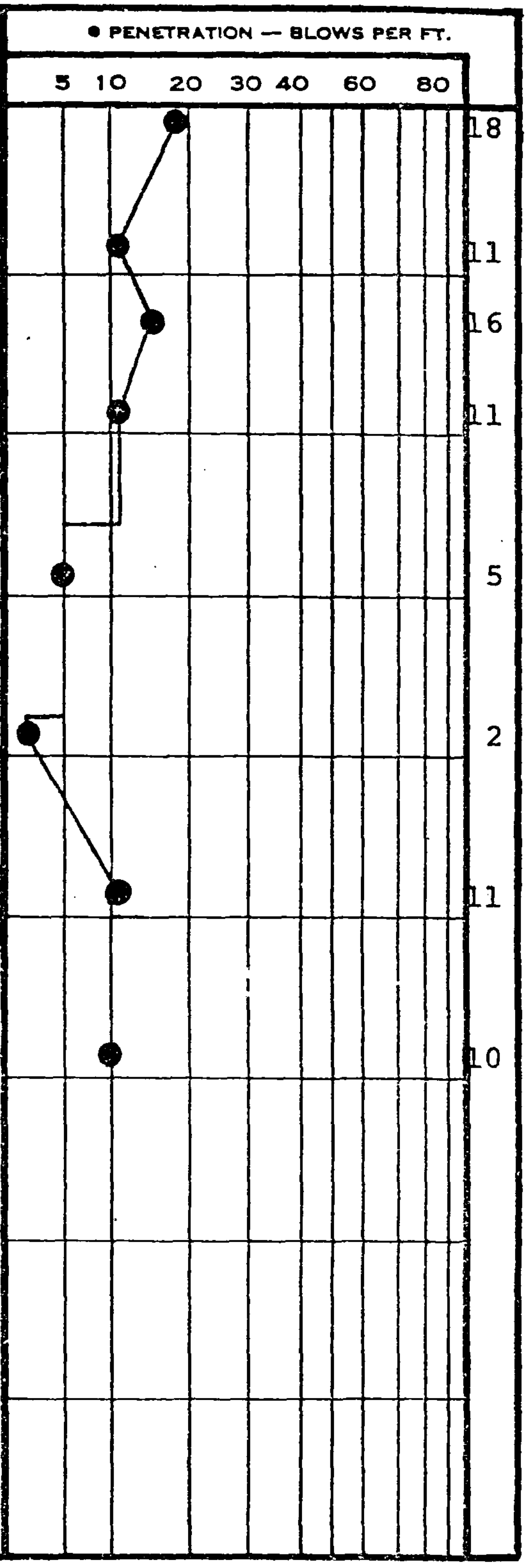
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 25.7
	Firm Tan Brown Medium Fine SAND		22.7
5	Firm Dark Brown Slightly Silty Medium Fine SAND		
	Firm Gray Coarse to Fine SAND		16.7
10	Firm Gray Green Fine SAND		12.7
15	Loose Gray Slightly Silty Fine SAND With Some Shells		6.7
20	Very Soft Gray Fine Sandy Silty CLAY With Fine Sand Seams		
25 30	Firm Gray Fine SAND With Many Shells		-6.8
35	Boring Terminated Encountered Shell Limestone Water Level @ 3' @ TOB		
40			



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

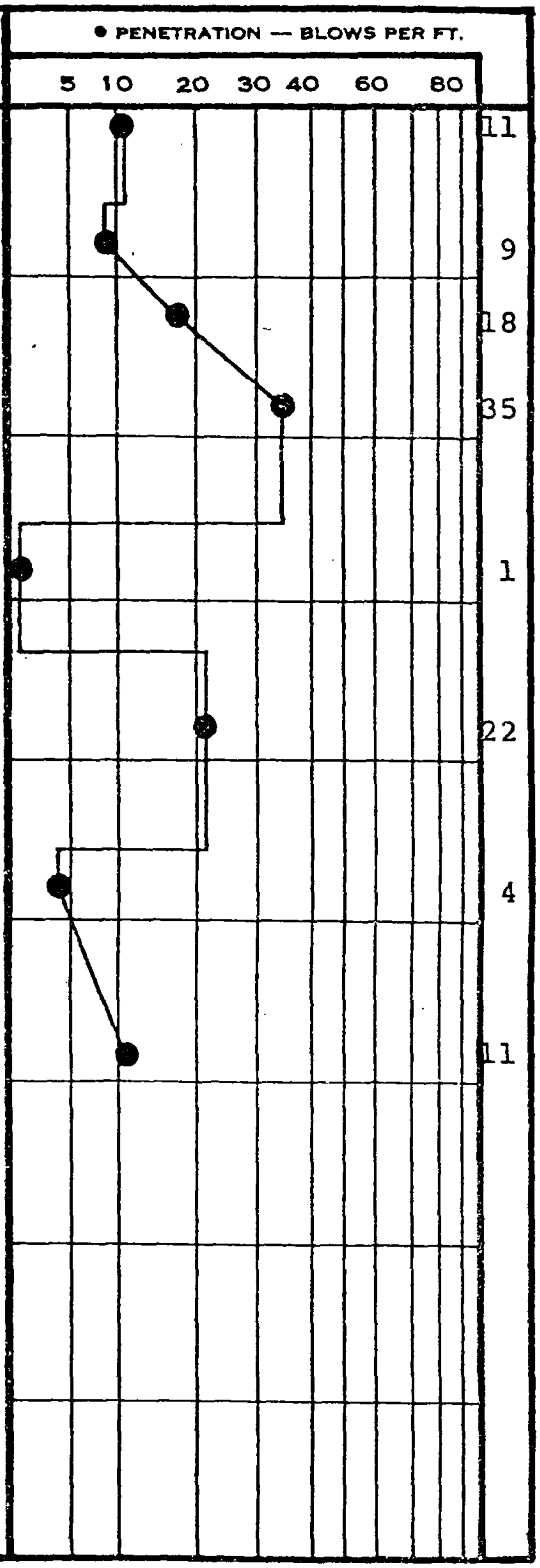
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

═ WATER TABLE — 24 HR.
■ UNDISTURBED SAMPLE
≟ WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER

BORING NO. SC-71
DATE DRILLED 1-13-78
JOB NO. SS7735
PAGE 1 OF 1

DESCRIPTION	ELEV.
Firm Brown Slightly Silty Medium Fine SAND	24.9
Loose Dark Brown Fine to Medium SAND (Organically Stained)	21.9
Firm Dark Gray Medium Fine SAND	
Dense Gray Medium Fine SAND With Some Shells	15.9
Very Soft Gray Silty CLAY With Fine to Medium Sand Seams and Some Shells	11.9
SHELLS With Silty Clay intermixed and Fine Medium Sand Seams	7.9
Very Loose to Firm Gray Fine to Medium SAND With Many Shells and Silty Clay Layers	1.9
Boring Terminated Encountered Shell Limestone Water Level @ 1.0' @ TOB	-8.1

DEPTH FT.: 5, 10, 15, 20, 25, 30, 35, 40



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

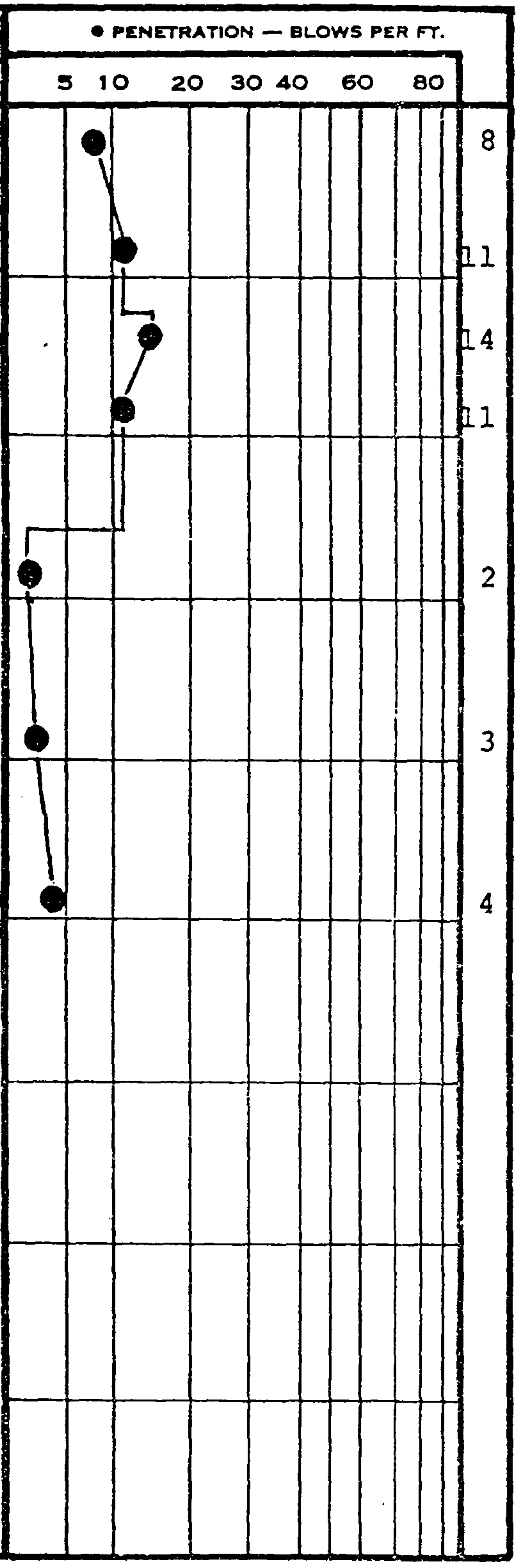
▨ UNDISTURBED SAMPLE
[50] % ROCK CORE RECOVERY
WATER TABLE — 24 HR.
WATER TABLE — 1 HR.
◀ LOSS OF DRILLING WATER

BORING NO. SC-72
DATE DRILLED 1-14-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DESCRIPTION	ELEV. 24.5
Loose to Firm Dark Brown Slightly Silty Fine to Medium SAND (Organically Stained)	18.0
Wood and Gray Fine SAND	15.5
Firm Gray Fine SAND	11.5
Very Loose Gray Clayey Silty Fine SAND with Shells	-5.5
Boring Terminated Encountered Shell Limestone Water Level 3.5' @ TOB	



DEPTH FT. 5 10 15 20 25 30 35 40

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

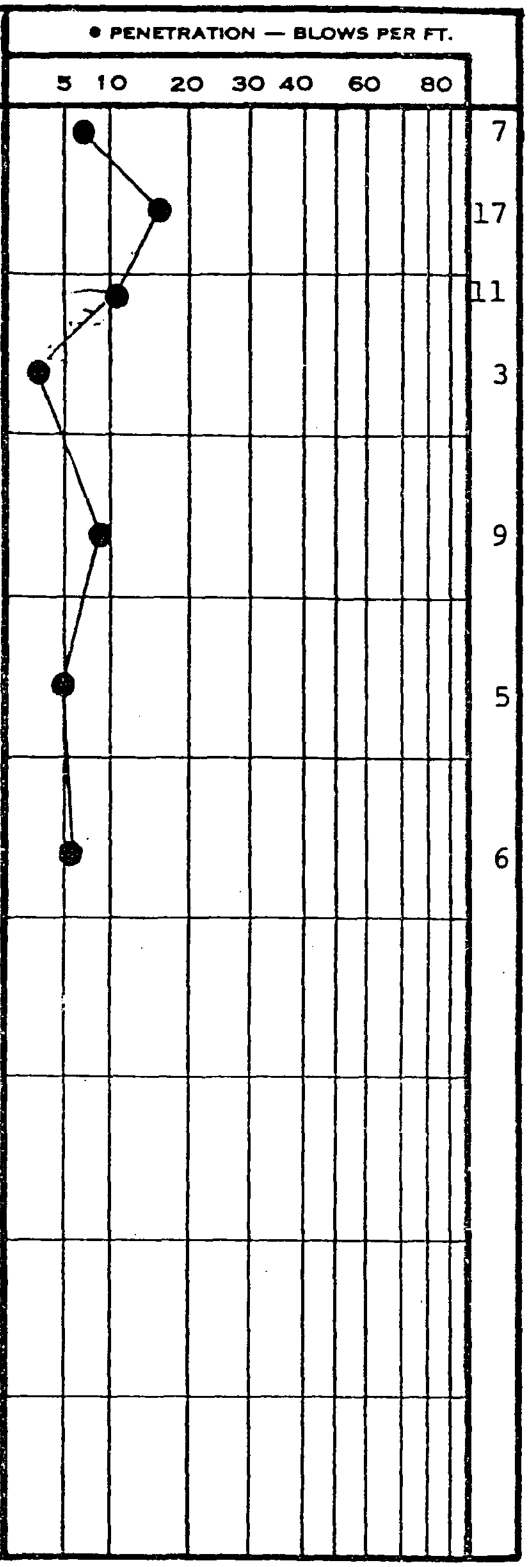
▬ UNDISTURBED SAMPLE
WATER TABLE — 24 HR.
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER

BORING NO. SC-73
DATE DRILLED 1-26-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 24.5
	Black Gray Silty Fine SAND*	
	Firm Gray Yellow Brown Fine Sandy Silty CLAY	21.5
5	Firm Gray Slightly Silty Medium Fine SAND	16.5
10	Very Loose Gray Slightly Clayey Silty Fine SAND	12.5
15	Loose Gray Medium Fine SAND	7.5
20 25	Loose Gray Silty Clayey Fine to Medium SAND With Shells	-3.5
30 35 40	Boring Terminated Encountered Shell Limestone Water Level 1.5' @ TOB *With Roots (Organically Stained)	



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

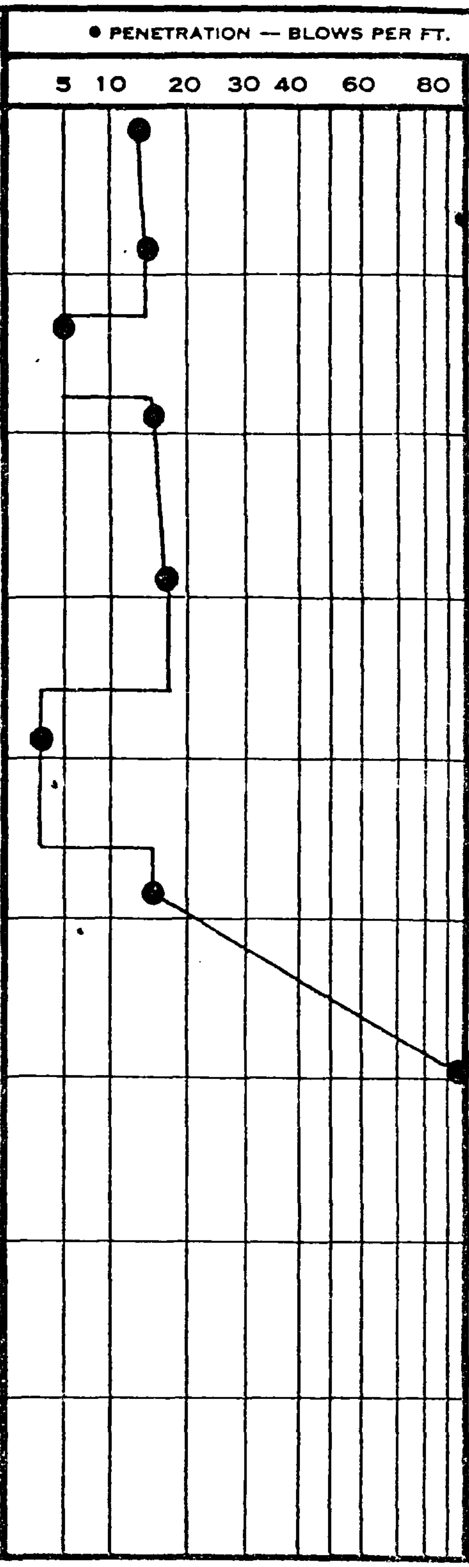
WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER

BORING NO. SC-74
DATE DRILLED 1-12-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.	PENETRATION — BLOWS PER FT.
		19.8	
	Loose Black Silty Fine to*		14
	Firm Tan Brown Silty Fine to Medium SAND	16.8	
5	Firm Tan Fine SAND		15
			5
10	Very Loose Light Gray Brown Slightly Silty Fine to Medium SAND	10.8	16
15	Firm Gray Medium Fine SAND		18
		1.8	
20	Very Loose Black Gray Silty Fine to Medium SAND With Shells and Some Clayey Seams		3
		-4.2	
25	Shells and Shell Limestone Fragments With Gray Clayey Silty Fine to Medium SAND		16
30			50/5
		-12.2	
35	Boring Terminated Encountered Shell Limestone Water Level 2.5' @ TOB		
40	*Medium SAND (Organically Stained)		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

WATER TABLE — 24 HR.

▬ UNDISTURBED SAMPLE

WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY

◀ LOSS OF DRILLING WATER

BORING NO. SC-75

DATE DRILLED 1-14-78

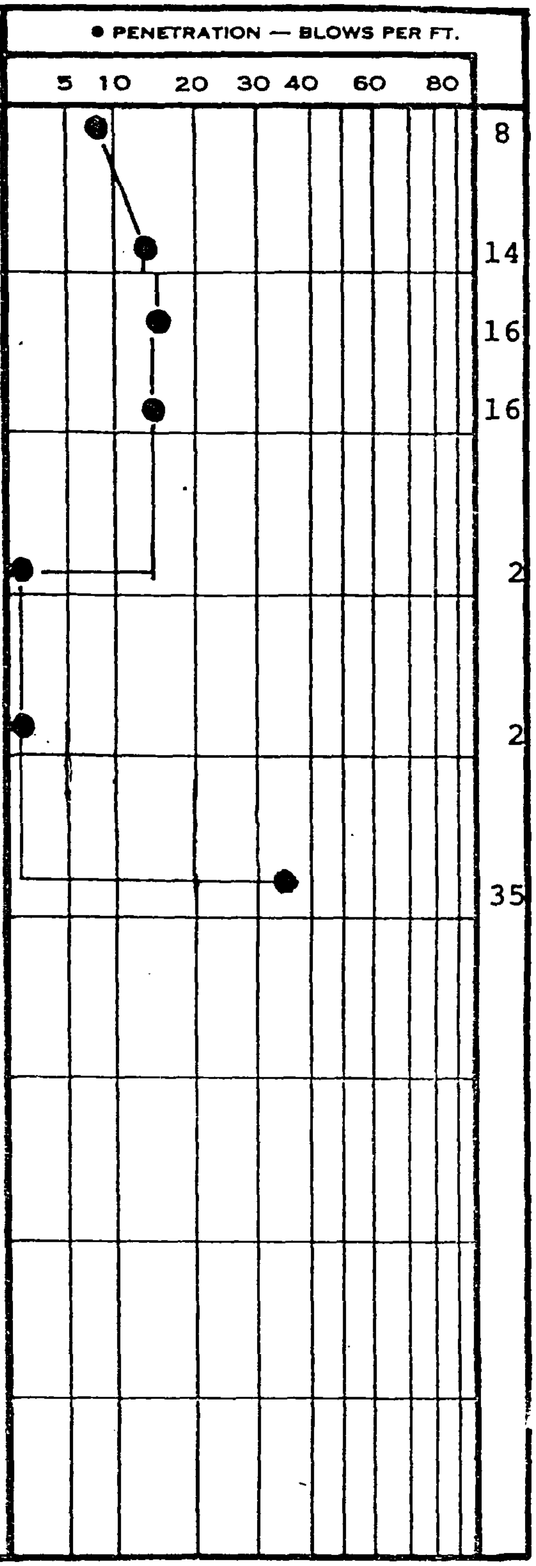
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 16.8
	Light Tan Silty Fine SAND		
	Yellow Fine Sandy Clayey SILT		
5	Firm Red Gray Brown Silty *		11.8
	Firm Gray Tan Slightly Silty Fine SAND		
10			
15	Very Soft Gray Silty CLAY with Fine Sandy Seams		2.3
20			
25	Dense Gray Silty Fine SAND with Shells and Same Fine **		-7.7
			-9.7
30	Boring Terminated Encountered Shell Limestone Water Level 2.5'@ TOB		
35			
40	*Fine SAND ** Gravel		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

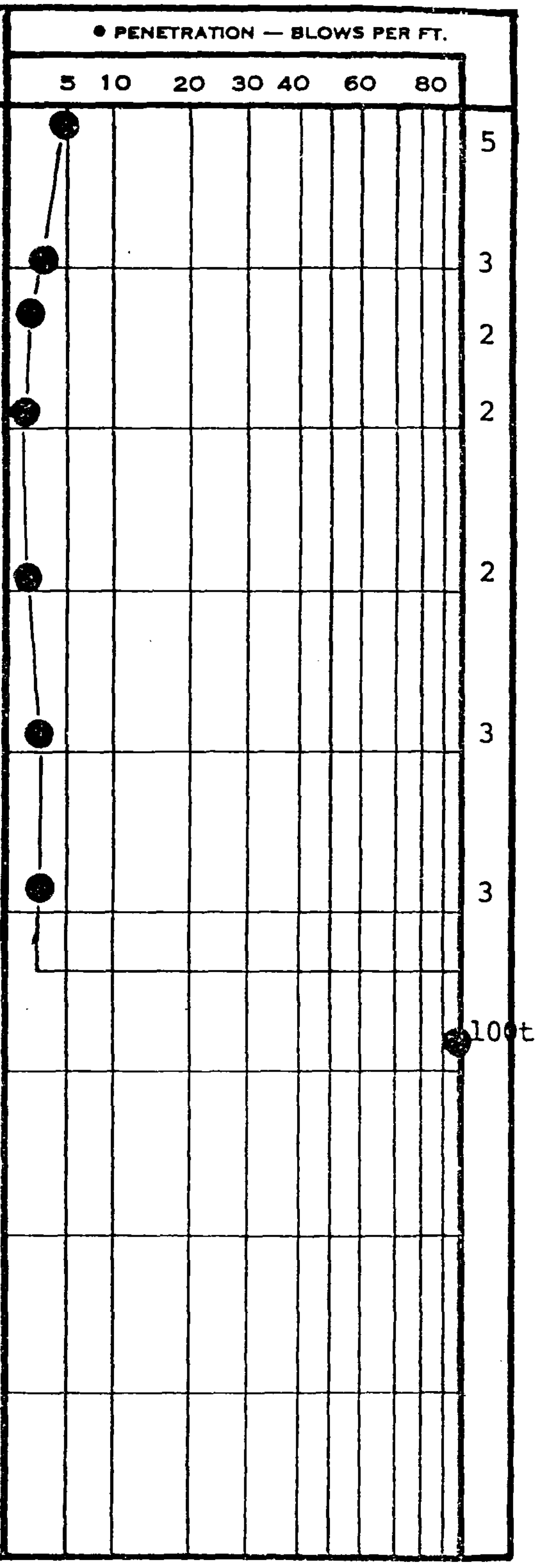
▬ UNDISTURBED SAMPLE
[50] % ROCK CORE RECOVERY
═ WATER TABLE — 24 HR.
═ WATER TABLE — 1 HR.
◀ LOSS OF DRILLING WATER

BORING NO. SC-76
DATE DRILLED 1-26-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DESCRIPTION	ELEV.
	14.0
Loose Gray Silty Fine to Medium SAND with Occasional Wood and Roots	
	7.5
Very Soft Gray Silty Fine Sandy CLAY with Shells and Sandy Seams	
	-13.0
Very Hard Cemented SAND and SHELLS	
	-17.0
Boring Terminated @ 31' Water Level Surface @ TOB	



DEPTH FT.: 5, 10, 15, 20, 25, 30, 35, 40

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

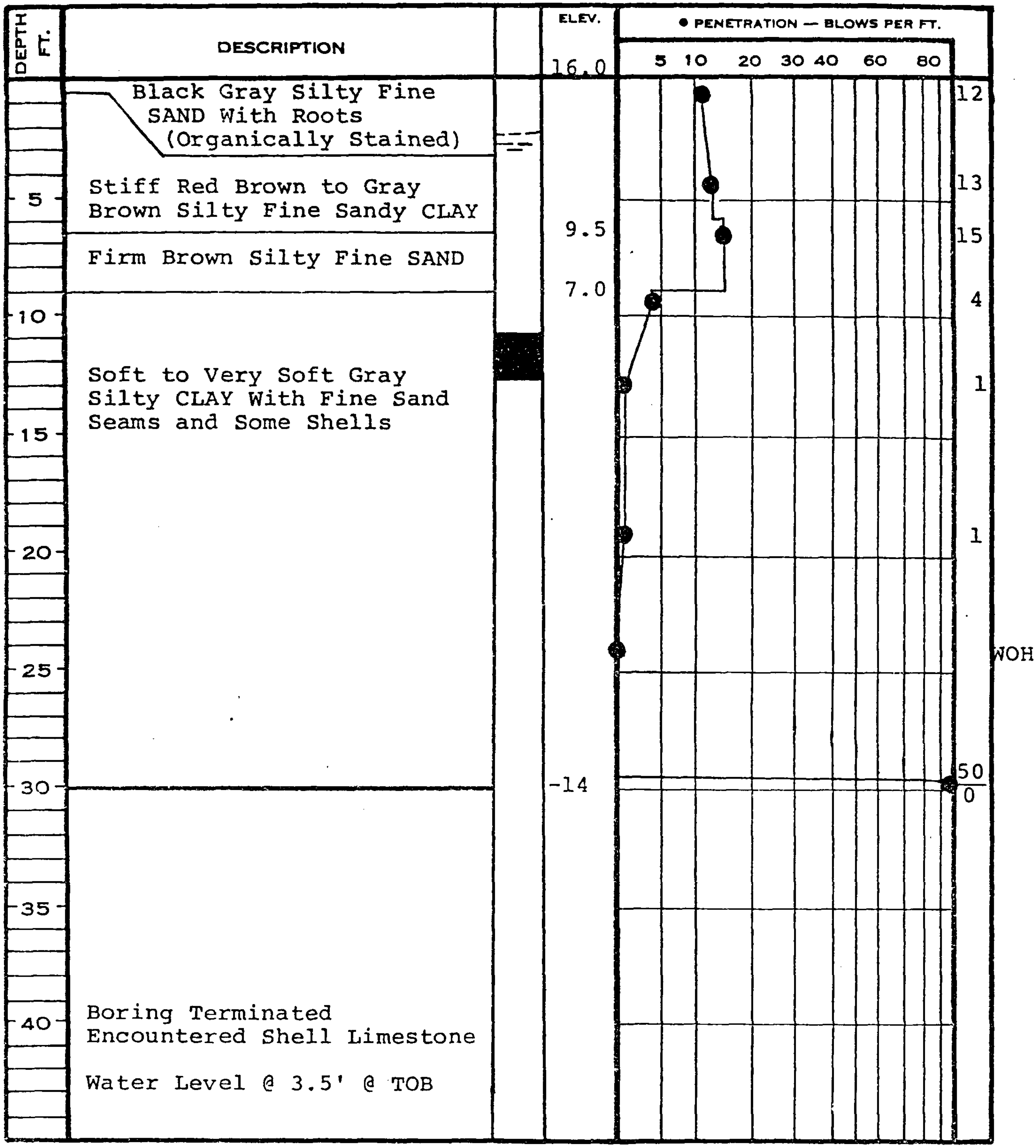
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-77
DATE DRILLED 1-31-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA



Black Gray Silty Fine SAND With Roots (Organically Stained)

Stiff Red Brown to Gray Brown Silty Fine Sandy CLAY

Firm Brown Silty Fine SAND

Soft to Very Soft Gray Silty CLAY With Fine Sand Seams and Some Shells

Boring Terminated Encountered Shell Limestone

Water Level @ 3.5' @ TOB

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

▬ UNDISTURBED SAMPLE

[50] % ROCK CORE RECOVERY

═ WATER TABLE — 24 HR.

≡ WATER TABLE — 1 HR.

◀ LOSS OF DRILLING WATER

BORING NO. SC-78

DATE DRILLED 12-28-77

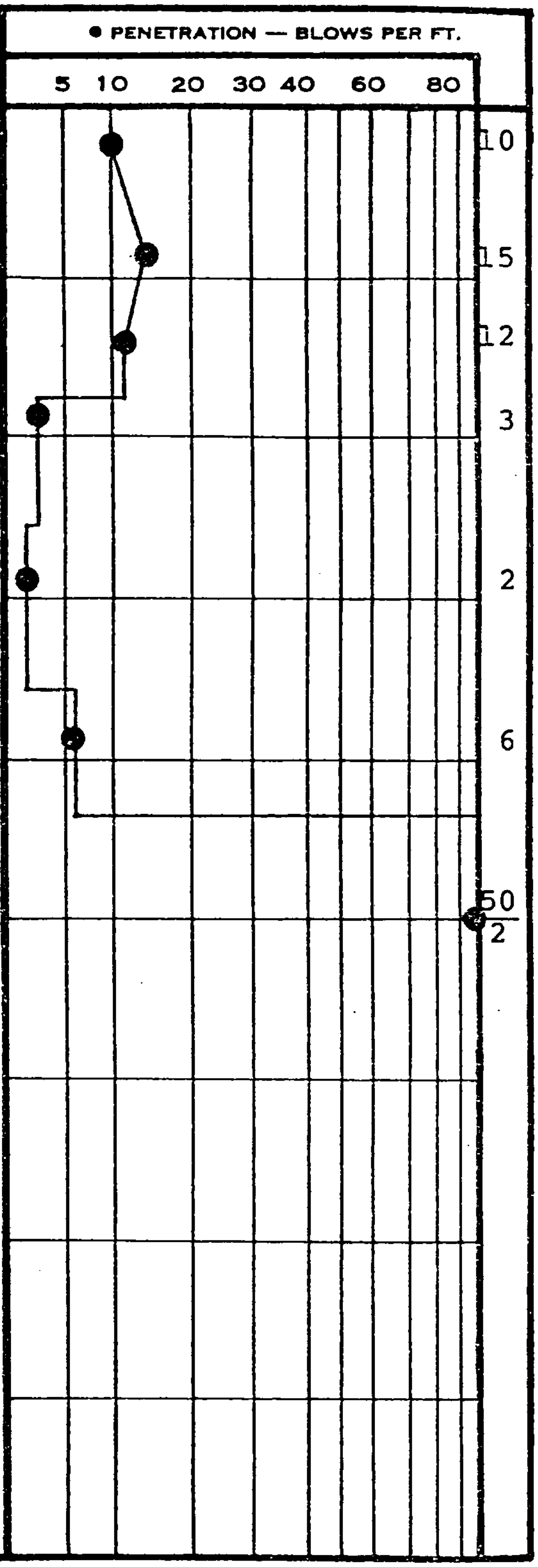
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.
		16.3
	Black Green Silty Fine SAND With Roots (Organically Stained)	
5	Stiff Gray Yellow Brown Silty Fine Very Sandy CLAY	
		7.3
10	Very Loose Gray Clayey Silty Fine SAND	
		4.3
15	Very Soft Black Gray Silty CLAY With Fine Sandy Seams	
		-1.7
20	SHELLS With Gray Clayey Silty Fine SAND	
		-5.7
	Shell and Limestone Fragments	
25		-8.7
	Boring Terminated	
30	Water Level @ 3' @ TOB	
35		
40		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

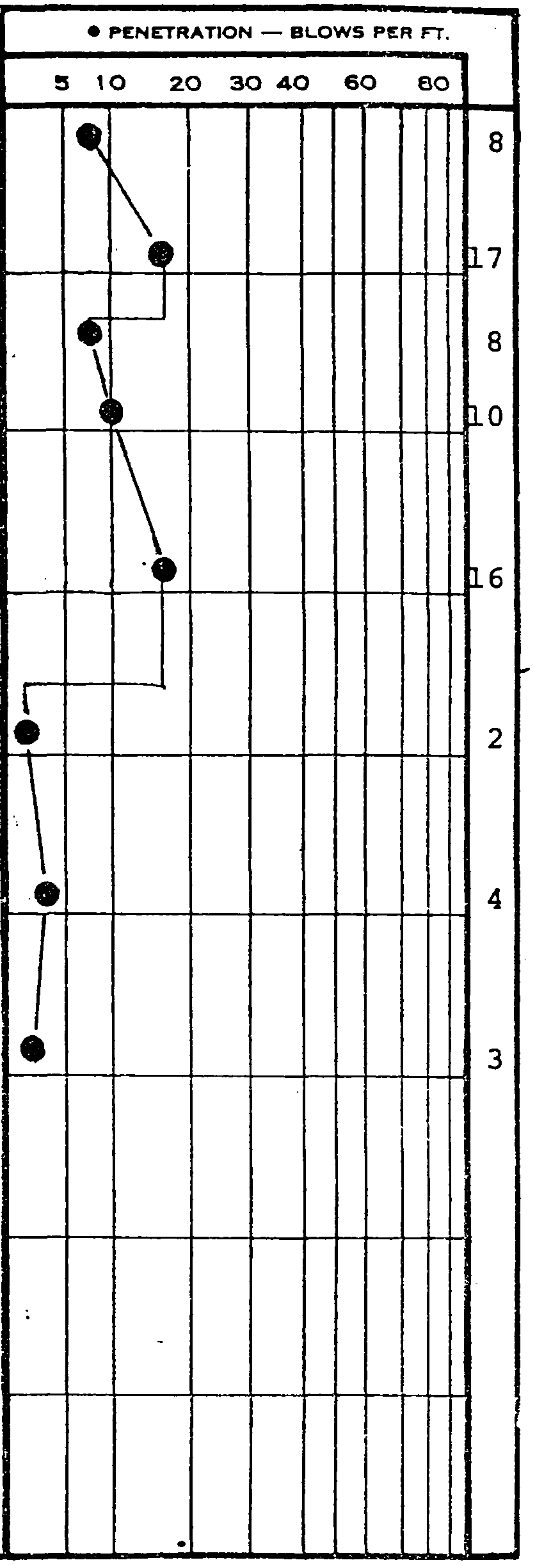
WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-80
DATE DRILLED 12-28-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV.
			16.9
	Black Silty Fine SAND	≡	
5	Firm Gray Brown Fine Sandy CLAY with Roots and Firm Brown Fine to Medium SAND with Gray Silty Clayey Seams		10.4
10 15	Loose Gray Fine to Medium SAND with Shells		-1.1
20 25 30	Very Soft to Soft Gray Silty CLAY with Fine Sandy Seams and Shells		-16.6
35 40	Boring Terminated Encountered Shell Limestone Water Level @ Surface @ TOB		



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

≡ WATER TABLE — 24 HR.

▬ UNDISTURBED SAMPLE ≡ WATER TABLE — 1 HR.

[50] % ROCK CORE RECOVERY ◀ LOSS OF DRILLING WATER

BORING NO. SC-81

DATE DRILLED 1-27-78

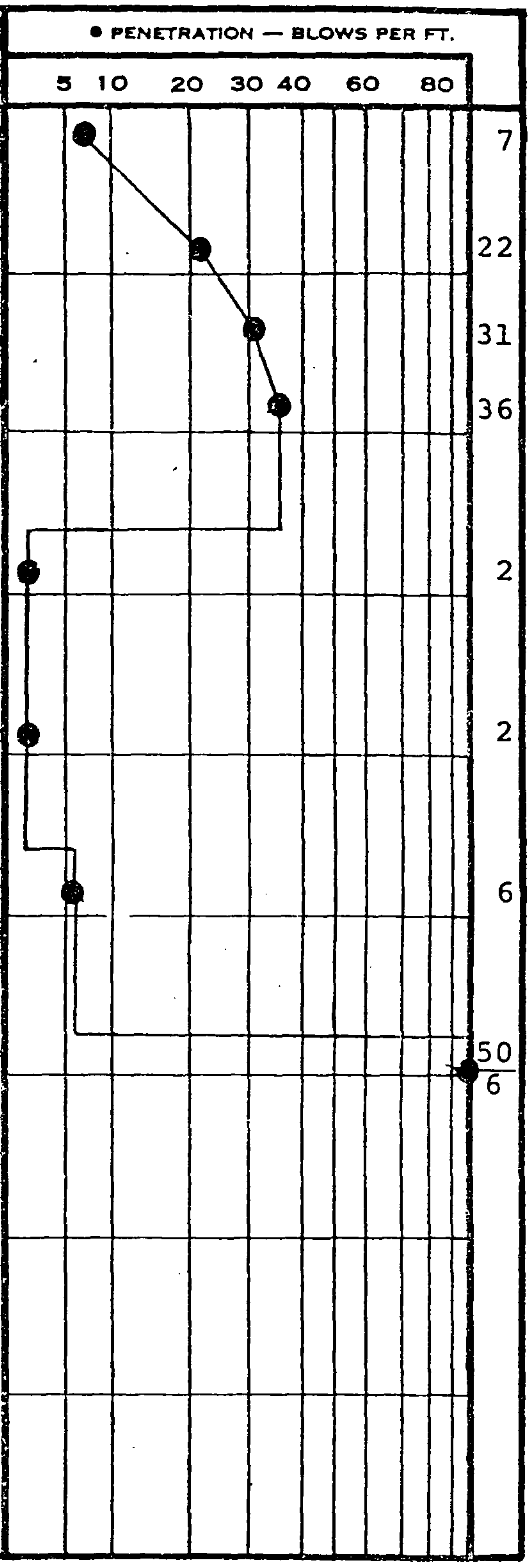
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DESCRIPTION	ELEV.
	24.2
Black Silty Fine SAND With Roots (Organically Stained)	
Loose to Firm Gray Slightly Silty Medium Fine SAND	17.7
Dense Gray Fine SAND With Shells	11.2
Very Loose Gray Silty Fine SAND With Shells	7.2
Very Soft Gray Silty CLAY With Fine Sand Seams and Shells	1.2
SHELLS With Silty Fine to Medium SAND	-6.8
Boring Terminated Encountered Shell Limestone Water Level @ 3' @ TOB	



SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I. D. SAMPLER 1 FT.

WATER TABLE -- 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-84
DATE DRILLED 12-28-77
JOB NO. SS7735
PAGE 1 OF 1

ATTACHMENT 3

CPT Sounding Data

Attachment 3-A

CPT Sounding Logs
(provided by Mid-Atlantic Drilling, Inc.)

Geosyntec consultants

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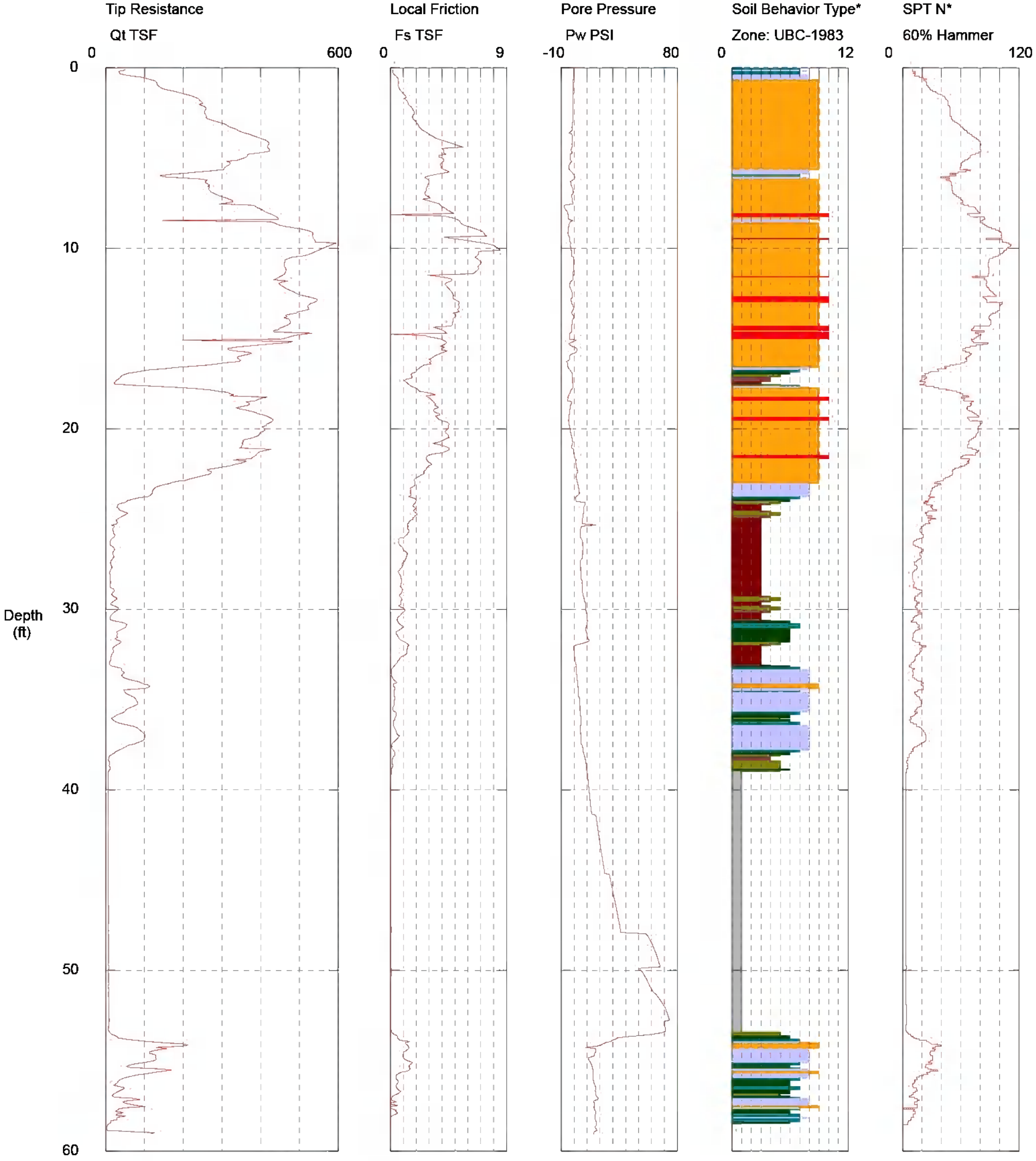
Operator: Cory Robison
Sounding: CPT-122
Cone Used: DSG0867

CPT Date/Time: 10/7/2013 4:54:35 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 59.06 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 (*)

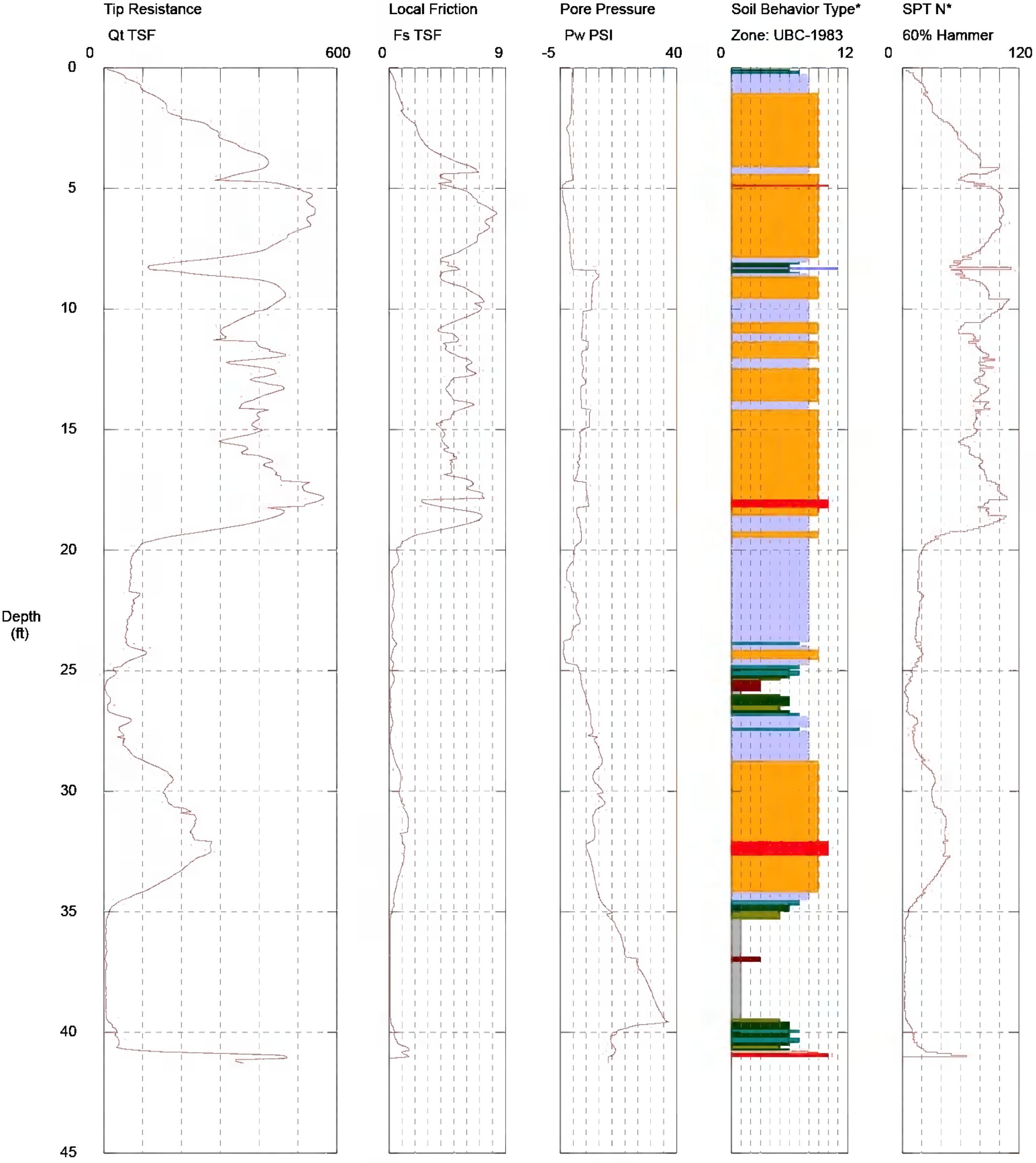
Geosyntec

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

Geosyntec
consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-123
Cone Used: DSG0867

CPT Date/Time: 10/7/2013 3:10:57 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 41.27 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

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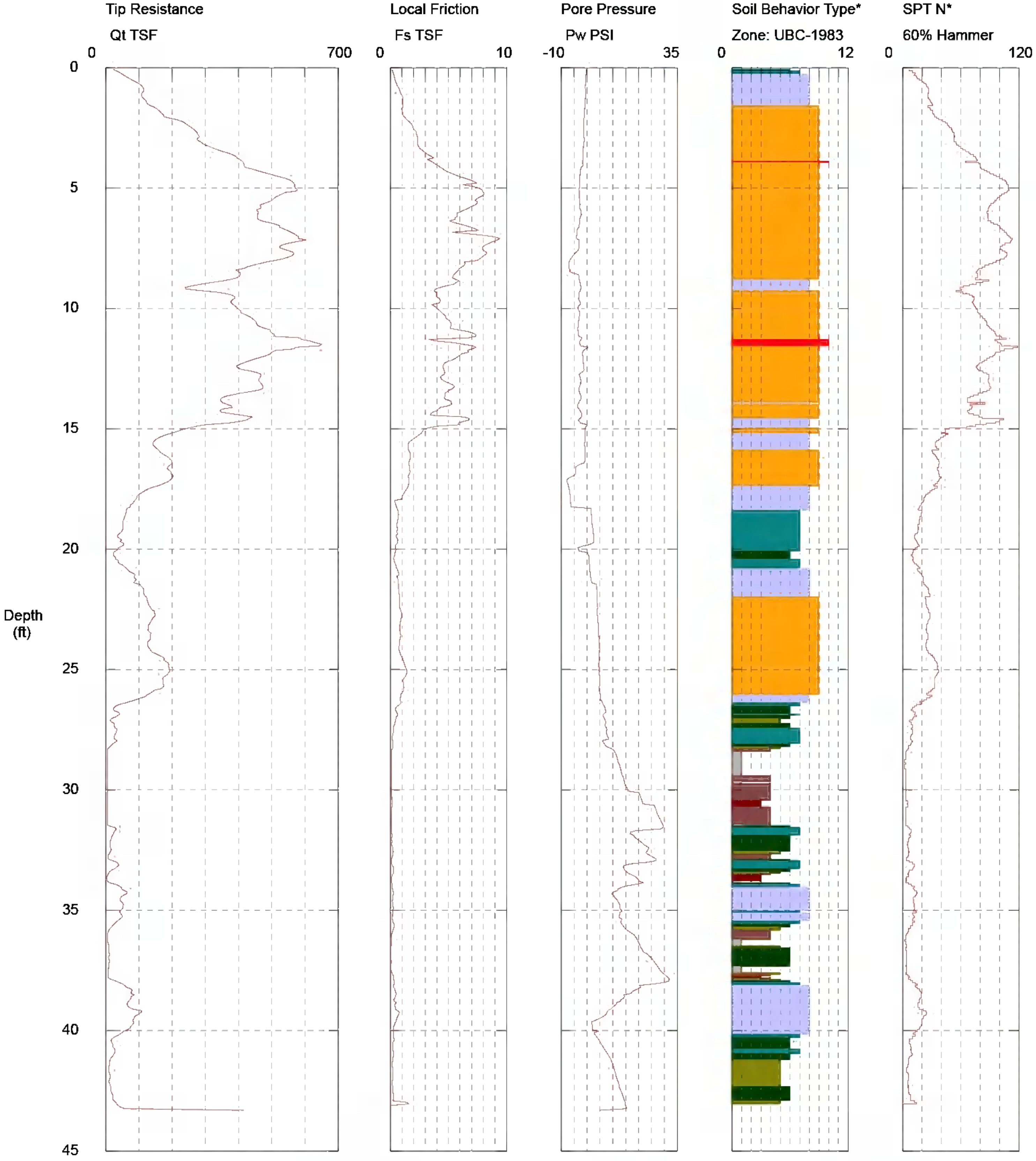
Operator: Cory Robison
Sounding: CPT-124
Cone Used: DSG0867

CPT Date/Time: 10/4/2013 12:52:27 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 43.31 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

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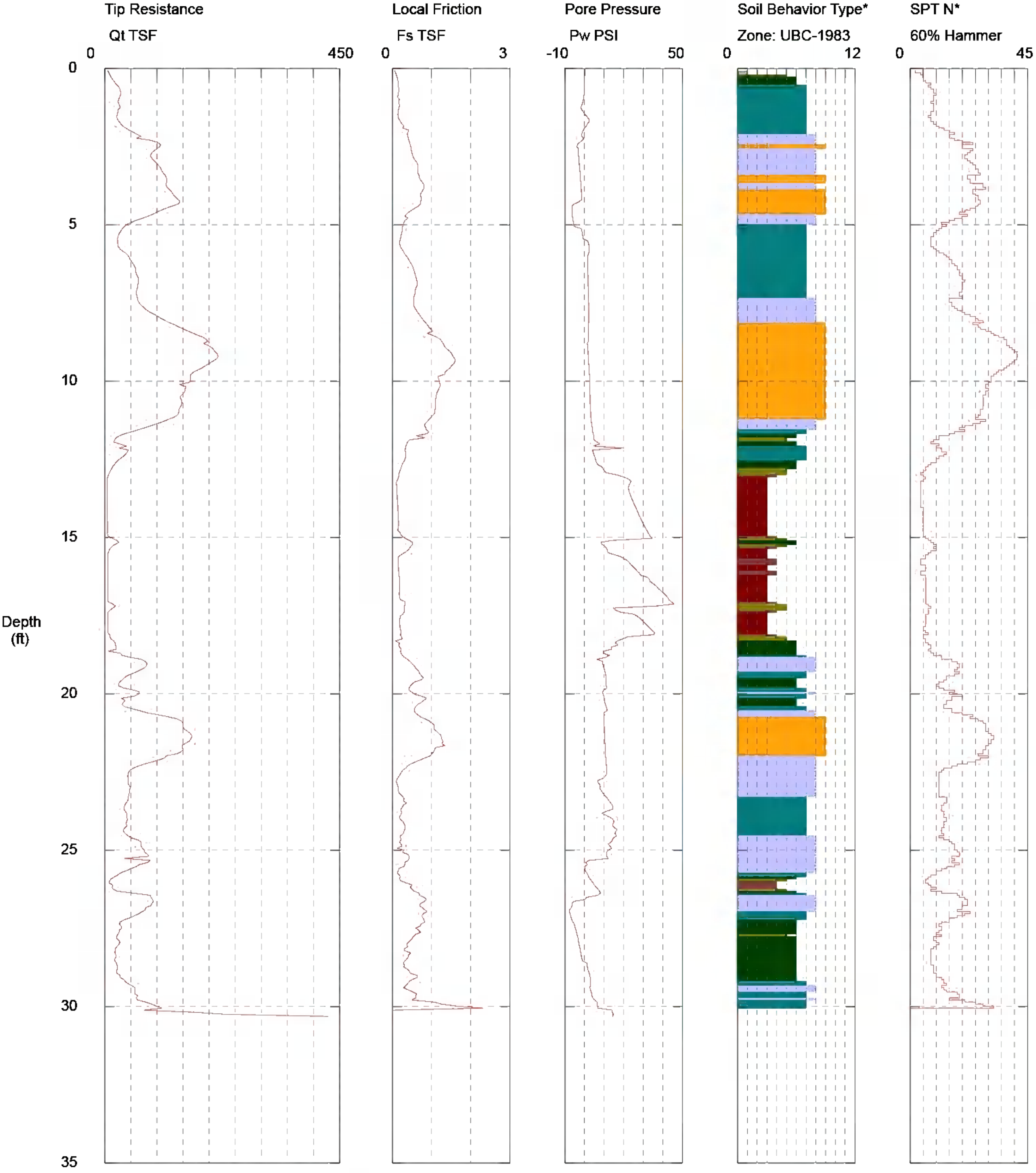
Operator: Cory Robison
Sounding: CPT-125
Cone Used: DSG0867

CPT Date/Time: 10/7/2013 2:00:35 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 30.31 feet

Depth Increment = 0.066 feet

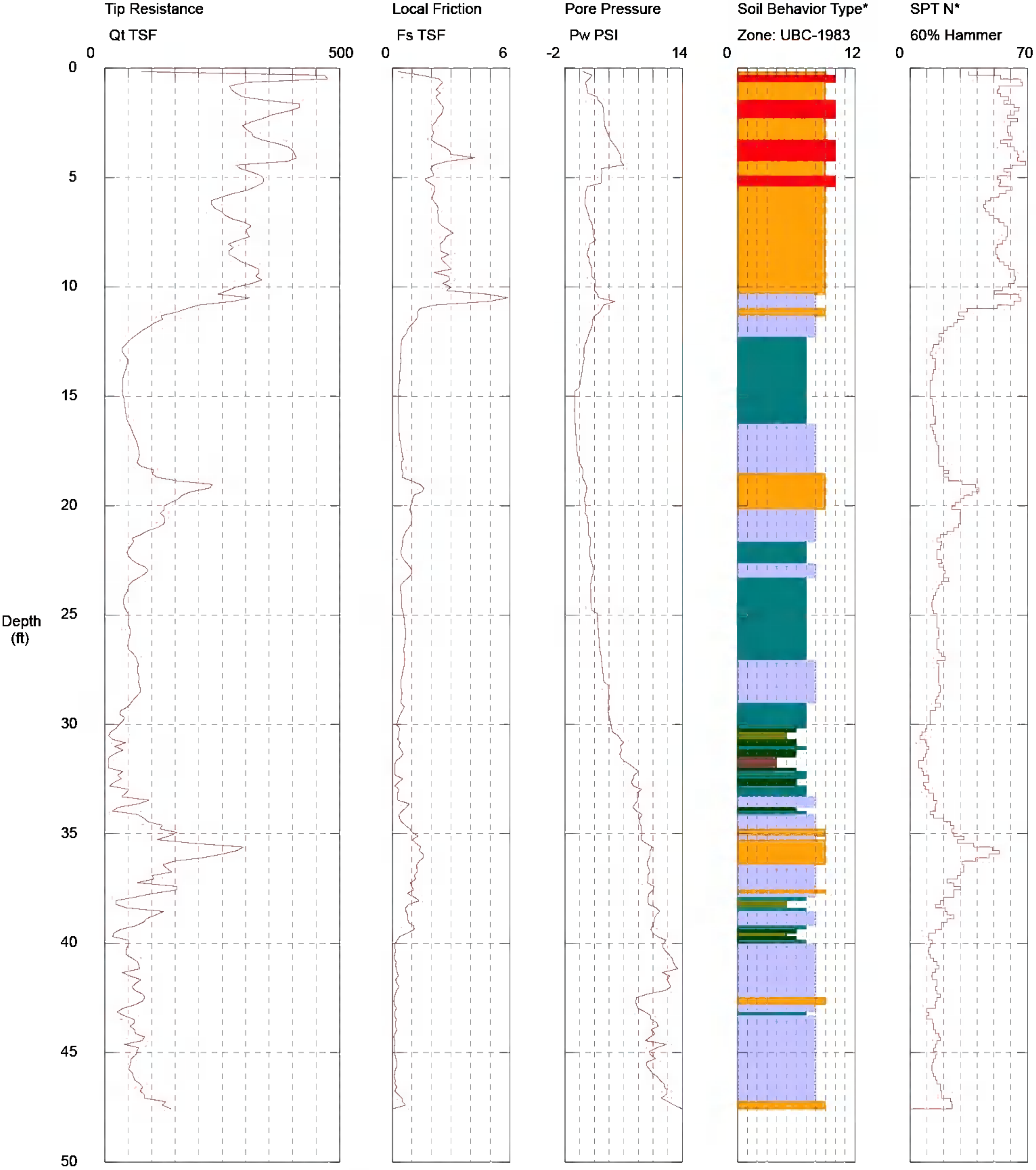
▨ 1 sensitive fine grained	▨ 4 silty clay to clay	▨ 7 silty sand to sandy silt	▨ 10 gravelly sand to sand
▨ 2 organic material	▨ 5 clayey silt to silty clay	▨ 8 sand to silty sand	▨ 11 very stiff fine grained (*)
▨ 3 clay	▨ 6 sandy silt to clayey silt	▨ 9 sand	▨ 12 sand to clayey sand (*)

Geosyntec

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

Operator: Cory Robison
Sounding: CPT-126
Cone Used: DSG1156

CPT Date/Time: 9/23/2013 2:34:57 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 47.57 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 (*)

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

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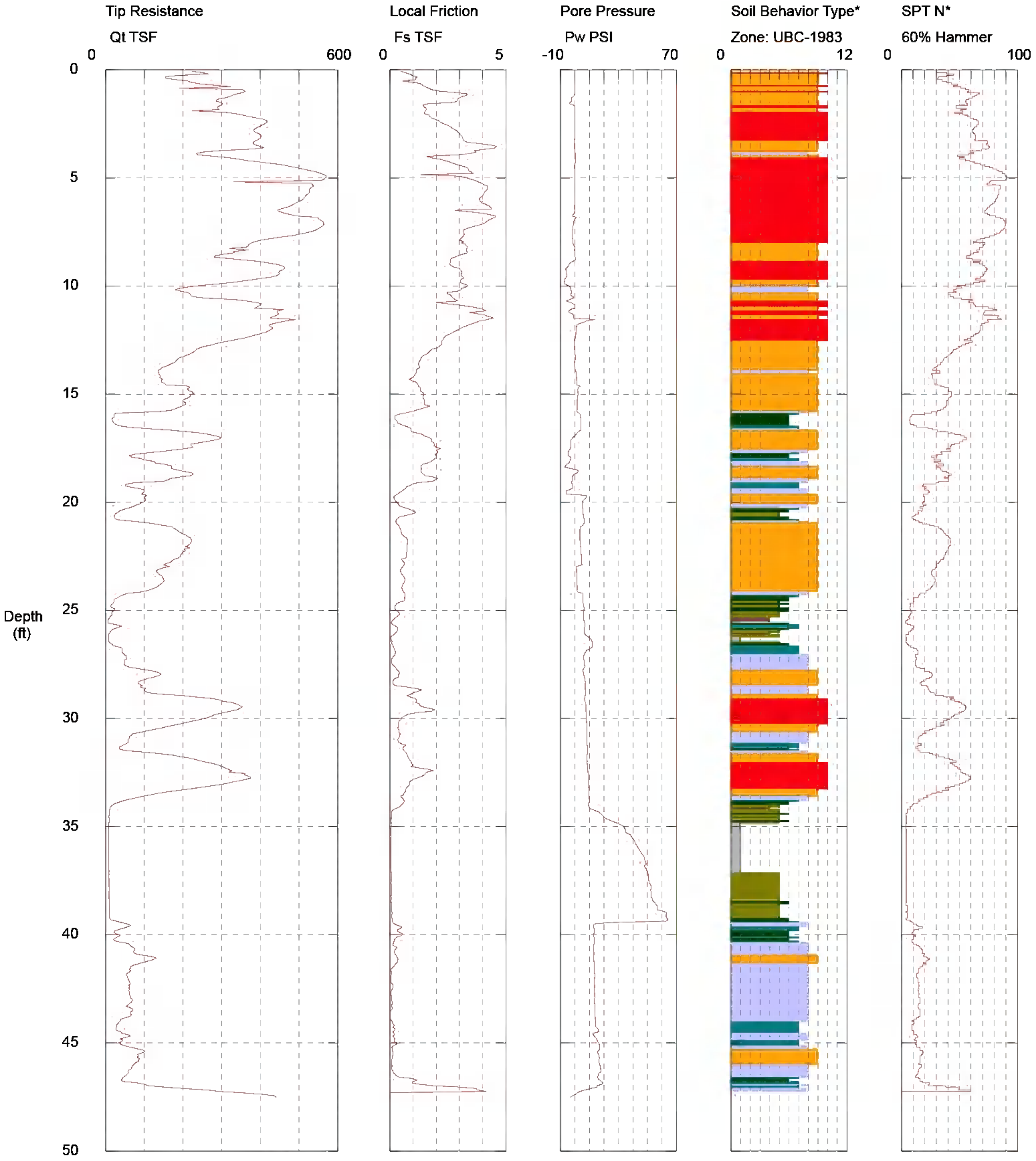
Operator: Cory Robison
Sounding: CPT-128
Cone Used: DDG1195

CPT Date/Time: 10/4/2013 10:41:08 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 47.51 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

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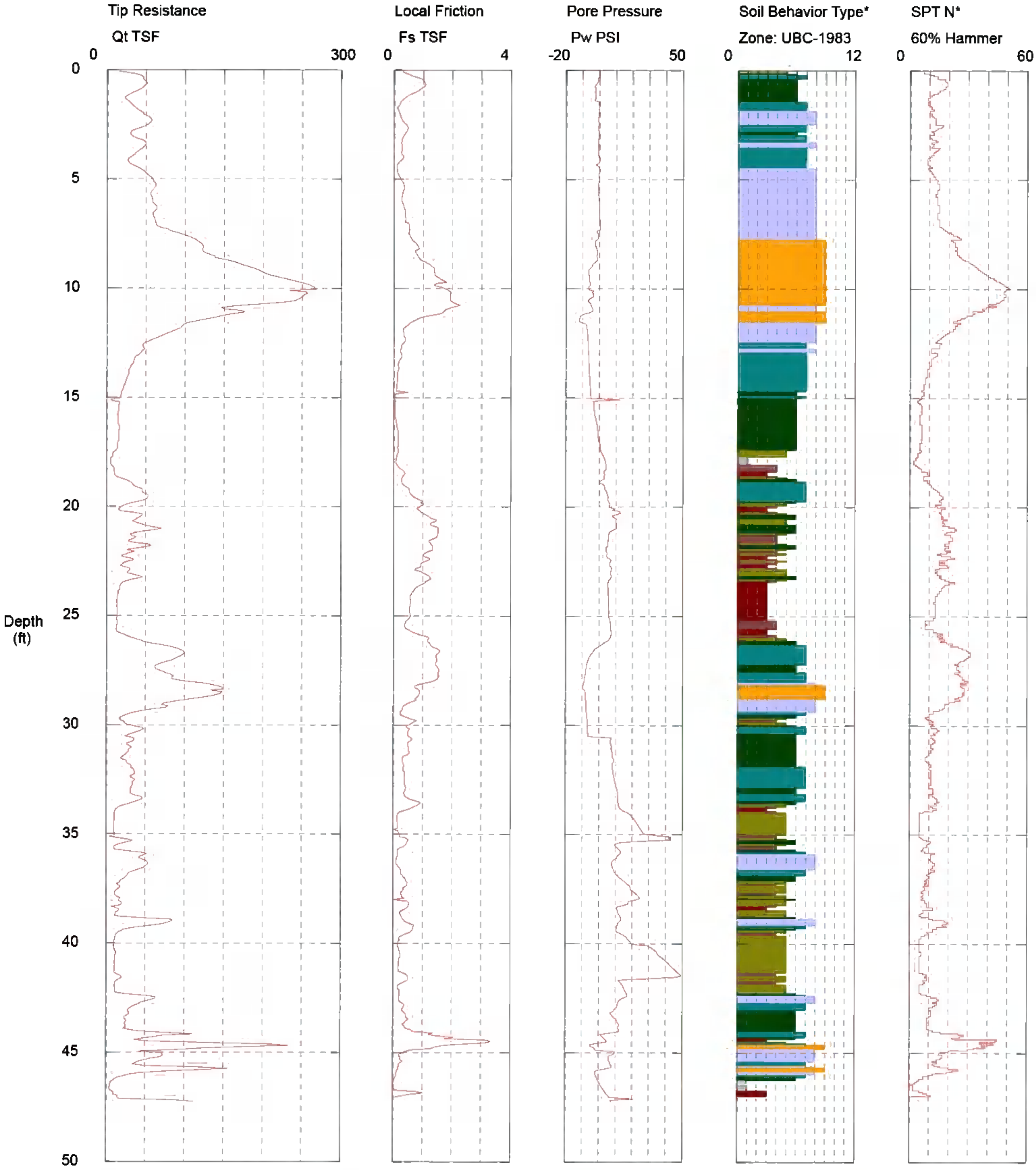
Operator: Cory Robison
Sounding: CPT-129
Cone Used: DSG0867

CPT Date/Time: 10/8/2013 8:33:59 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 47.24 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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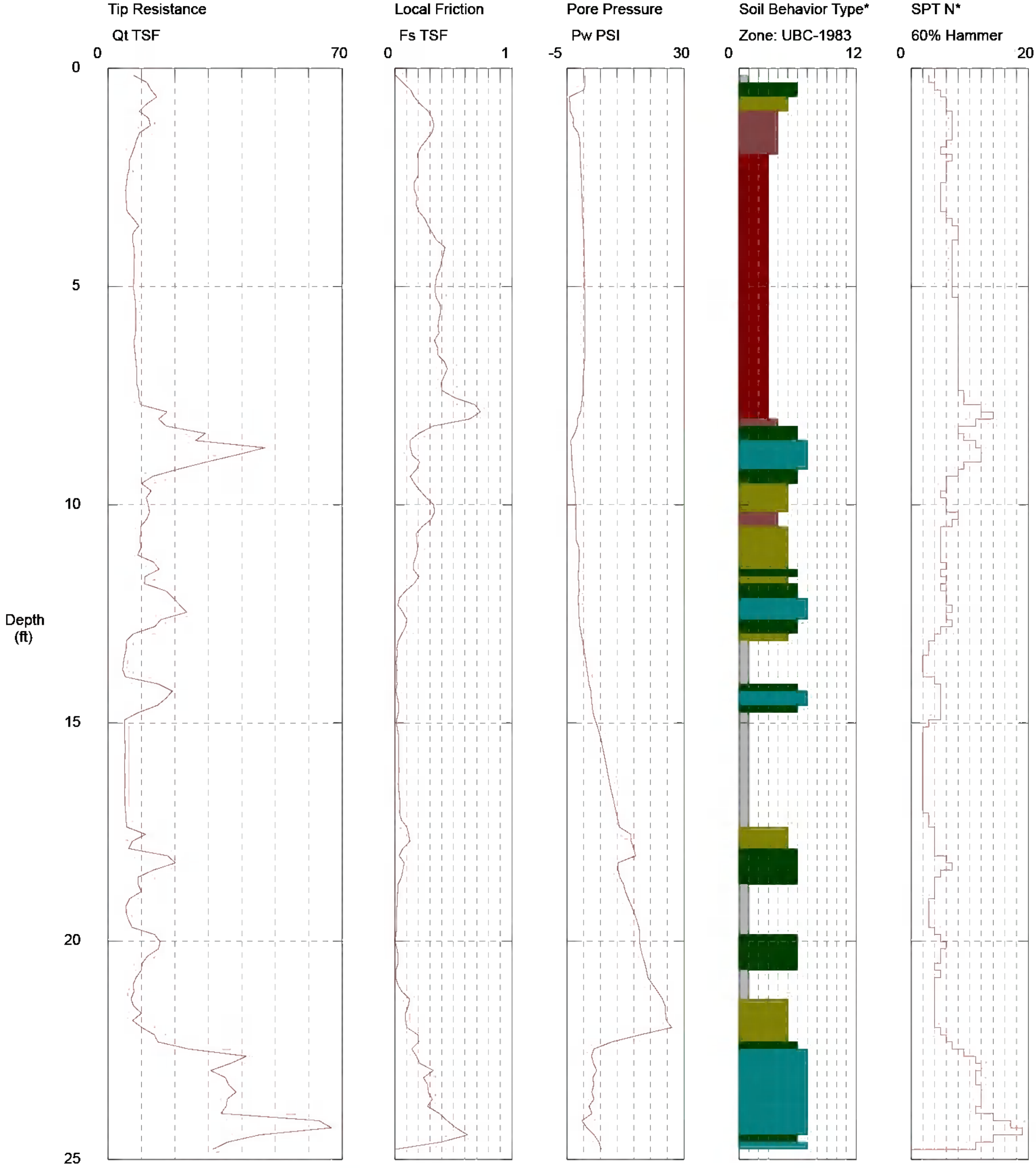
Operator: Cory Robison
Sounding: CPT-130
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 10:41:33 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 24.77 feet

Depth Increment = 0.164 feet

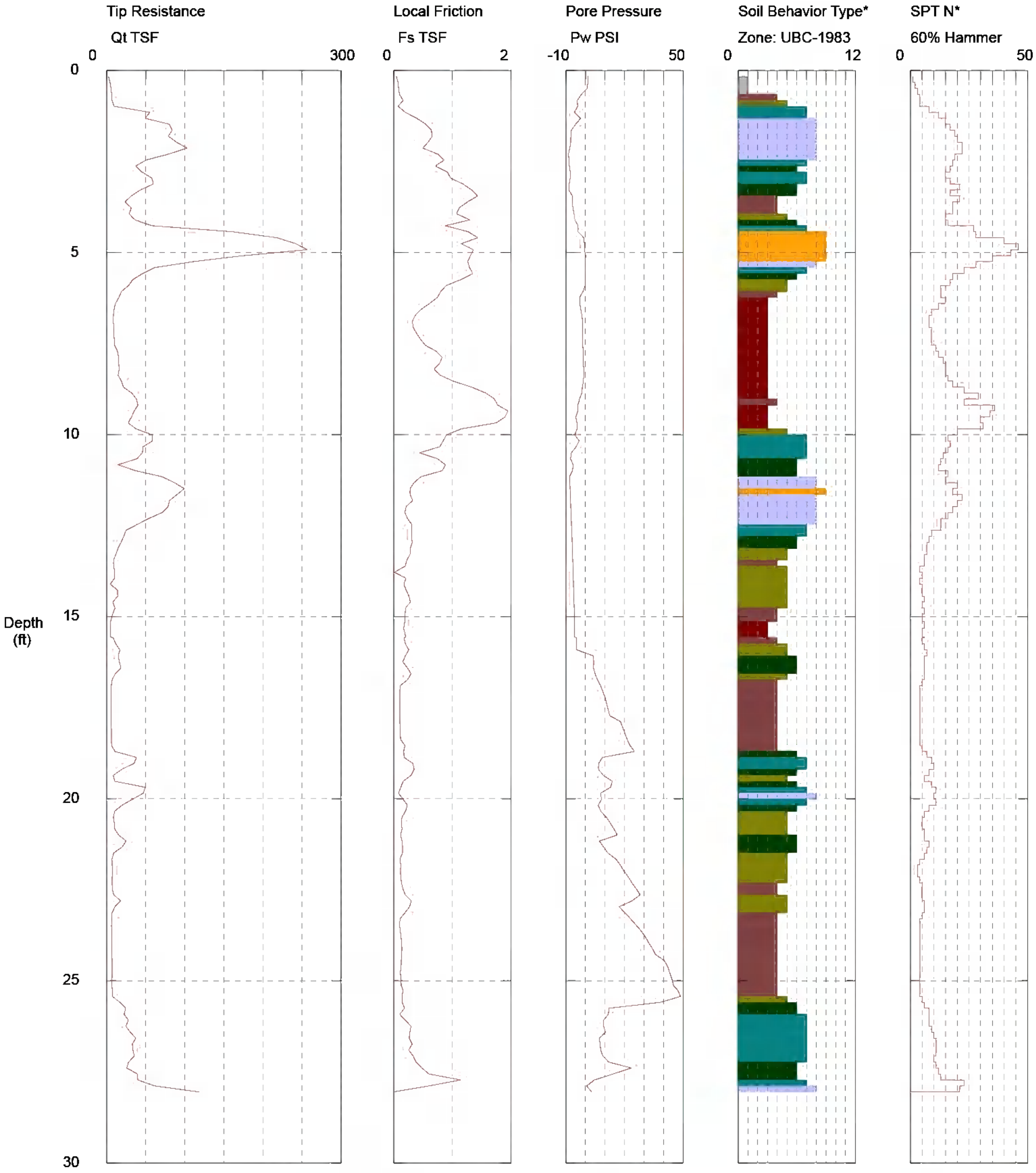
▨ 1 sensitive fine grained	▨ 4 silty clay to clay	▨ 7 silty sand to sandy silt	🟥 10 gravelly sand to sand
▨ 2 organic material	▨ 5 clayey silt to silty clay	▨ 8 sand to silty sand	▨ 11 very stiff fine grained (*)
▨ 3 clay	▨ 6 sandy silt to clayey silt	▨ 9 sand	▨ 12 sand to clayey sand (*)

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

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-130A
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 11:25:10 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 28.05 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 (*)

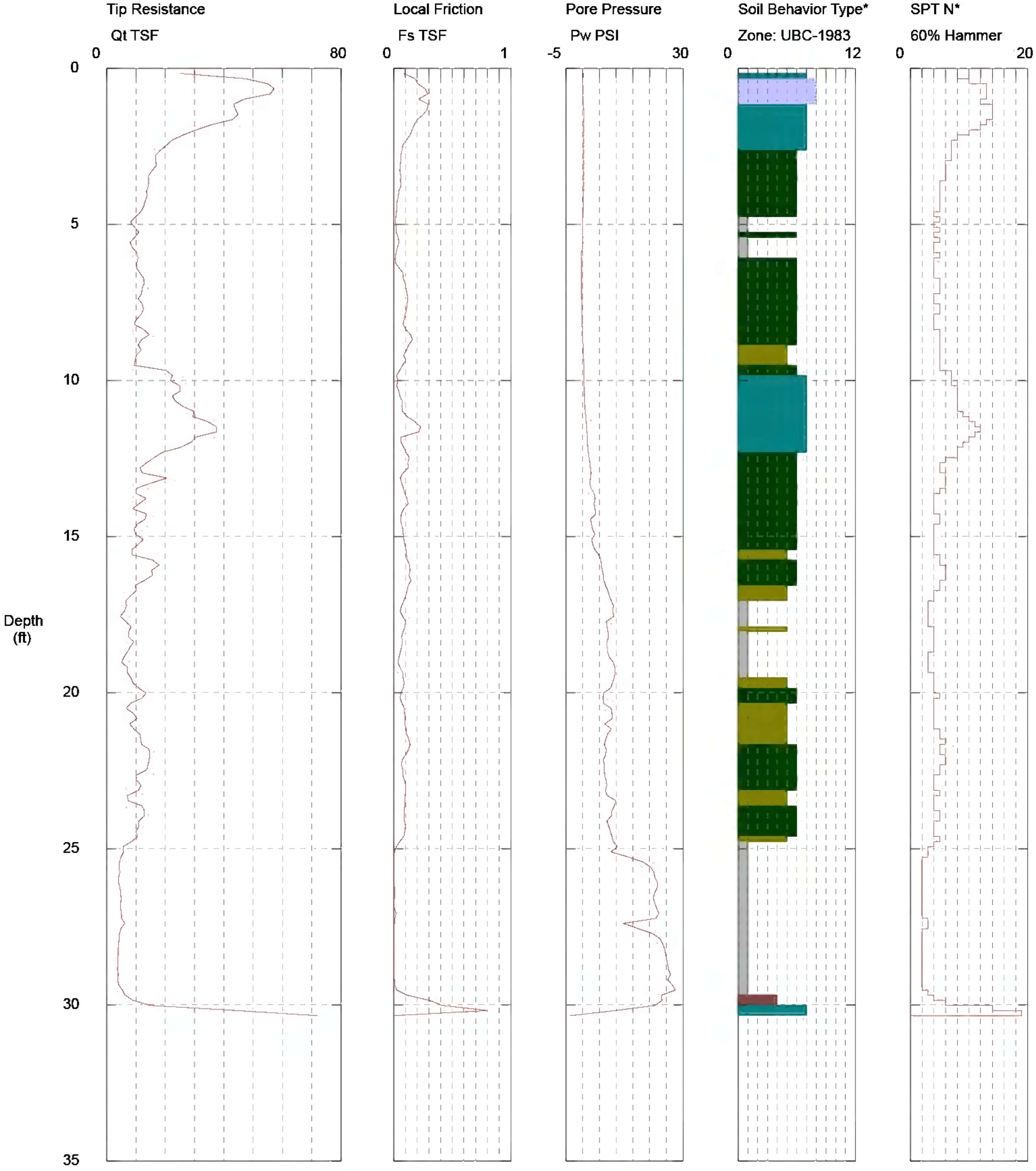
Geosyntec

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

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-131
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 9:33:29 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 30.35 feet

Depth Increment = 0.164 feet

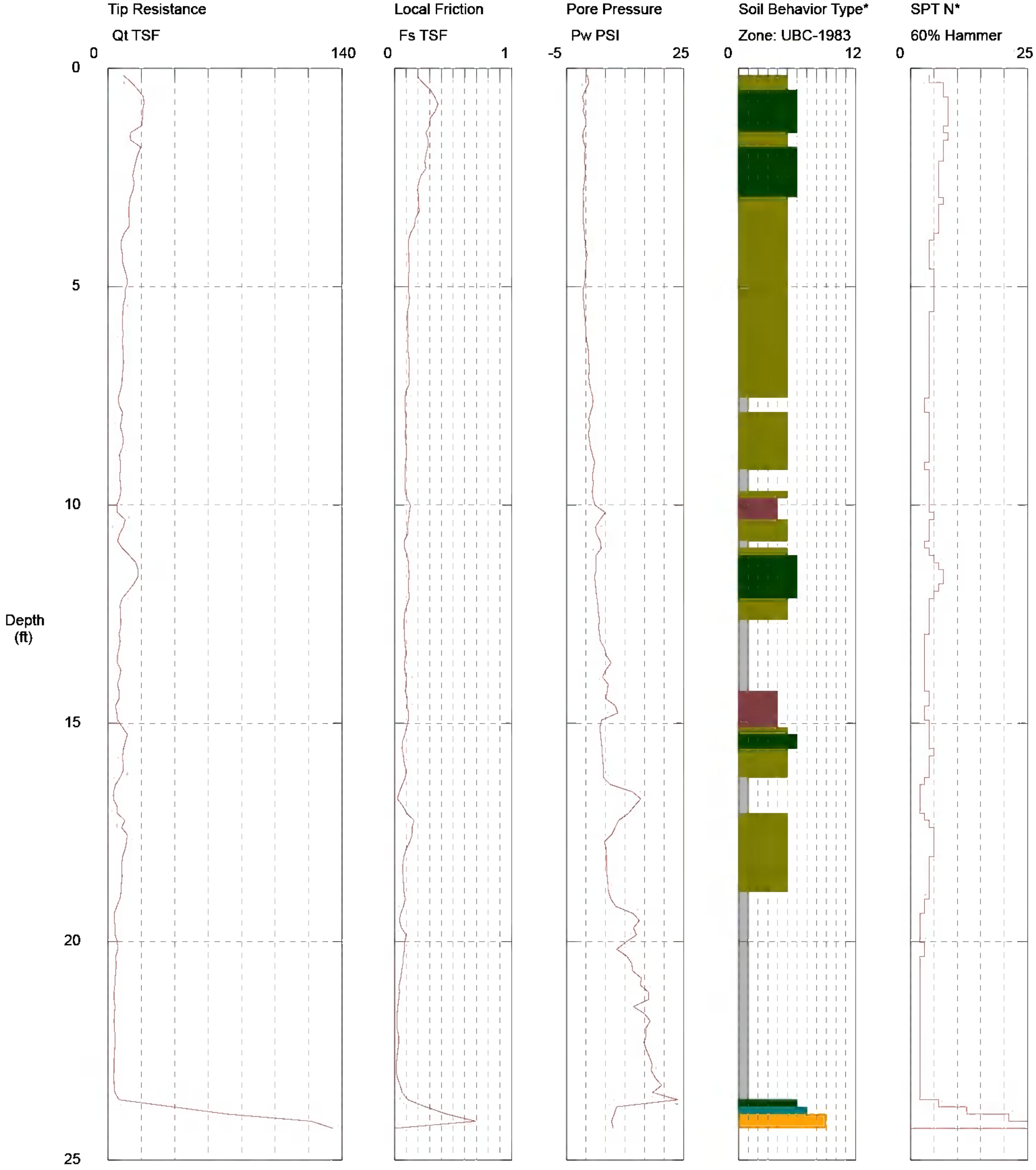
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Geosyntec
consultants
engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-132
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 3:20:51 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 24.28 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

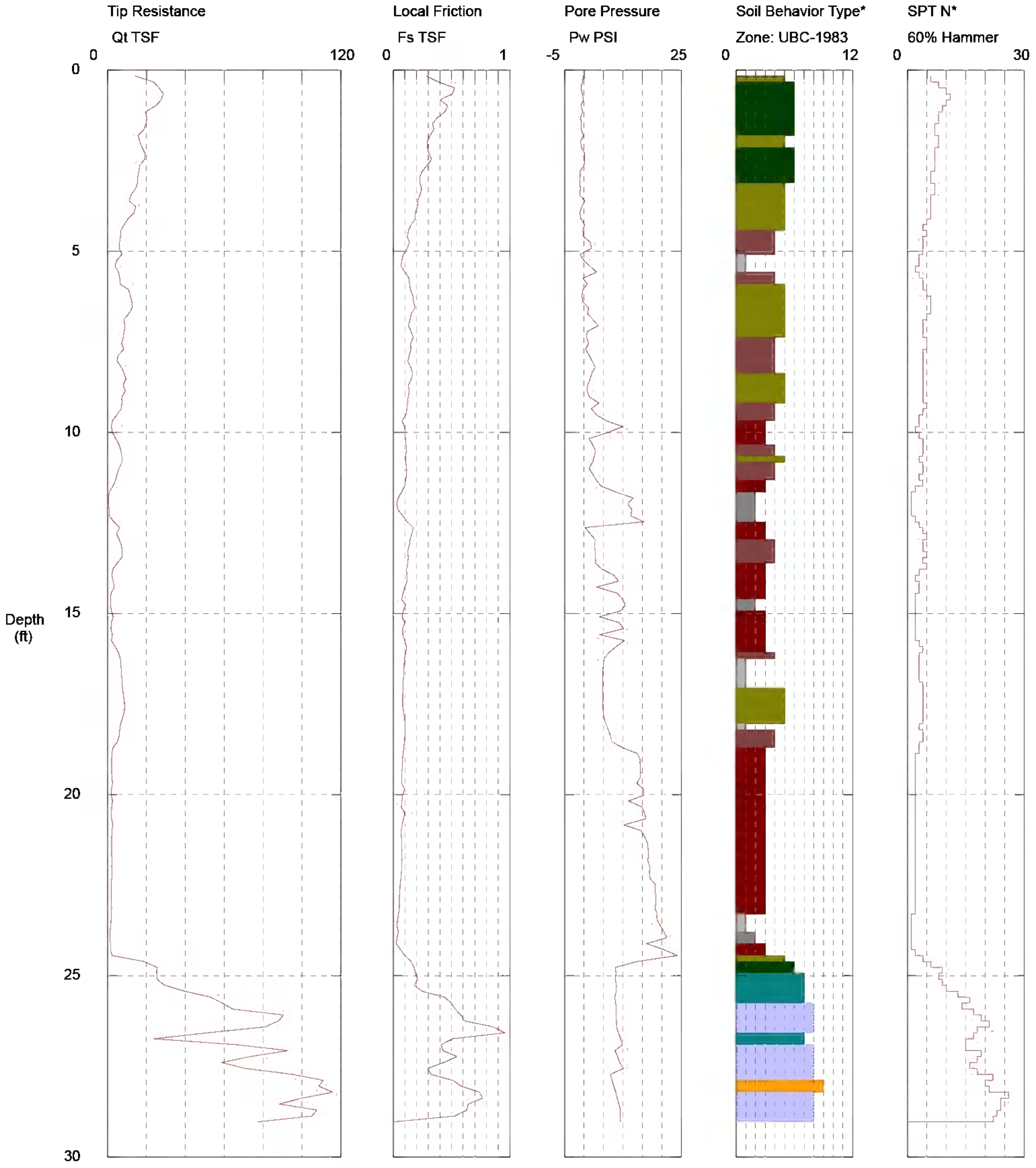
*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants

engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-133
Cone Used: DSG1156

CPT Date/Time: 9/23/2013 4:24:30 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 29.04 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 (*)

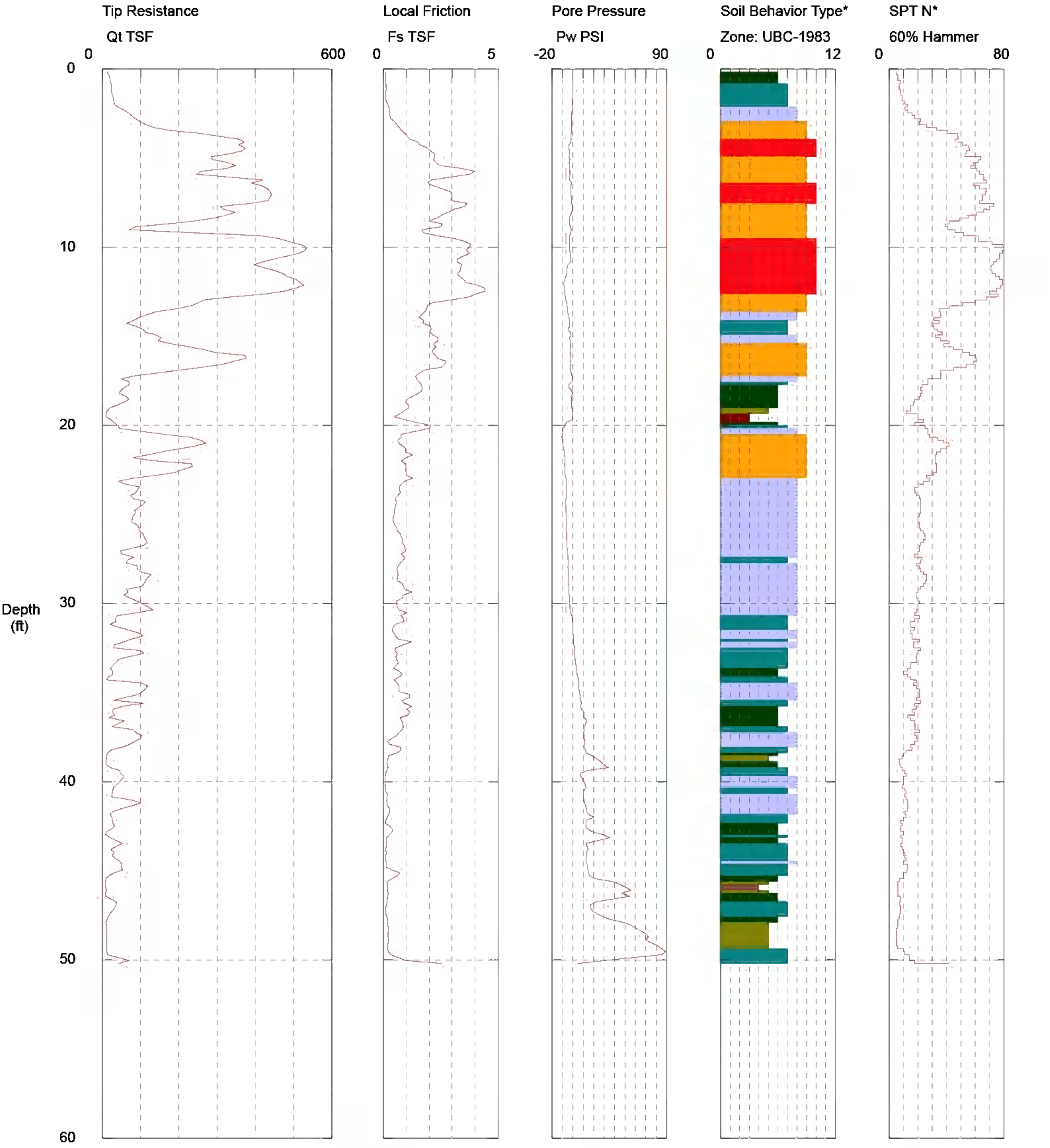
Geosyntec

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

MID-ATLANTIC DRILLING

Operator: Cory Robison
Sounding: CPT-204
Cone Used: DDG1242

CPT Date/Time: 3/1/2016 3:19:11 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 50.20 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 (*)

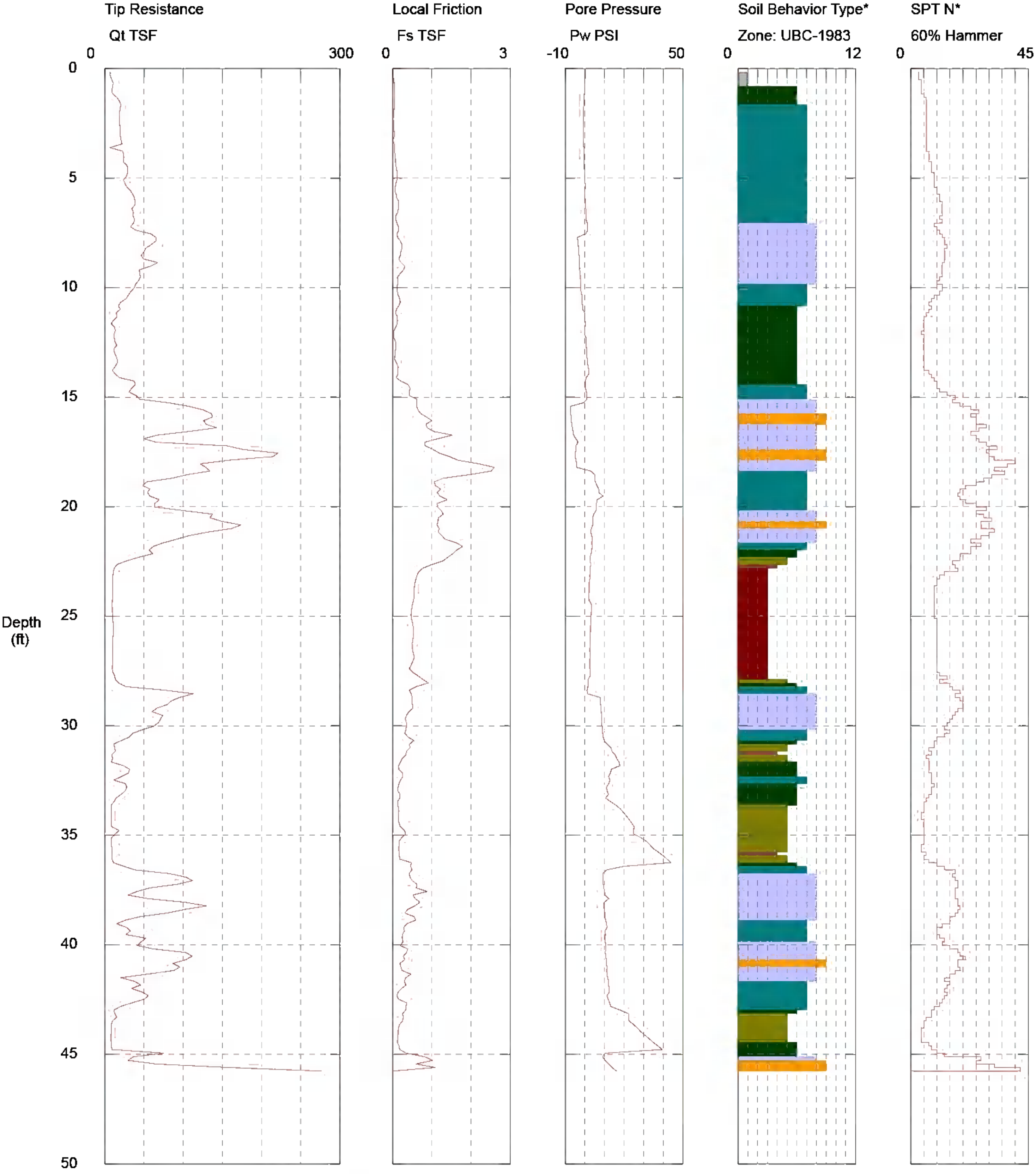
South Ash Pond

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

MID-ATLANTIC DRILLING

Operator: Cory Robison
Sounding: CPT-205
Cone Used: DDG1242

CPT Date/Time: 3/1/2016 2:10:28 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 45.77 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 (*)

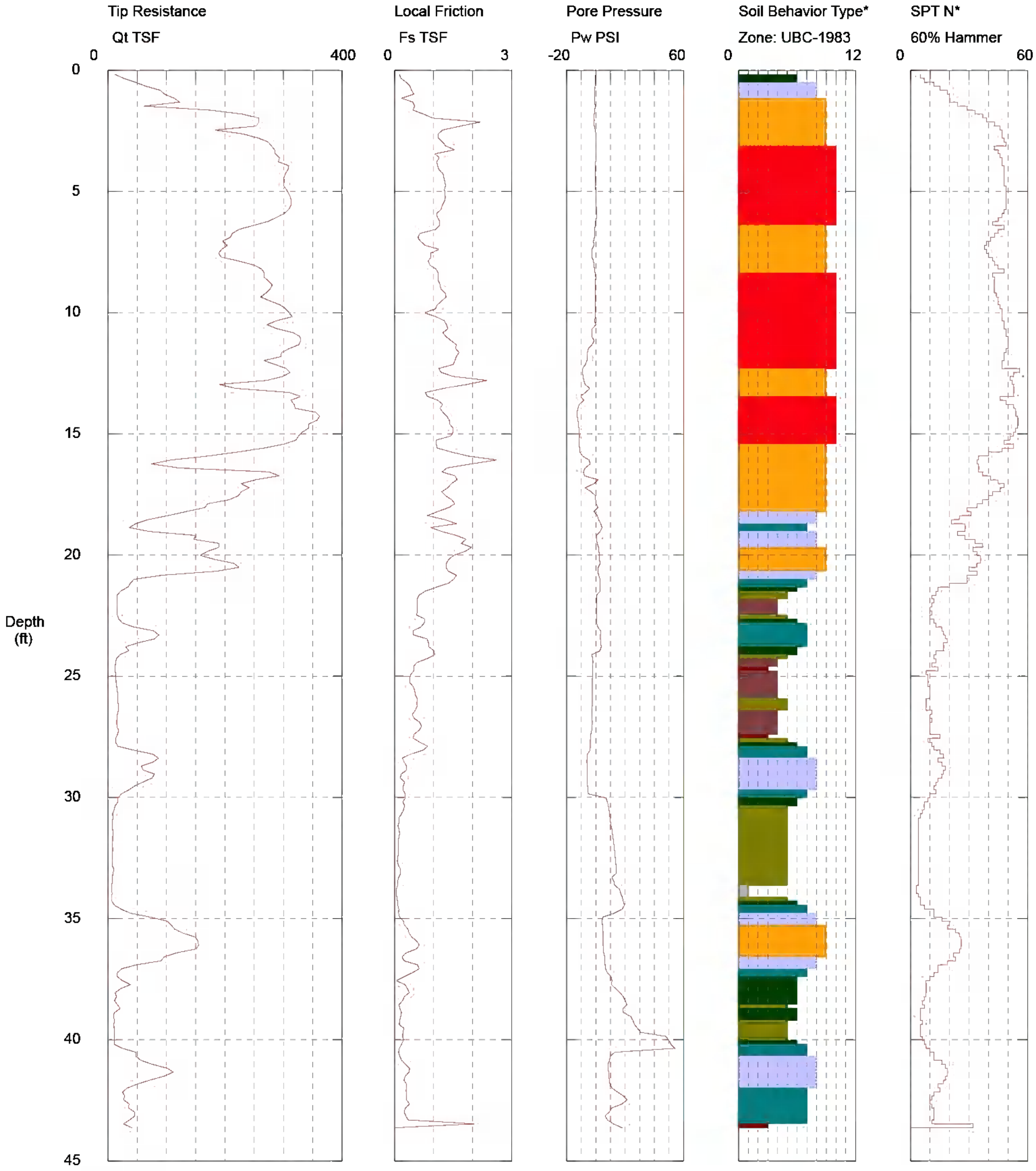
South Ash Pond

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

MID-ATLANTIC DRILLING

Operator: Cory Robison
Sounding: CPT-206
Cone Used: DDG1242

CPT Date/Time: 3/1/2016 12:43:25 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 43.64 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 (*)

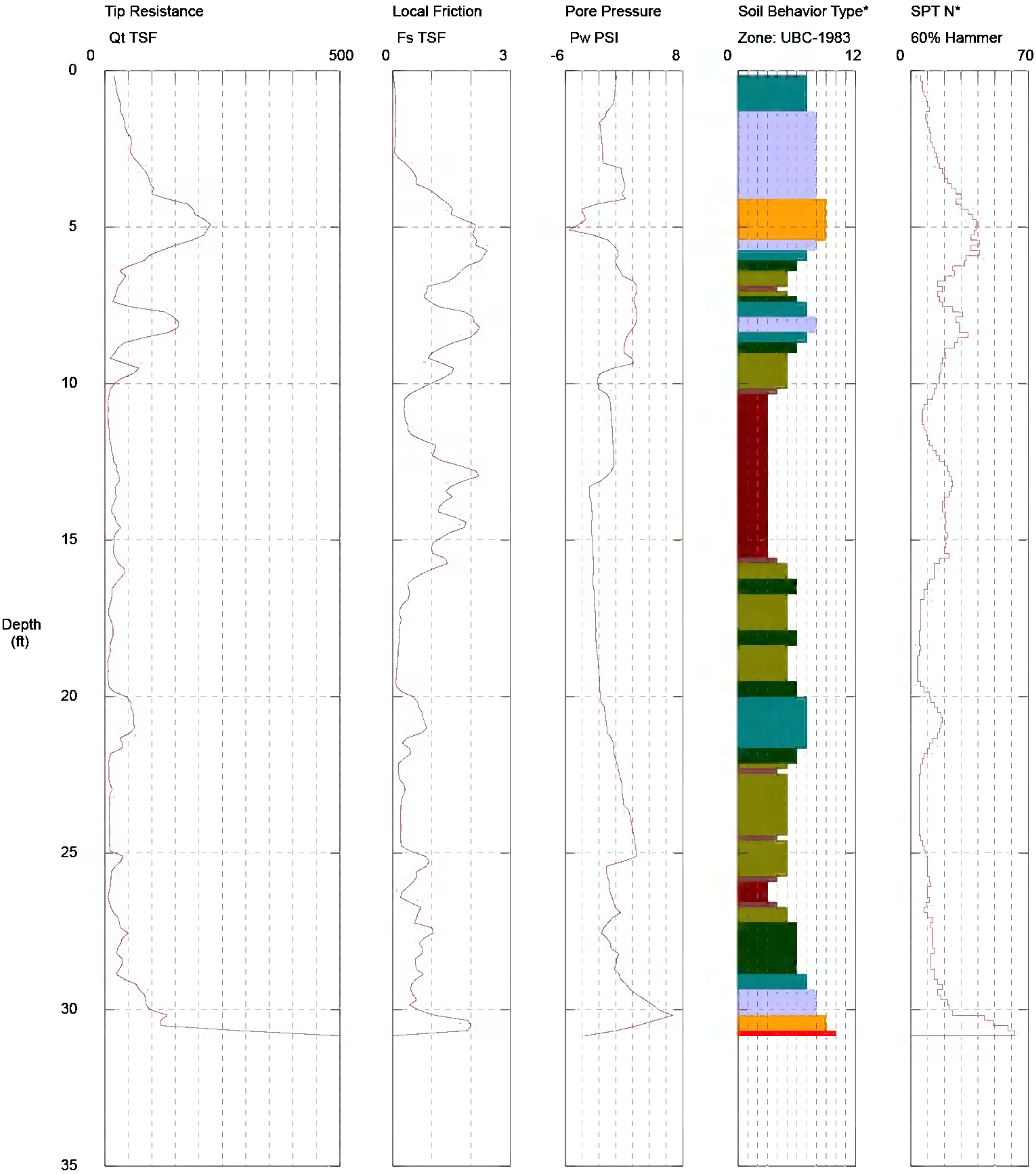
South Ash Pond

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

MID-ATLANTIC DRILLING

Operator: Cory Robison
Sounding: CPT-207
Cone Used: DSG1156

CPT Date/Time: 3/1/2016 10:22:41 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 30.84 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 (*)

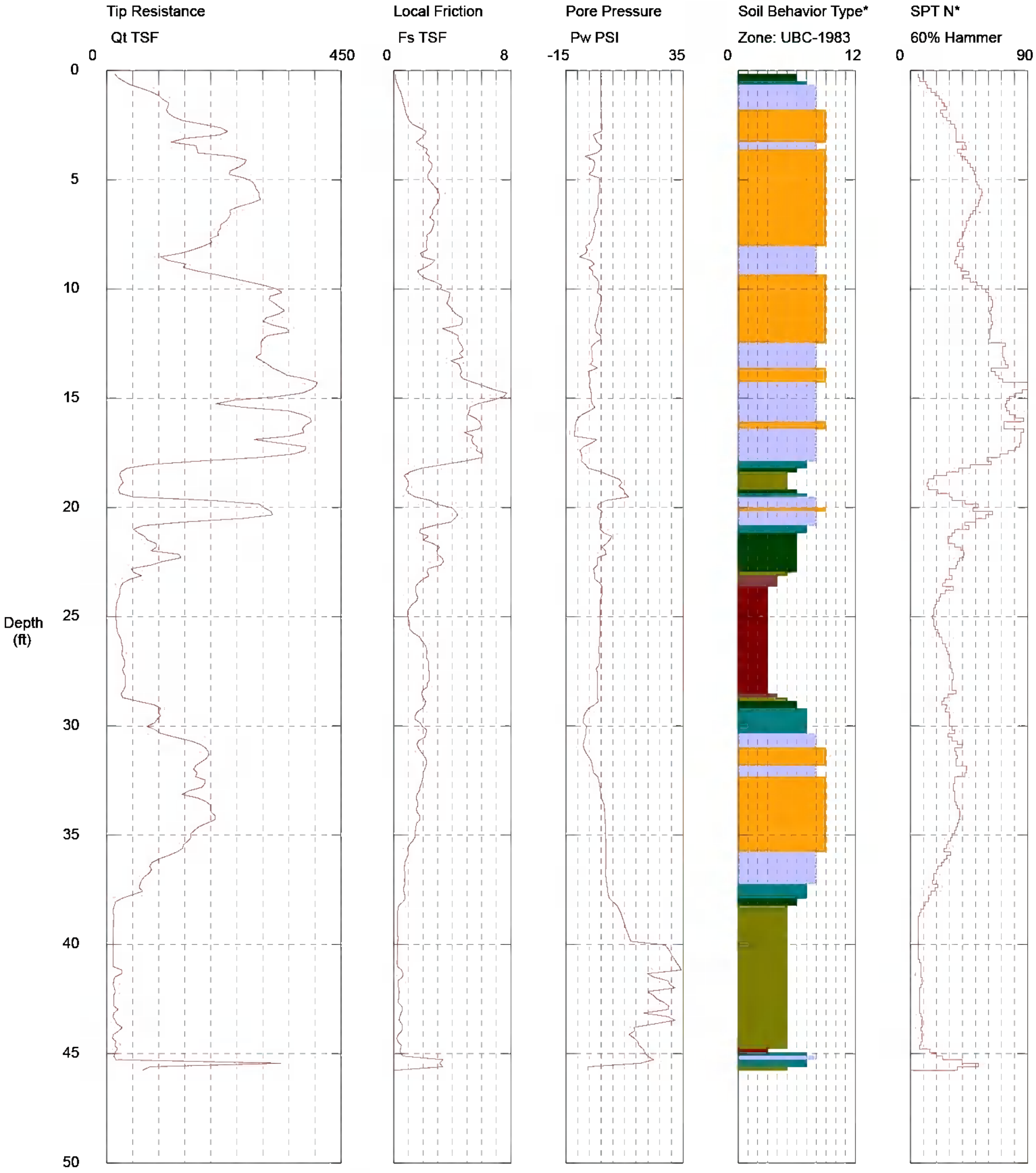
South Ash Pond

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

MID-ATLANTIC DRILLING

Operator: Cory Robison
Sounding: CPT-208
Cone Used: DSG1156

CPT Date/Time: 3/1/2016 8:25:23 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 45.77 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 (*)

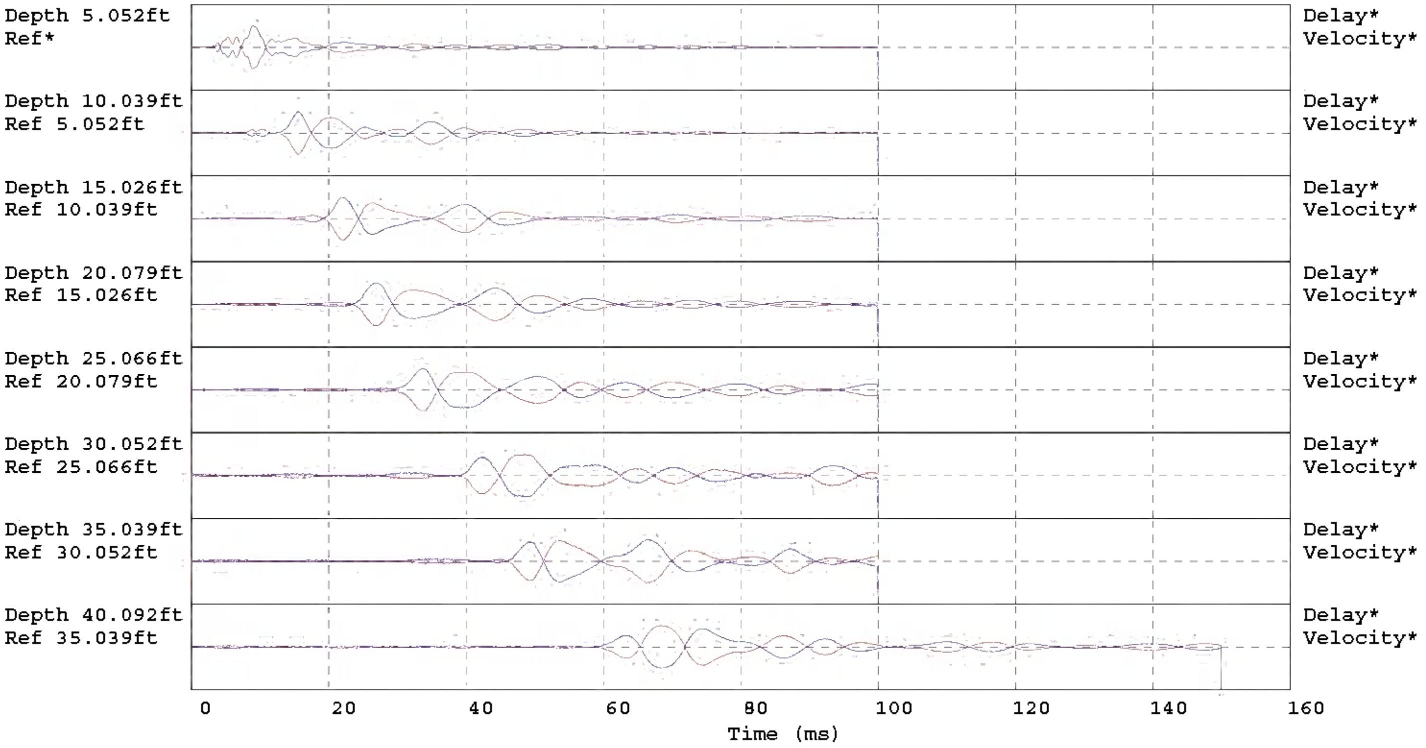
South Ash Pond

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

Attachment 3-B

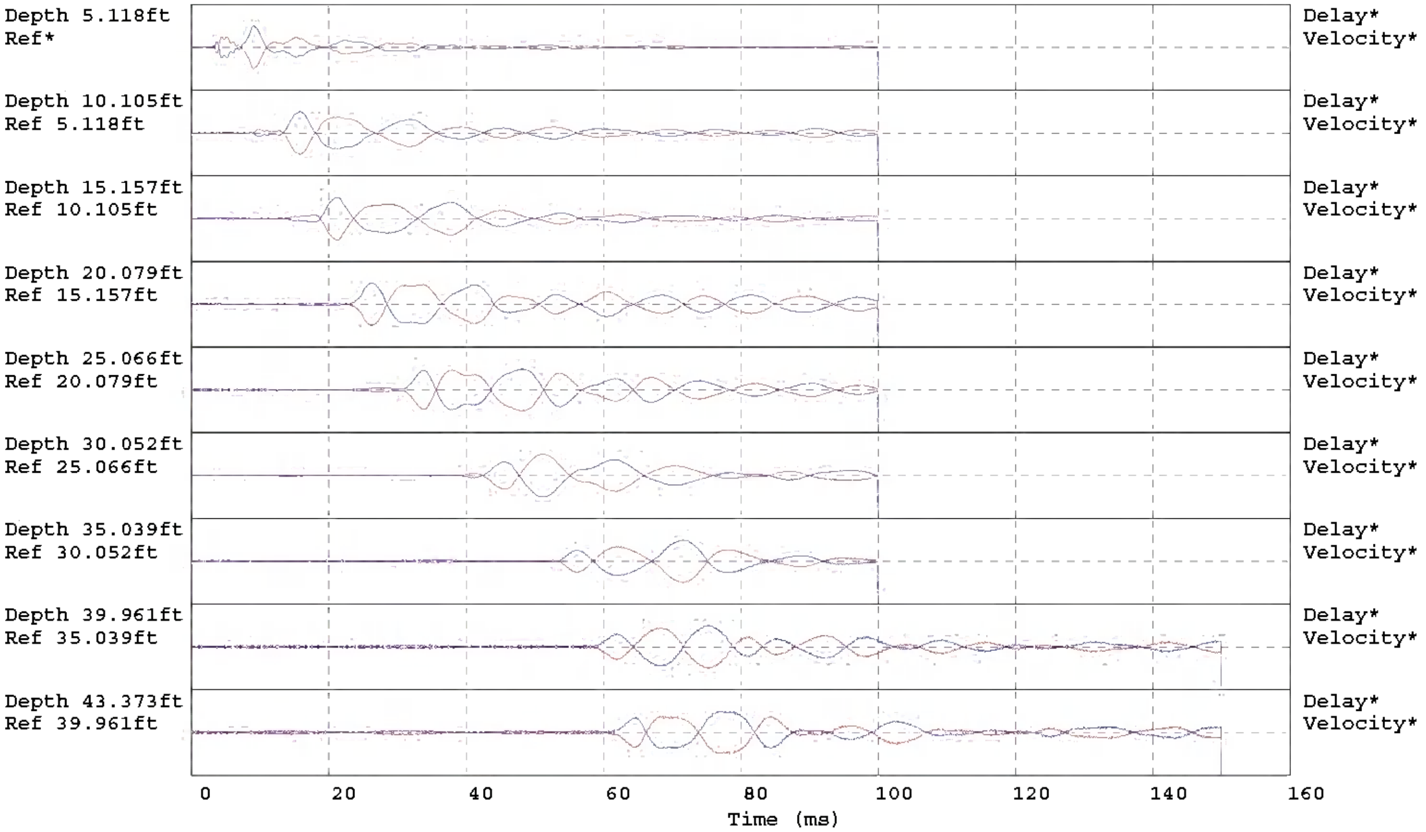
CPT Shear Wave Velocity Data
(provided by Mid-Atlantic Drilling, Inc.)

Mid Atlantic Drilling
CPT-123



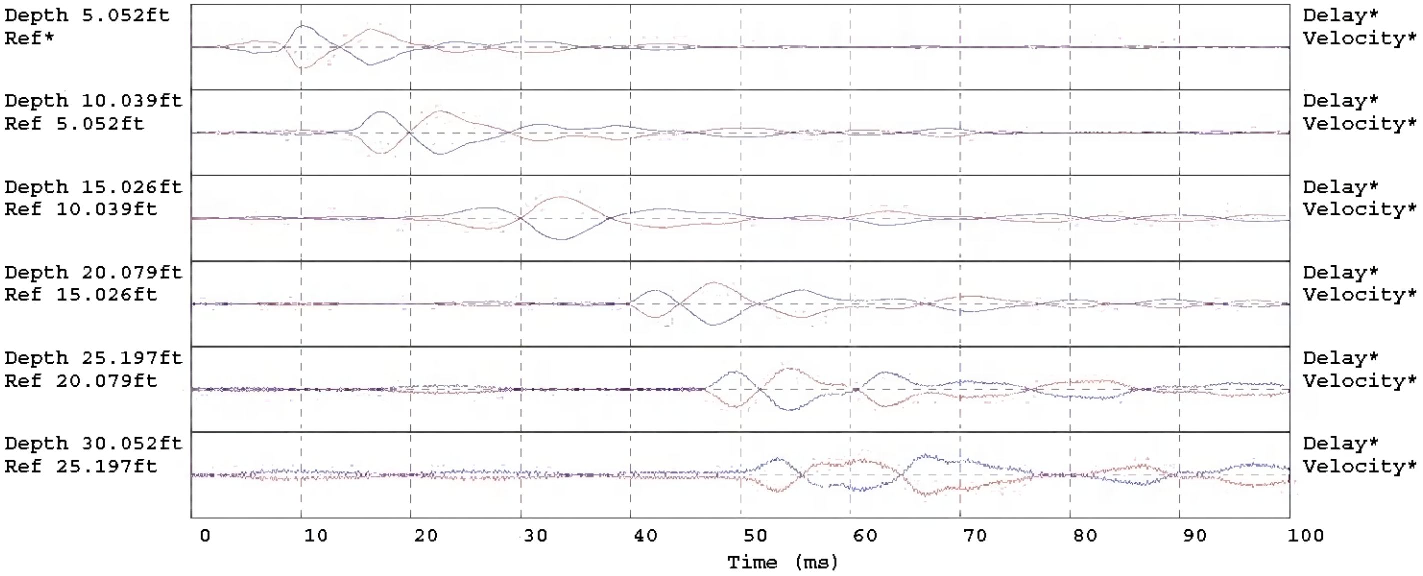
Hammer to Rod String Distance 3 (m)
* = Not Determined

Mid Atlantic Drilling
CPT-124



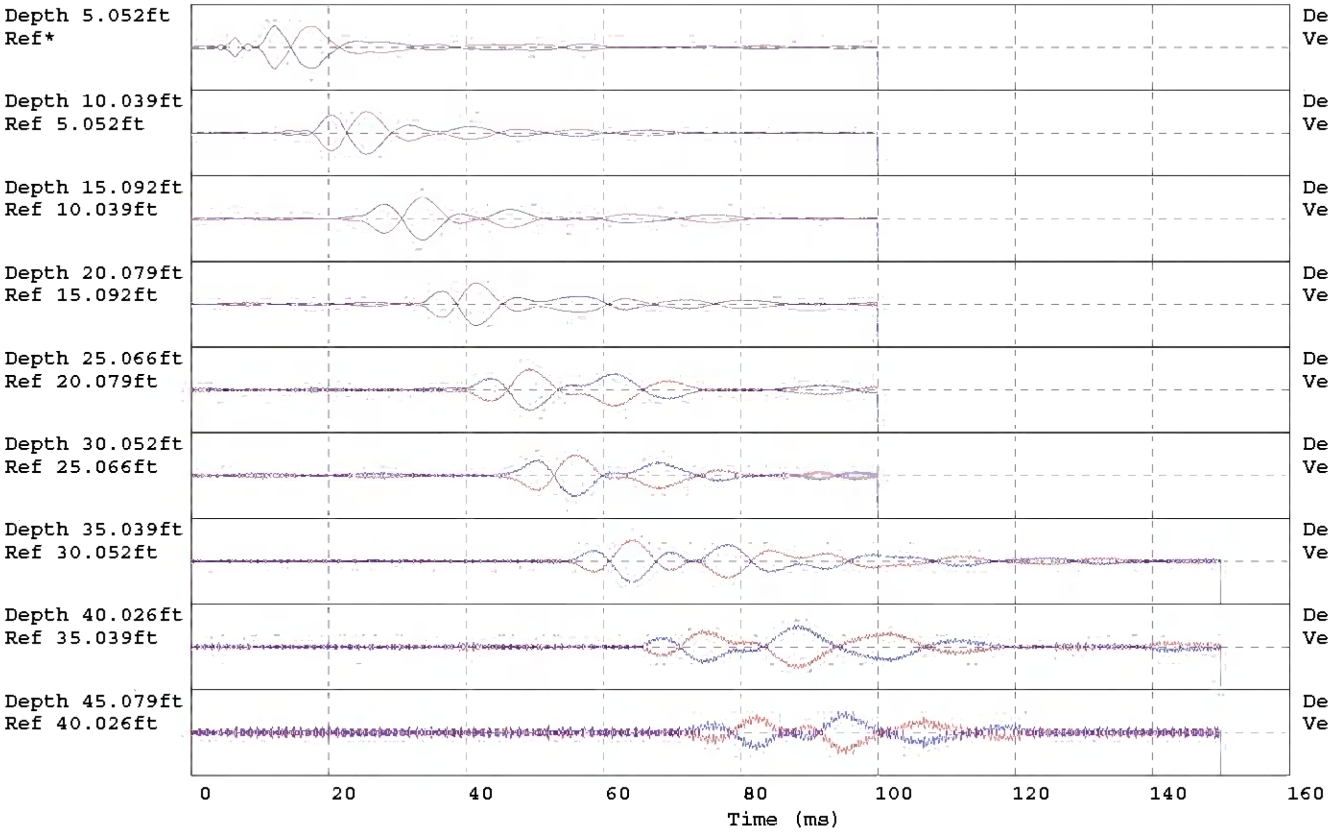
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid Atlantic Drilling
CPT-125



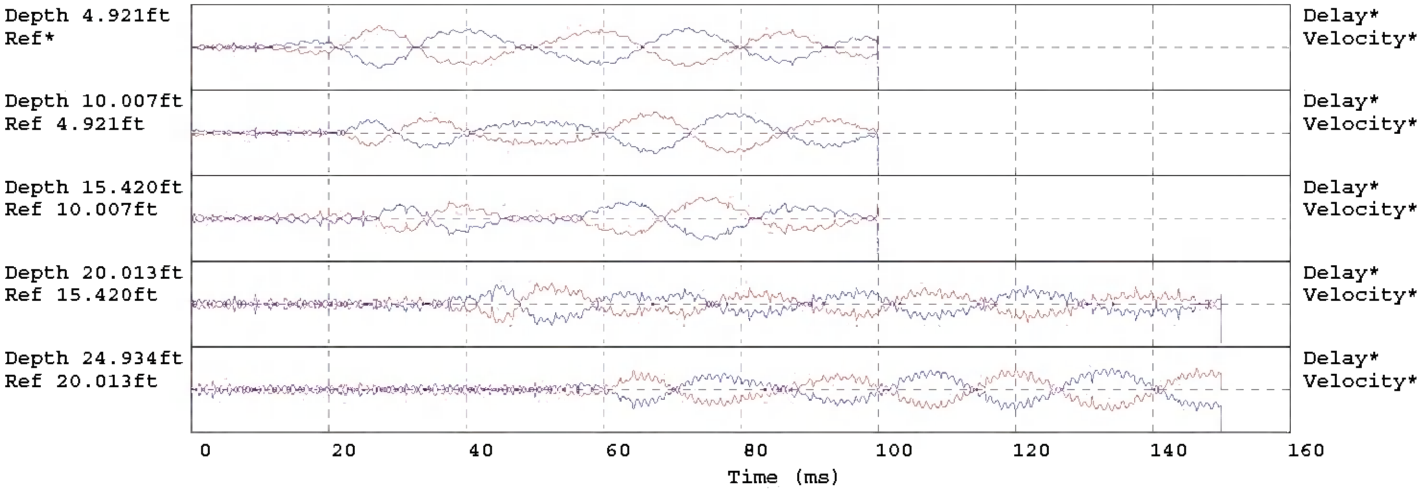
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid Atlantic Drilling
CPT-129



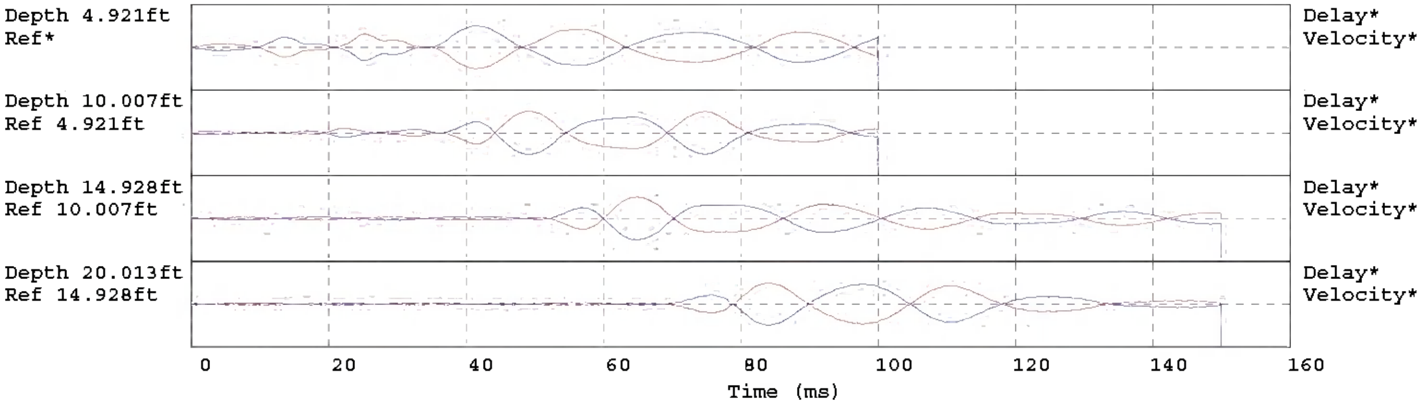
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid Atlantic Drilling
CPT-130A



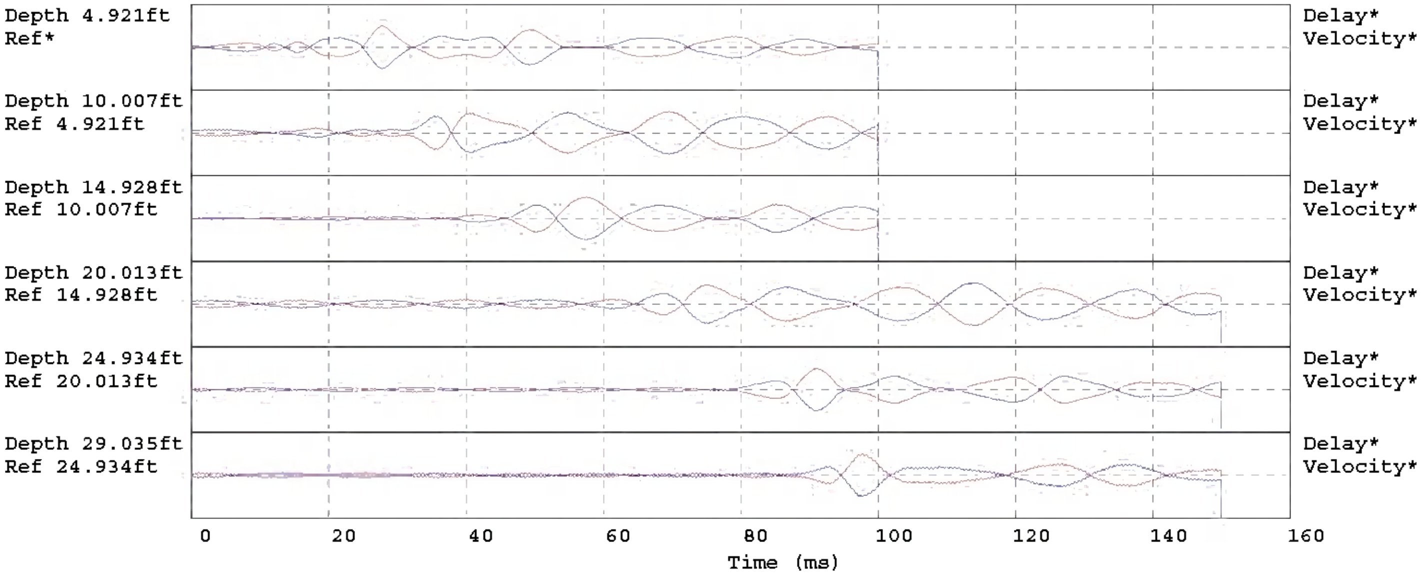
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid Atlantic Drilling
CPT-132



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

Mid Atlantic Drilling
CPT-133



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

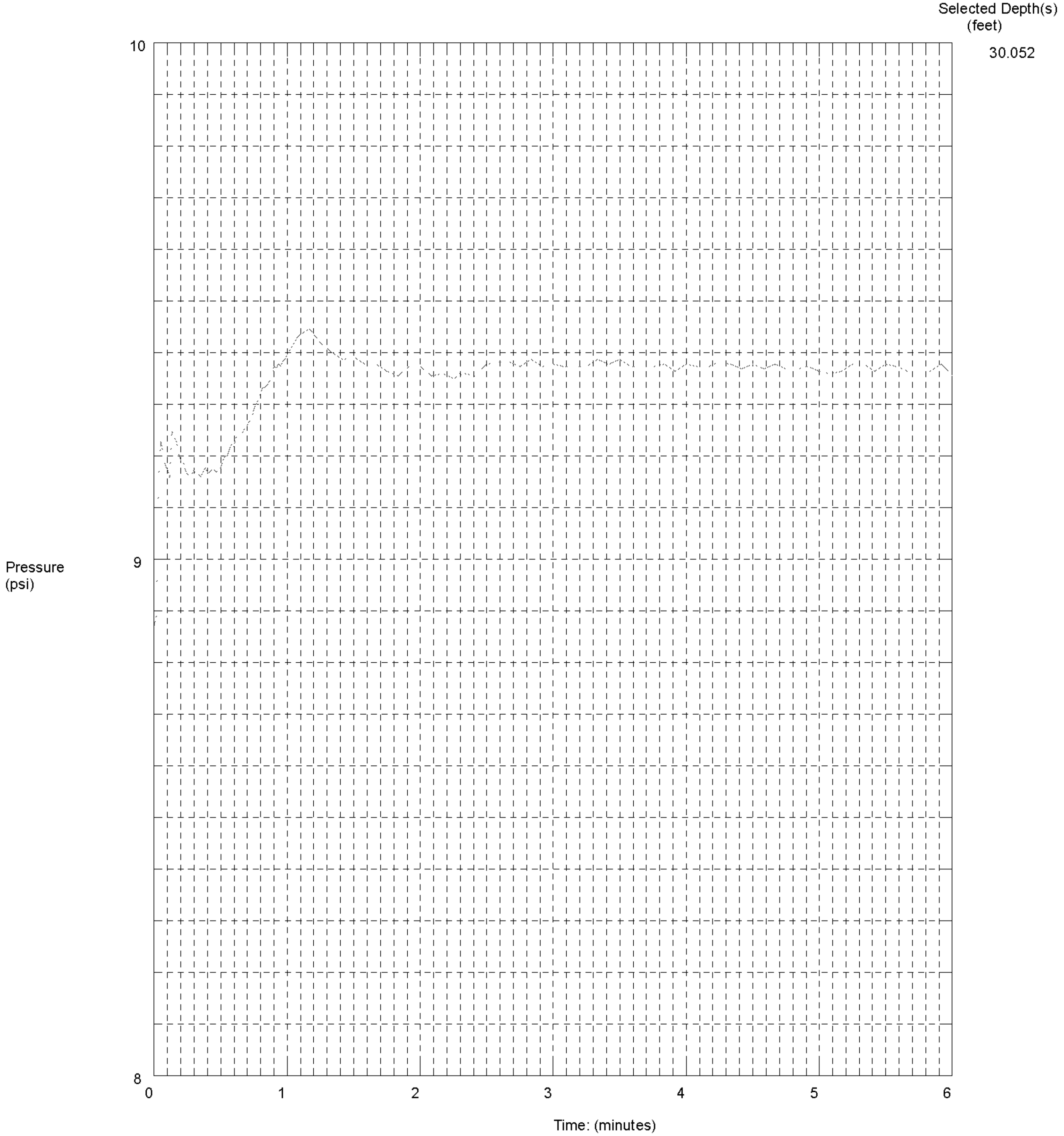
Attachment 3-C

CPT Dissipation Test Data
(provided by Mid-Atlantic Drilling, Inc.)

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-122
Cone Used: DSG0867

CPT Date/Time: 10/7/2013 4:54:35 PM
Location: Georgetown S.C.
Job Number: GSC-5242

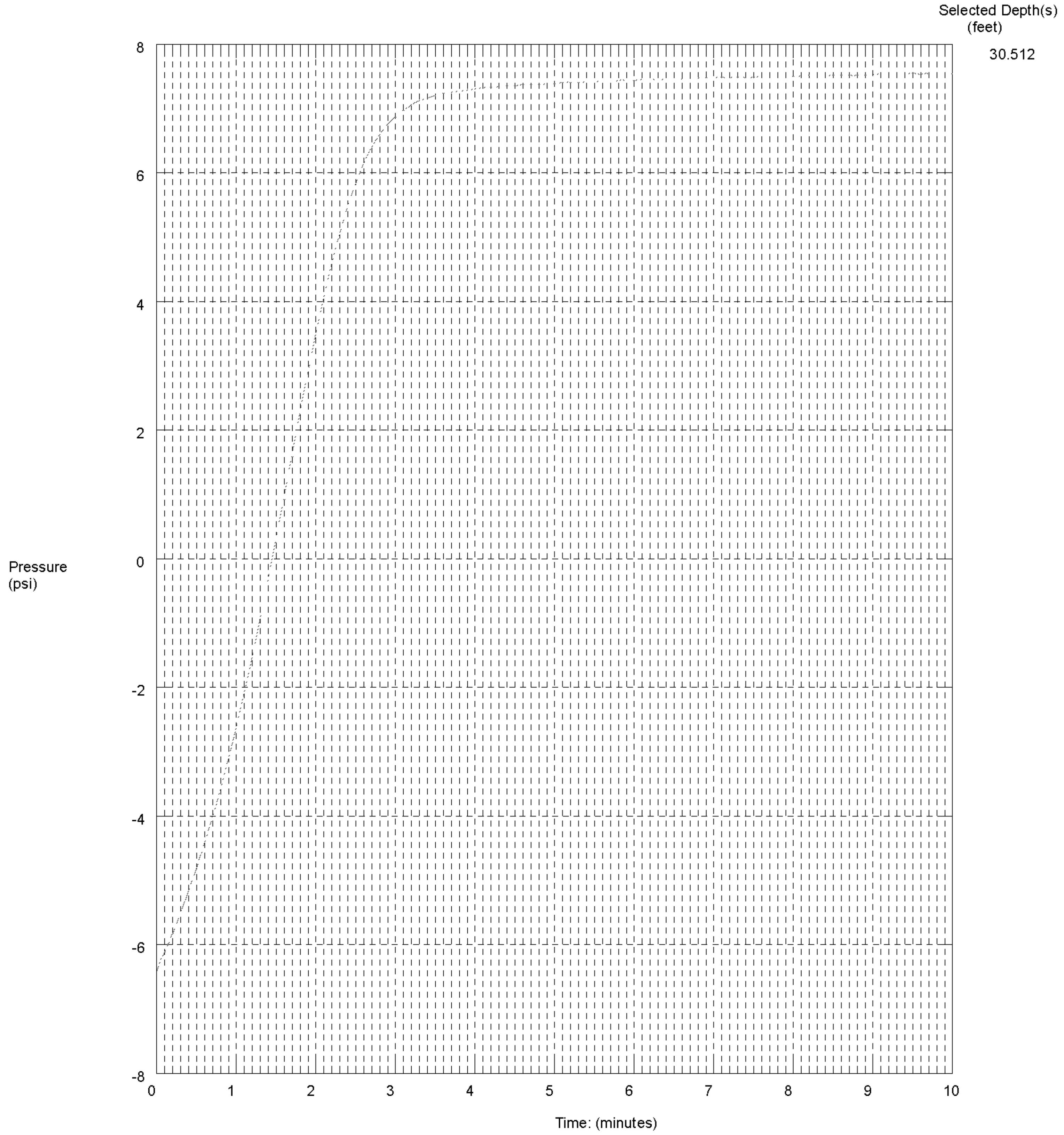


Maximum Pressure = 9.445 psi
Hydrostatic Pressure = 13.043 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-129
Cone Used: DSG0867

CPT Date/Time: 10/8/2013 8:33:59 AM
Location: Georgetown S.C.
Job Number: GSC-5242

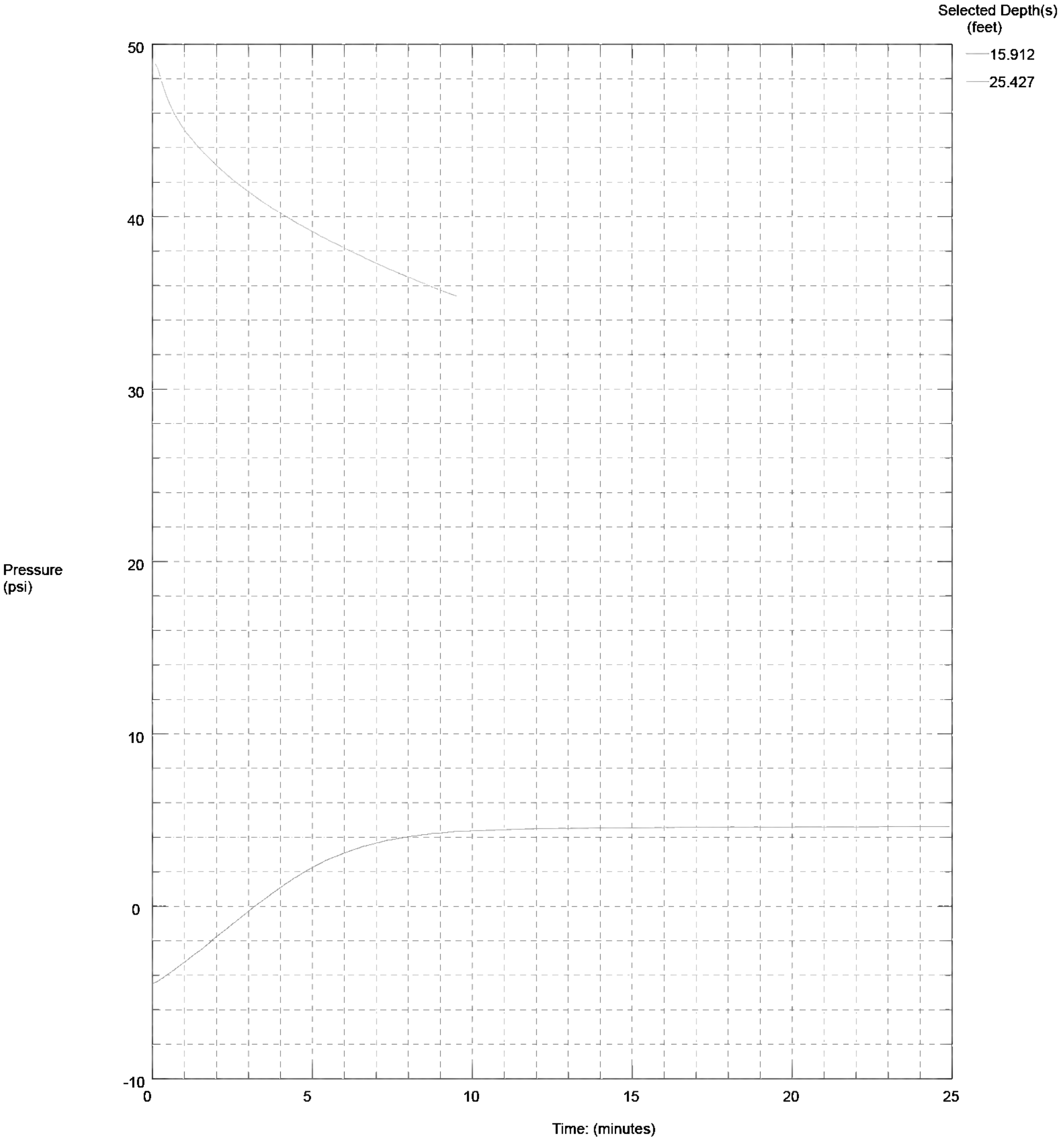


Maximum Pressure = 7.55 psi
Hydrostatic Pressure = 13.242 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-130A
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 11:25:10 AM
Location: Georgetown S.C.
Job Number: GSC-5242

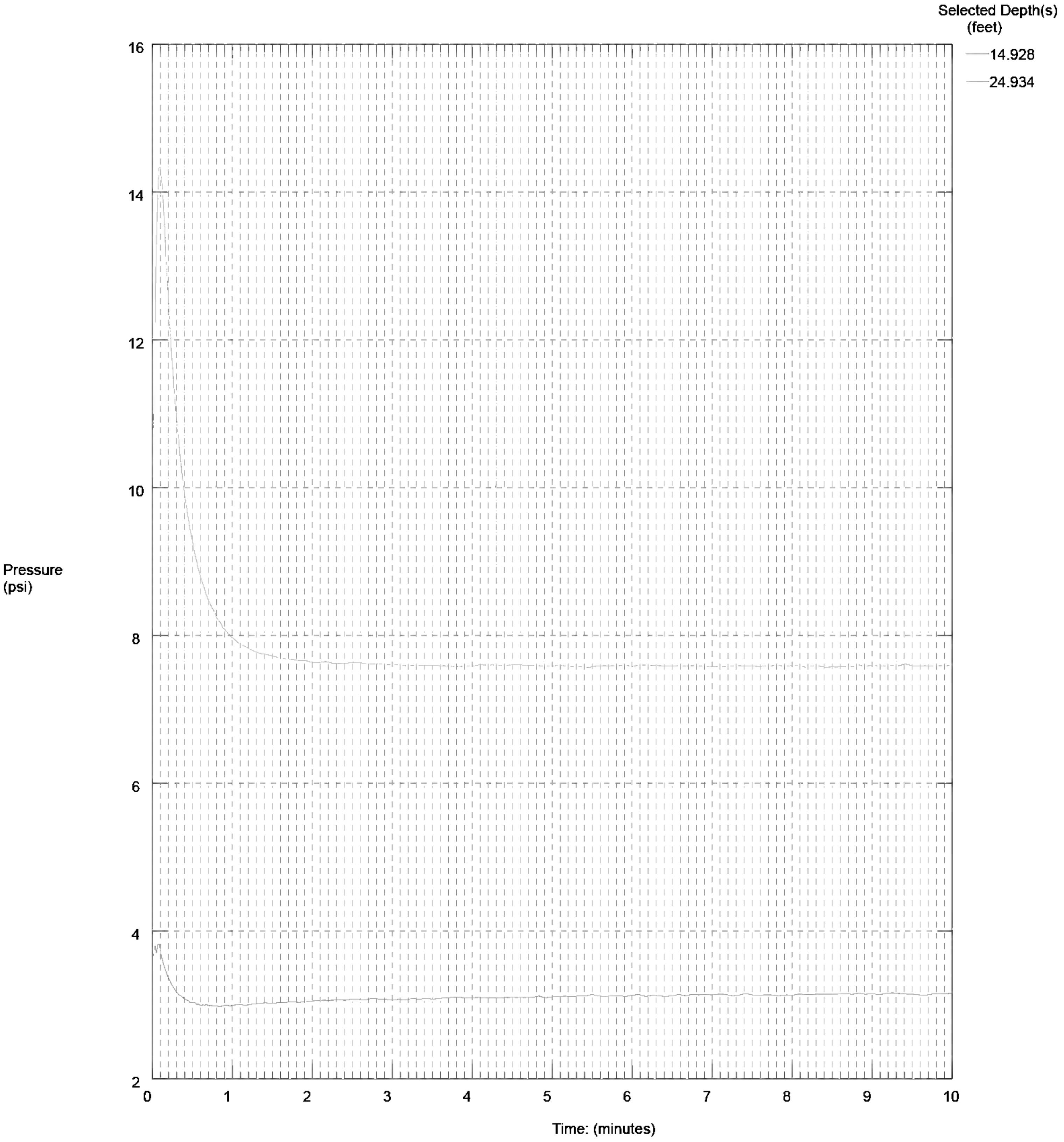


Maximum Pressure = 49.929 psi
Hydrostatic Pressure = 11.035 psi

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-131
Cone Used: DDG1195

CPT Date/Time: 9/24/2013 9:33:29 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Pressure = 14.334 psi
Hydrostatic Pressure = 10.821 psi

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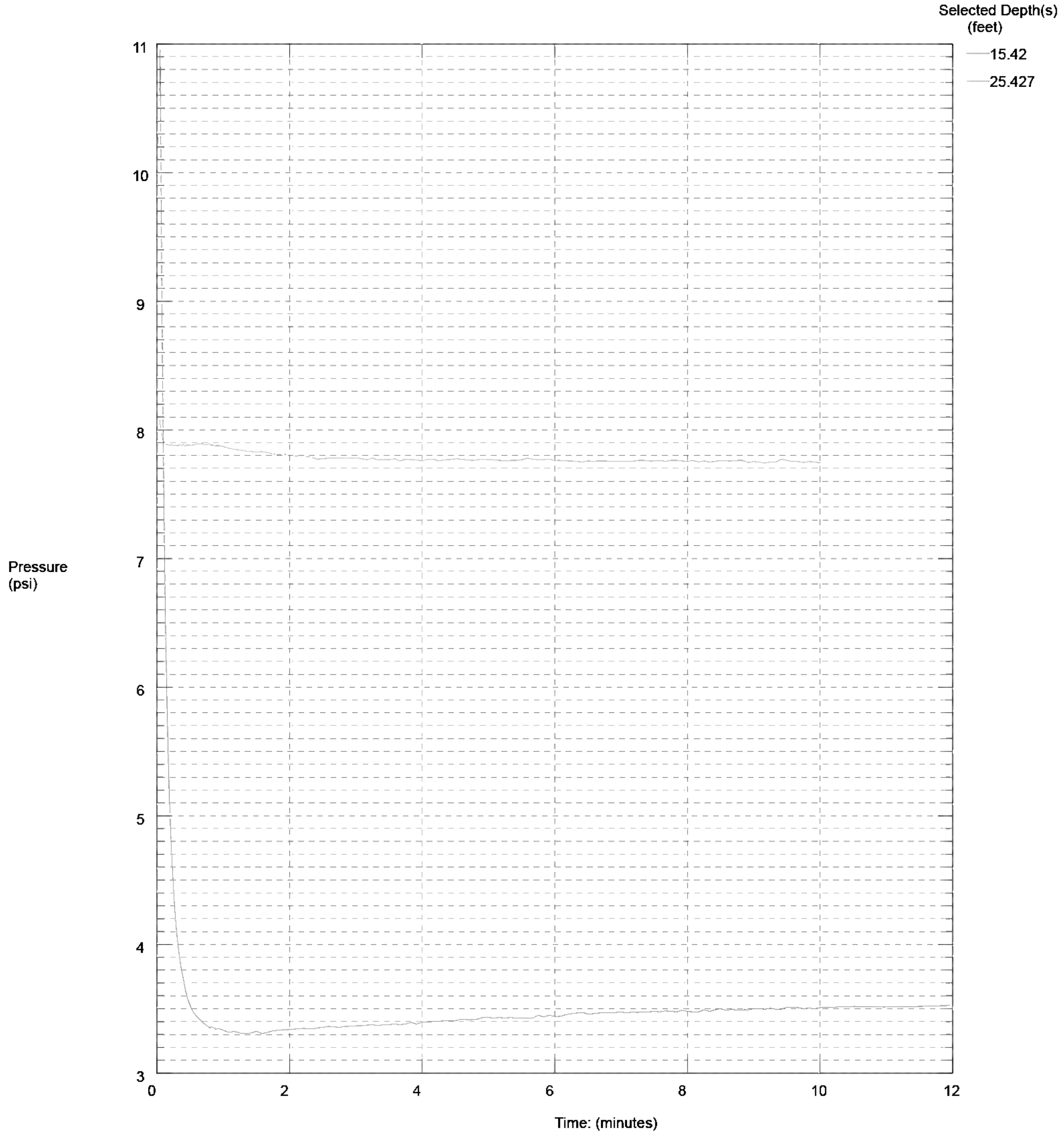
Geosyntec

..

Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-133
Cone Used: DSG1156

CPT Date/Time: 9/23/2013 4:24:30 PM
Location: Georgetown S.C.
Job Number: GSC-5242

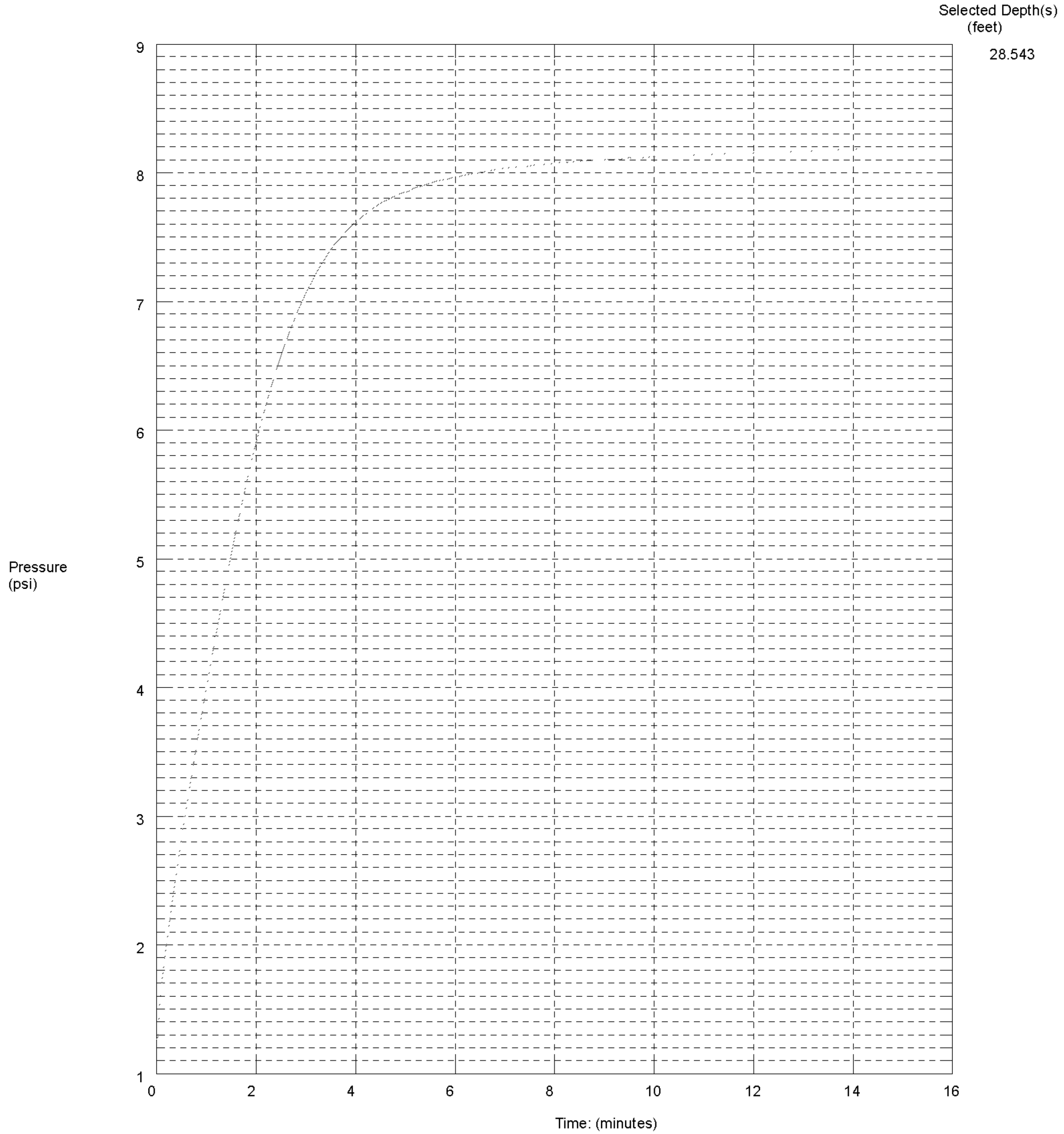


Maximum Pressure = 10.955 psi
Hydrostatic Pressure = 11.035 psi

MID-ATLANTIC DRILLING

Operator Cory Robison
Sounding: CPT-205
Cone Used: DDG1242

CPT Date/Time: 3/1/2016 2:10:28 PM
Location: Georgetown S.C.
Job Number:



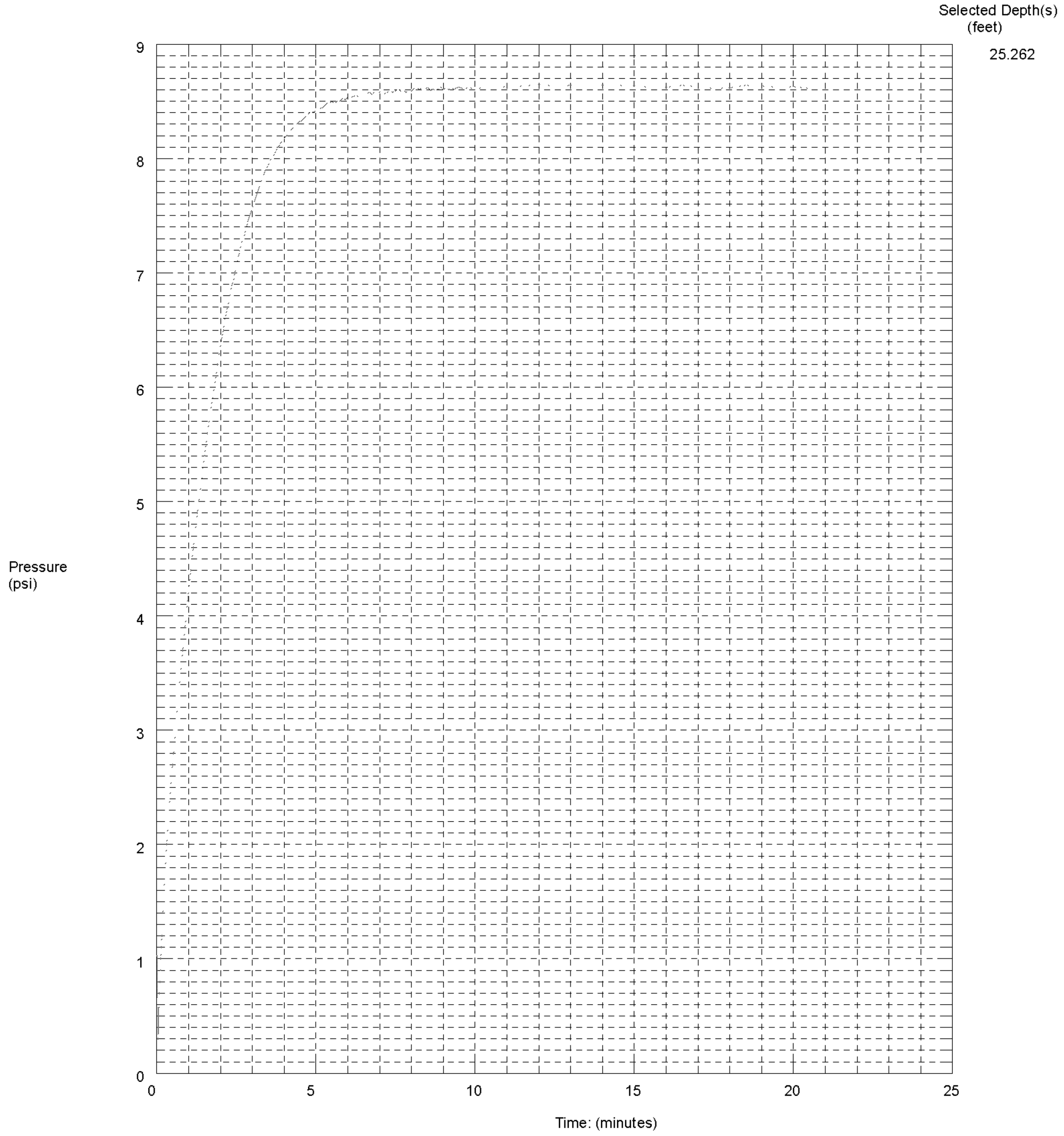
Maximum Pressure = 8.189 psi
Hydrostatic Pressure = 12.388 psi

South Ash Pond
Geosyntec

MID-ATLANTIC DRILLING

Operator Cory Robison
Sounding: CPT-207
Cone Used: DSG1156

CPT Date/Time: 3/1/2016 10:22:41 AM
Location: Georgetown S.C.
Job Number:



Maximum Pressure = 8.644 psi
Hydrostatic Pressure = 10.964 psi

South Ash Pond
Geosyntec

ATTACHMENT 4

Laboratory Testing Results

Attachment 4-A

Geosyntec Results
(Provided by Excel Geotechnical Testing)

Index Testing

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Test Results Summary

Project Name: Winyah Generating Station
South Ash Pond
Project No.: 618

Sample Information		Test Information										Remarks
		Moisture Content	Grain Size Analysis ASTM D 422					Atterberg Limits ASTM D 4318			Engineering Classification	
Site ID (-)	Lab No. (-)	ASTM D 2216 (%)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Silt Content (%)	Clay Content (%)	LL (-)	PL (-)	PI (-)	ASTM D 2487 (-)	
SPT-109, SS-01 (5.0-6.5')	13J175											
SPT-109, SS-02 (10.0-11.5')	13J176	22.4			14.5							
SPT-109, SS-03 (15.0-16.5')	13J177	19.6	0.0	78.7	21.3							
SPT-109, SS-04 (20.0-21.5')	13J178	21.2			38.1							
SPT-109, ST-01 (21.5-23.5')	13J361		0.1	62.5	37.4	12.9	24.5	48.0	19.0	29.0	SC	
SPT-109, SS-05 (25.0-26.5')	13J179											
SPT-109, ST-02 (26.6-28.5')	13J362											
SPT-109, SS-06 (30.0-31.5')	13J180											
SPT-109, SS-07 (35.0-36.5')	13J181	28.7			8.3							
SPT-109, SS-08 (40.0-41.5')	13J182	71.7	0.0	23.7	76.3	22.9	53.4	110	33	77		
SPT-109, SS-09 (45.0-46.5')	13J183	37.2			29.1							
SPT-109, SS-10 (50.0-51.5')	13J184											
SPT-110, SS-01 (5.0-6.5')	13J185											
SPT-110, SS-02 (10.0-11.5')	13J186	16.3			15.9							
SPT-110, ST-01 (12.0-14.0')	13J363											
SPT-110, SS-03 (15.0-16.5')	13J187	18.0			15.9							
SPT-110, SS-04 (20.0-21.5')	13J188											
SPT-110, SS-05 (25.0-26.5')	13J189	24.9			12.8							
SPT-110, SS-06 (30.0-31.5')	13J190											
SPT-110, SS-07 (35.0-36.5')	13J191											
SPT-110, ST-02 (36.5-38.5')	13J364		2.6	81.9	15.5	4.8	10.7	NP	NP	NP	SC	
SPT-110, SS-08 (40.0-41.5')	13J192	25.2	17.8	65.7	16.5							
SPT-110, SS-09 (45.0-46.5')	13J193	21.7										
SPT-110, SS-10 (50.0-51.5')	13J194											
SPT-110, SS-11 (55.0-56.5')	13J195	13.1	35.9	45.3	18.8							
SPT-110, SS-12 (60.0-61.5')	13J196											
SPT-110, SS-13 (65.0-66.5')	13J197	50.0	0.0	13.0	87.0	38.4	48.6					
SPT-110, ST-03 (66.5-68.5')	13J365											
SPT-111, SS-01 (5.0-6.5')	13J198	13.7	2.9	79.8	17.3	8.3	9.0					
SPT-111, SS-02 (10.0-11.5')	13J199											
SPT-111, SS-03 (15.0-16.5')	13J200											
SPT-111, SS-04 (20.0-21.5')	13J201											
SPT-111, SS-05 (25.0-26.5')	13J202	19.1	0.2	85.6	14.2							

Notes:

2-18-14
PD, NSR

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Test Results Summary

Project Name: Winyah Generating Station

South Ash Pond

Project No.: 618

Sample Information		Test Information										Remarks
Site ID (-)	Lab No. (-)	Moisture Content ASTM D 2216 (%)	Grain Size Analysis ASTM D 422					Atterberg Limits ASTM D 4318			Engineering Classification ASTM D 2487 (-)	
			Gravel Content (%)	Sand Content (%)	Fines Content (%)	Silt Content (%)	Clay Content (%)	LL (-)	PL (-)	PI (-)		
SPT-111, SS-06 (30.0-31.5')	13J203	21.7			9.5							
SPT-111, SS-07 (35.0-36.5')	13J204	34.6	2.3	74.0	23.7							
SPT-111, SS-08 (40.0-41.5')	13J205	20.8			8.1							
SPT-111, SS-09 (45.0-46.5')	13J206											
SPT-111, SS-10 (50.0-51.5')	13J207											
SPT-112, SS-01 (0.0-1.5')	13J208											
SPT-112, SS-02 (2.5-4.0')	13J209	17.6			10.9							
SPT-112, SS-03 (5.0-6.5')	13J210											
SPT-112, ST-01 (8.0-10.0')	13J366	21.1	0	78.7	21.3							
SPT-112, SS-04 (10.0-11.5')	13J211	13.6			13.3							
SPT-112, SS-05 (15.0-16.5')	13J212											
SPT-112, SS-06 (20.0-21.5')	13J213											
SPT-112, SS-07 (25.0-26.5')	13J214	29.2	0.1	91.1	8.8							
SPT-112, SS-08 (30.0-31.5')	13J215	26.8			5.4							
SPT-112, ST-02 (35.0-37.0')	NR											1
SPT-112, SS-09 (40.0-41.5')	13J216	23.8			9.7							
SPT-112, SS-10 (45.0-46.5')	13J217											
SPT-112, ST-03 (46.5-48.5')	13J367		0.3	2.3	97.4	21.5	75.9	151	55	96	MH	
SPT-112, SS-11 (50.0-51.5')	13J218											
SPT-112, SS-12 (55.0-56.5')	13J219	15.3			22.7							
SPT-112, SS-13 (60.0-61.5')	13J220	48.9	0.0	9.3	90.7	42.0	48.7					
SPT-112, ST-04 (65.0-67.0')	13J368											
SPT-113, SS-01 (5.0-6.5')	13J221											
SPT-113, SS-02 (10.0-11.5')	13J222	31.8	7.7	61.8	30.5							
SPT-113, ST-01 (12.0-14.0')	NR											1
SPT-113, SS-03 (15.0-16.5')	13J223	43.8	0.0	81.1	18.9							
SPT-113, SS-04 (20.0-21.5')	13J224											
SPT-113, SS-05 (25.0-26.5')	13J225											
SPT-113, ST-02 (26.5-28.5')	13J369	40.5	10.4	23.6	66.0	43.8	22.2	NP	NP	NP	ML	

Note:
1- No Sample

2-18-14
PD, NSR

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Test Results Summary

Project Name: Winyah Generating Station

South Ash Pond

Project No.: 618

Sample Information		Test Information									Remarks
Site ID	Lab No.	Carbonate Content ASTM D 4373	pH ASTM D 4972 (1)		Bulk Density (Dry Unit Weight) Modified ASTM D 2937			Density		Specific Gravity ASTM D 854	
			Distilled Water	Calcium Chloride	Bulk Unit Weight	Dry Unit Weight	Moisture Content	Minimum	Maximum		
(-)	(-)	(%)	(-)	(-)	(pcf)	(pcf)	(%)	(pcf)	(pcf)	(-)	
SPT-109, ST-01 (21.5-23.5')	13J361				133.6	116.5	14.7				
					127.5	106.8	19.4				
SPT-110, ST-02 (36.5-38.5')	13J364				117.7	86.5	36.1				
					123.3	96.9	27.2				
					126.2	108.6	16.2				
SPT-112, ST-03 (46.5-48.8')	13J367				86.5	41.8	107.0				
SPT-113, ST-02 (26.5-28.8')	13J369	Note 2	5.7	5.8							

Notes:
1 - Method B was used which may not be very accurate..
2 - Not enough material available to satisfy ASTM Standard D 4373 requirement for testing.

2-9-14
NSR

Excel Geotechnical Testing, Inc.
"Excellence In Testing"

953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Test Results Summary

Project Name: Winyah Generating Station
Project No.: 771

Sample Information		Test Information												Remarks
Site ID	Lab No.	Moisture Content ASTM D 2216	Grain Size Analysis ASTM D 422					Atterberg Limits ASTM D 4318			Dry Unit Weight$^{(1)}$ Modified ASTM D 2937		Engin. Classifi. ASTM D 2487	
			Gravel Content	Sand Content	Fines Content	Silt Content	Clay Content	LL	PL	PI	Moisture Content	Dry Unit Weight		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(%)	(pcf)	(-)	
SPT-302-S-1 (0-2')	16C014													
SPT-302-S-2 (2-4')	16C015													
SPT-302-S-3 (4-6')	16C016	27.9			6.9									
SPT-302-S-4 (6-8')	16C017													
SPT-302-S-5 (8-10')	16C018													
SPT-302-S-6 (10-12')	16C019													
SPT-302-S-7 (12-14')	16C020	25.1			34.4									
SPT-302-S-8 (14-16')	16C021	22.1			14.9									
SPT-302-S-9 (16-18')	16C022	28.4	0.5	68.2	31.3									
SPT-302-S-10 (18-20')	16C023													
SPT-302-S-11 (23-25')	16C024		1.3	76.0	22.7			39.0	18.0	21			SC	See TX
SPT-302-S-12 (28-30')	16C025													
SPT-302-S-13 (33-35')	16C026	22.9		93.8	6.2									
SPT-302-S-14 (39-41')	16C027													
SPT-302-S-15 (43.5-45')	16C028													
SPT-302-S-16 (48.5-50')	16C029													
SPT-302-S-17 (53.5-55')	16C030													
SPT-303-S-1 (0-2')	16C031													
SPT-303-S-2 (2-4')	16C032													
SPT-303-S-3 (4-6')	16C033													
SPT-303-S-4 (6-8')	16C034													
SPT-303-S-5 (8-10')	16C035	17.1			11.5									
SPT-303-S-6 (10-12')	16C036													
SPT-303-S-7 (12-14')	16C037													
SPT-303-S-8 (14-16')	16C038													
SPT-303-S-9 (16-18')	16C039													
SPT-303-S-10 (18-20')	16C040													
SPT-303-S-11 (23.5-25')	16C041													
SPT-303-S-12 (28.5-30')	16C042	25.0			38.6									
SPT-303-S-13 (33.5-35')	16C043													
SPT-303-S-14 (38.5-40')	16C044													
SPT-303-S-15 (40-42')	16C045													
SPT-303-S-16 (42-43.5')	16C046													
SPT-303-S-17 (44-45.5')	16C047													

Notes:

11-14-16
APK, NSR

Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Test Results Summary

Project Name: Winyah Generating Station
Project No.: 771

Sample Information		Test Information												Remarks
Site ID	Lab No.	Moisture Content ASTM D 2216	Grain Size Analysis ASTM D 422					Atterberg Limits ASTM D 4318			Dry Unit Weight$^{(1)}$ Modified ASTM D 2937		Engin. Classifi. ASTM D 2487	
			Gravel Content	Sand Content	Fines Content	Silt Content	Clay Content	LL	PL	PI	Moisture Content	Dry Unit Weight		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(%)	(pcf)	(-)	
SPT-303-S-18 (48-50')	16C048													
SPT-303-S-19 (50-51')	16C049	32.8			11.7									
SPT-303-S-20 (53.5-55')	16C050													
SPT-303A-S-13 (33-35')	16C051													
SPT-303A-S-14 (39-41')	16C052													
SPT-303A-S-15 (43-45')	16C053			2.6	97.4	30.6	66.8	108.0	38.0	70			CH	See TX
SPT-303A-S-16 (48-50')	16C054													
SPT-309-S-1 (0-2')	16C055													
SPT-309-S-2 (2-4')	16C056													
SPT-309-S-3 (4-6')	16C057													
SPT-309-S-4 (6-8')	16C058													
SPT-309-S-5 (8-10')	16C059	17.6		78.5	21.5									
SPT-309-S-6 (10-12')	16C060													
SPT-309-S-7 (12-14')	16C061													
SPT-309-S-8 (14-16')	16C062													
SPT-309-S-9 (16-18')	16C063	22.0			9.6									
SPT-309-S-10 (18-20')	16C064													
SPT-309-S-11 (23.5-25')	16C065													
SPT-309-S-12 (28.5-30')	16C066	31.6			7.3									
SPT-309-S-13 (33.5-35')	16C067	30.7	0.1	88.0	11.9									
SPT-309-S-14 (38.5-40')	16C068													
SPT-309-S-15 (43.5-45')	16C069	27.5			11.1									
SPT-309-S-16 (48.5-50')	16C070													
SPT-309-S-17 (53.5-55')	16C071	29.8			29.5									
SPT-309-S-18 (58.5-60')	16C072													
SPT-310-S-1 (0-2')	16C073													
SPT-310-S-2 (2-4')	16C074													
SPT-310-S-3 (4-6')	16C075													
SPT-310-S-4 (6-8')	16C076													
SPT-310-S-5 (8-10')	16C077													
SPT-310-S-6 (10-12')	16C078	23.5			22.8									
SPT-310-S-7 (12-14')	16C079													
SPT-310-S-8 (14-16')	15C080													
SPT-310-S-9 (16-18')	15C081	21.7			10.5									

Notes:

4-14-16
APK, NSR

Excel Geotechnical Testing, Inc.

"Excellence in Testing"

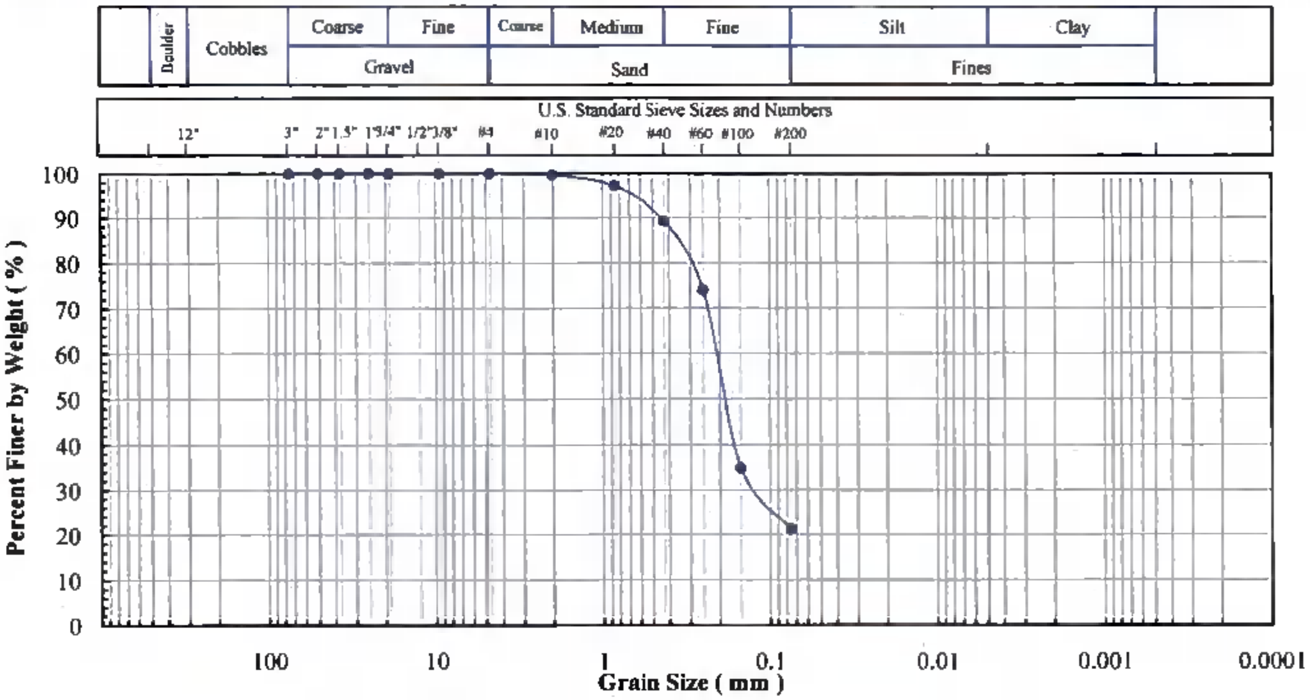
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-109, SS-03 (15.0-16.5')
Lab Sample No: 13J177

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content, Eng. Classification, Atterberg Limits



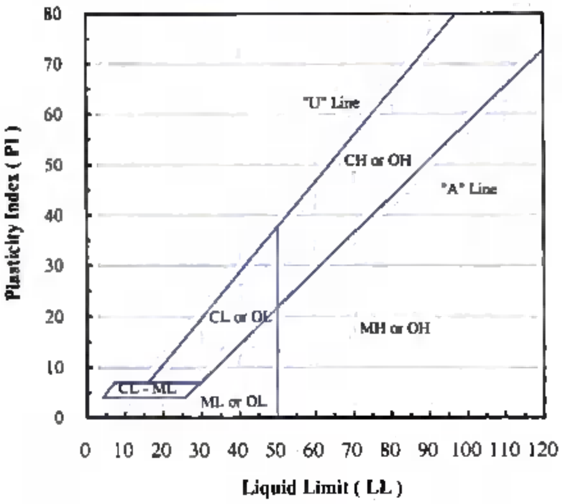
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.7
#20	0.850	97.2
#40	0.425	89.2
#60	0.250	74.0
#100	0.150	34.9
#200	0.075	21.3

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	78.7
Fines (%):	21.3
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-109, SS-03 (15.0-16	13J177	19.6	21.3				

Note(s):

11-06-13
TK, NSR

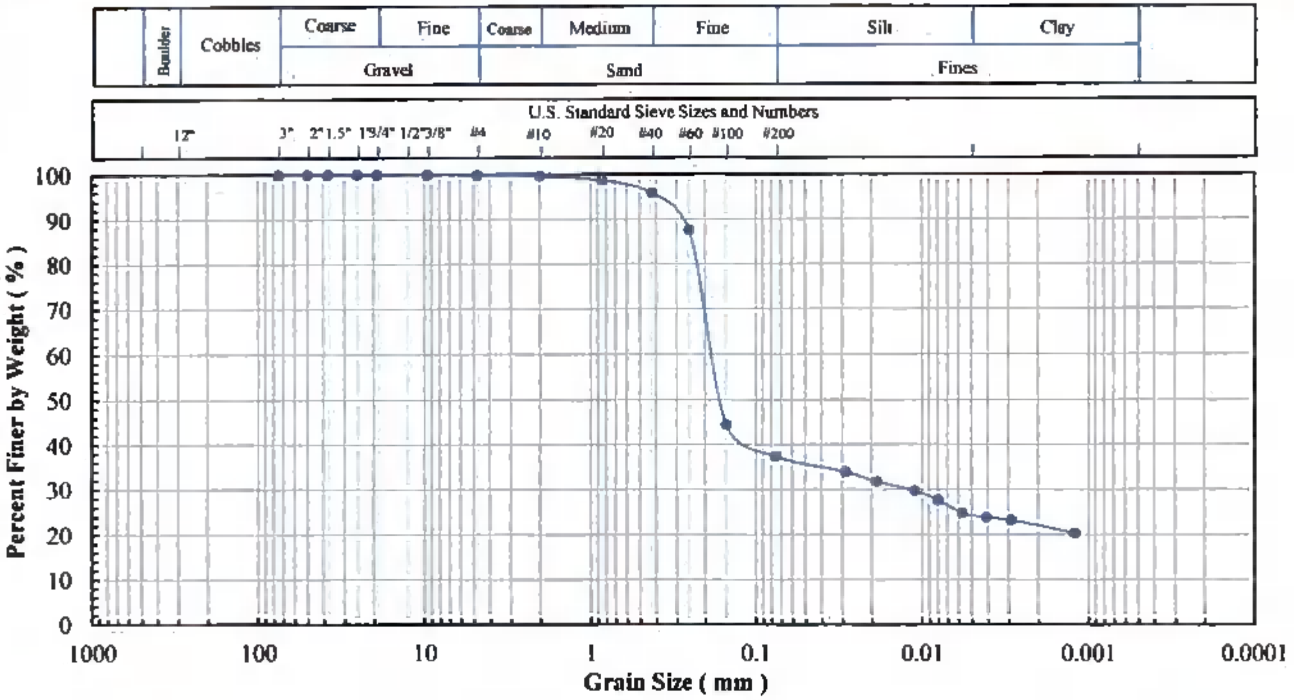
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-109, ST-01 (21.0-23.5')
Lab Sample No:	13J361

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.,
Eng. Classification, Atterberg Limits



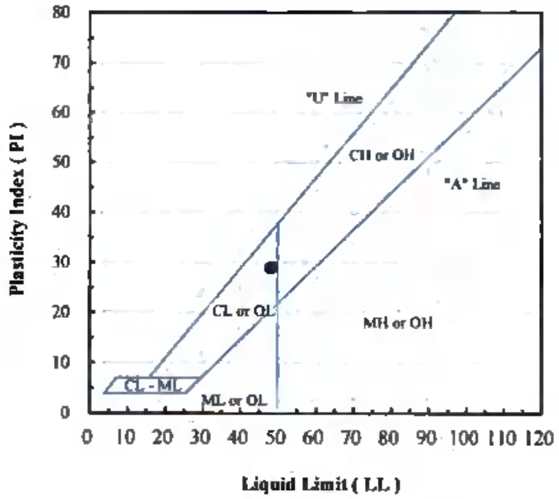
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.9
#10	2.00	99.7
#20	0.850	98.8
#40	0.425	95.9
#60	0.250	87.7
#100	0.150	44.5
#200	0.075	37.4

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0288	33.9
0.0110	29.7
0.0057	24.9
0.0029	23.3
0.0012	20.4

Gravel (%):	0.1
Sand (%):	62.5
Fines (%):	37.4
Silt (%):	12.9
Clay (%):	24.5

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-109, ST-01 (21.0-23.5')	13J361		37.4	48	19	29	SC - Clayey sand

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.
Engineering classification is based on the assumption that the fines are either CL or CH.

12-08-13
NJR

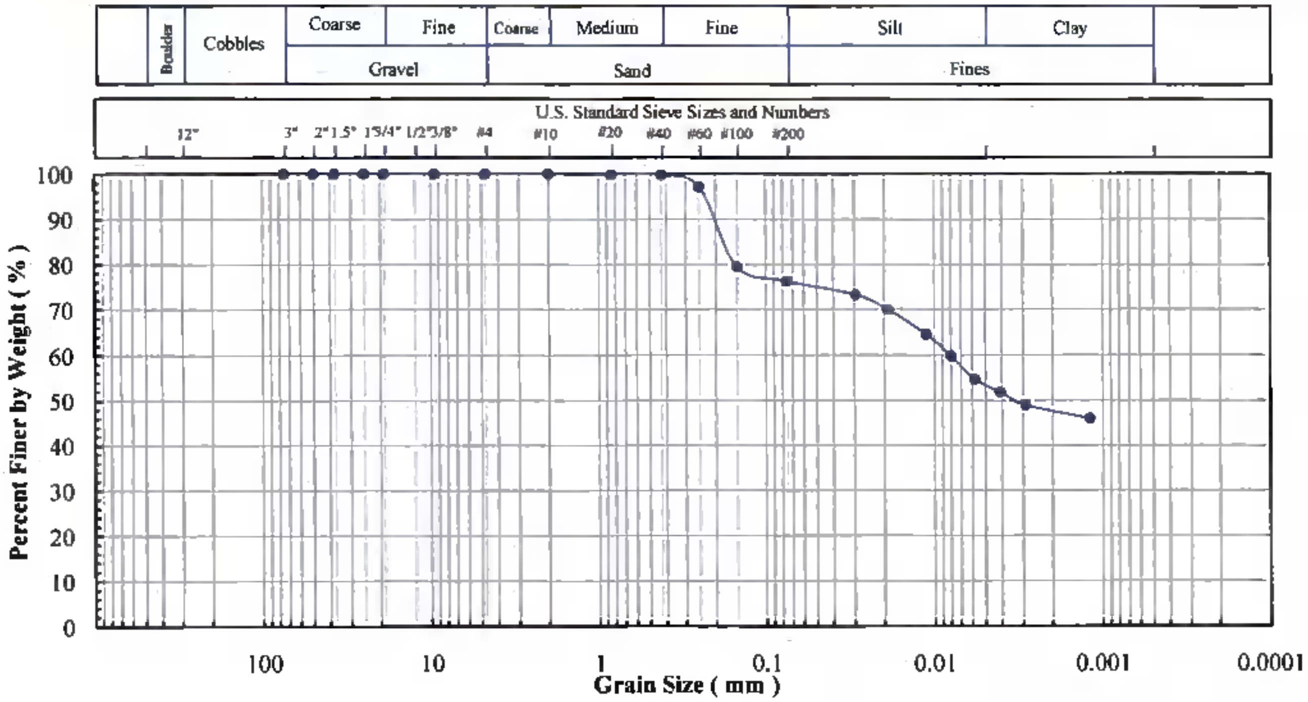
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-109, SS-08 (40.0-41.5')
Lab Sample No: 13J182

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.,
Eng. Classification, Atterberg Limits



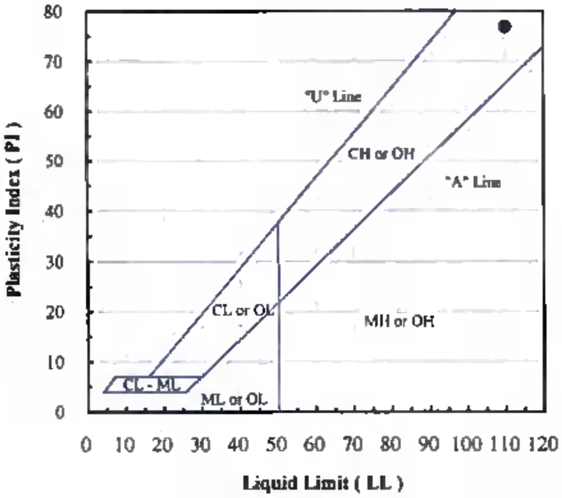
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.7
#60	0.250	97.1
#100	0.150	79.5
#200	0.075	76.3

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0295	73.2
0.0112	64.5
0.0058	54.8
0.0029	49.0
0.0012	46.1

Gravel (%):	
Sand (%):	23.7
Fines (%):	76.3
Silt (%):	22.9
Clay (%):	53.4

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
-109, SS-08 (40.0-4	13J182	71.7	76.3	110	33	77	CH - Fat clay with sand

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.

11-06-13
TR, NSR

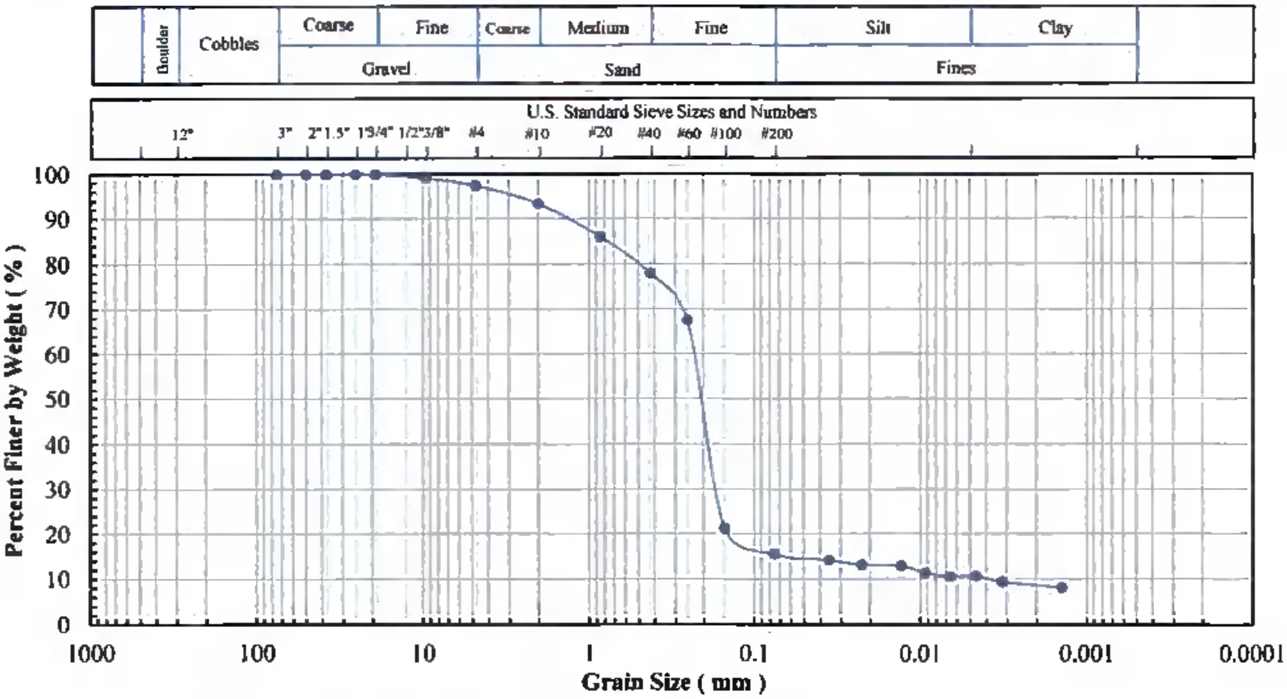
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-110, ST-02 (36.5-38.5')
Lab Sample No:	13J364

ASTM C 136, D 422, D 854, D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont., Eng. Classification, Atterberg Limits



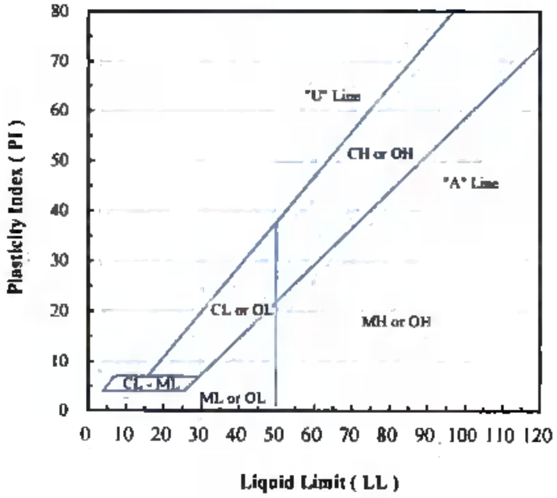
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	99.2
#4	4.75	97.4
#10	2.00	93.4
#20	0.850	86.0
#40	0.425	78.0
#60	0.250	67.6
#100	0.150	21.3
#200	0.075	15.5

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0354	14.2
0.0130	12.9
0.0066	10.5
0.0032	9.4
0.0014	8.1

Gravel (%):	2.6
Sand (%):	81.9
Fines (%):	15.5
Silt (%):	4.8
Clay (%):	10.7

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-110, ST-02 (36.5-38.5')	13J364		15.5	NP	NP	NP	SC - Clayey sand

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.
Engineering classification is based on the assumption that the fines are either CL or CH.

12-08-13
NSR

Excel Geotechnical Testing, Inc.

"Excellence in Testing"

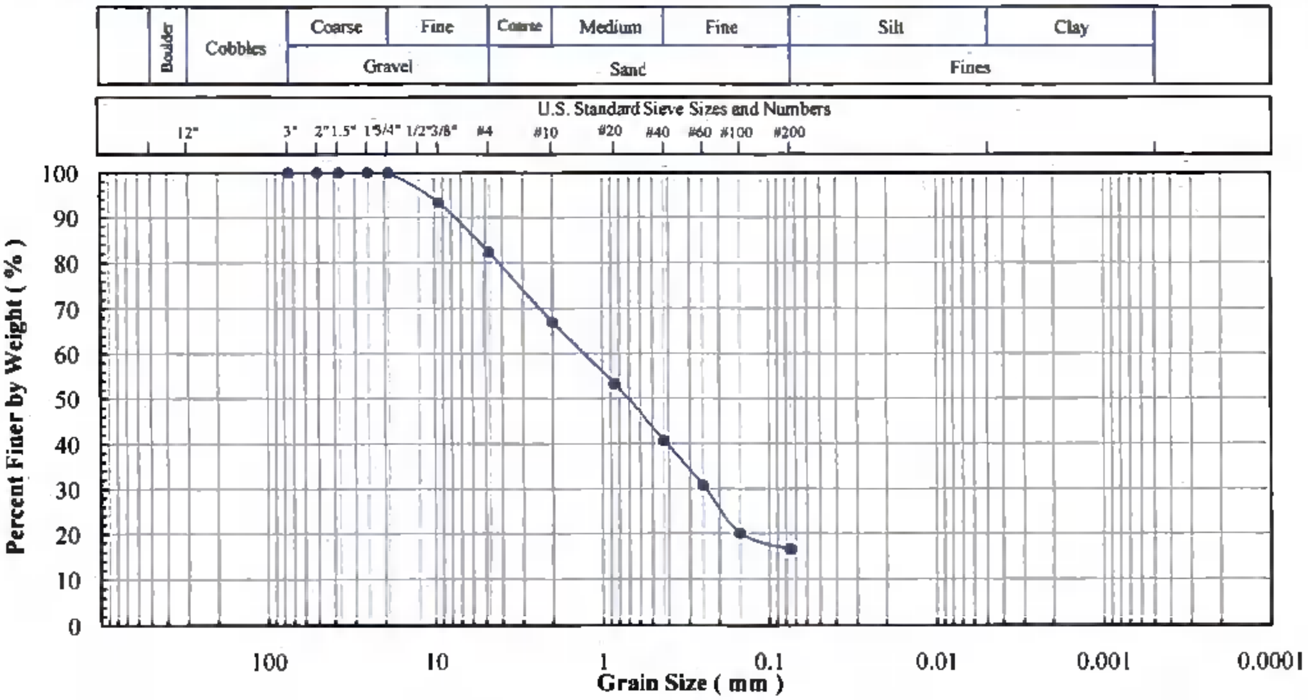
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-110, SS-08 (40.0-41.5')
Lab Sample No:	13J192

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



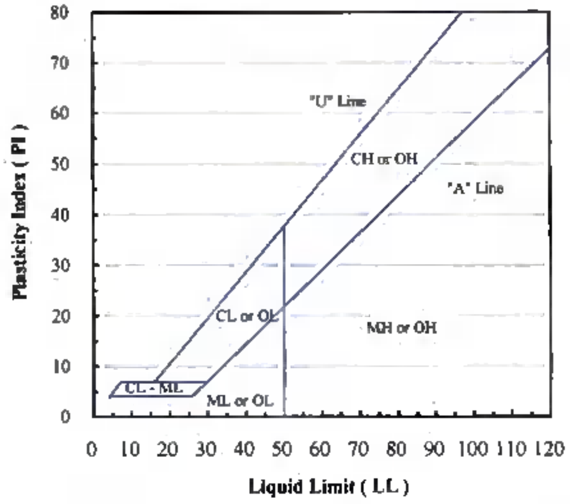
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	93.3
#4	4.75	82.2
#10	2.00	67.1
#20	0.850	53.2
#40	0.425	40.7
#60	0.250	30.8
#100	0.150	20.1
#200	0.075	16.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	17.8
Sand (%):	65.7
Fines (%):	16.5
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-110, SS-08 (40.0-41	13J192	25.2	16.5				

Note(s):

11-06-13
TR, NSR

Excel Geotechnical Testing, Inc.

"Excellence In Testing"

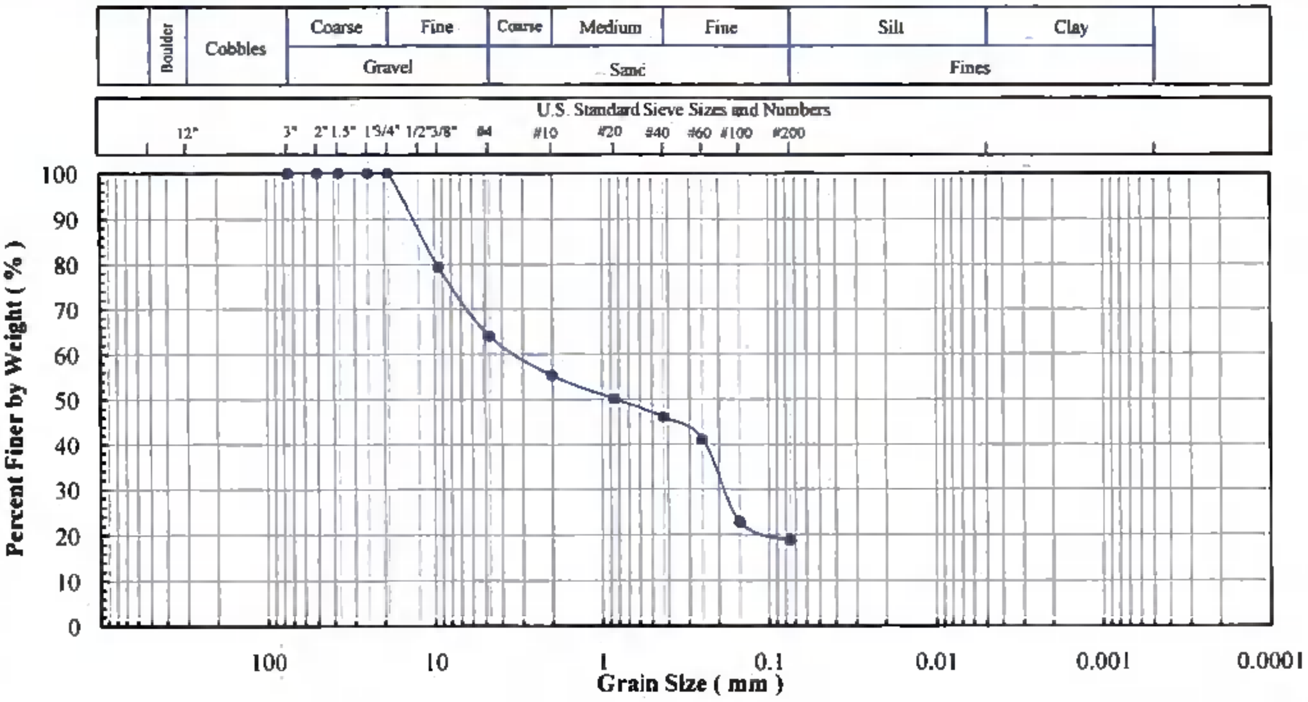
953 Forrest Street, Roswell, Georgia 30076
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, SS-11 (55.0-56.5')
Lab Sample No: 13J195

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



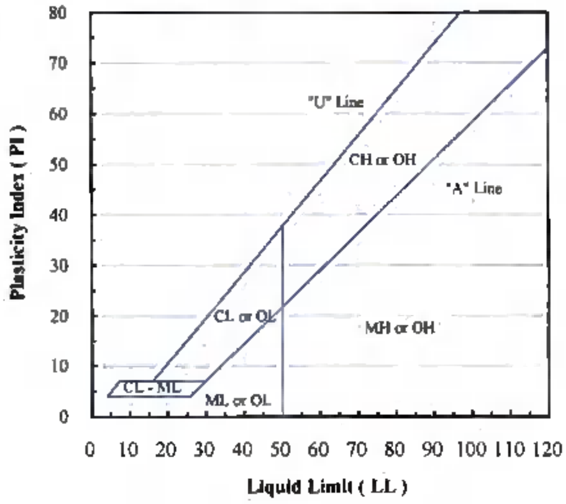
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	79.3
#4	4.75	64.1
#10	2.00	55.3
#20	0.850	50.1
#40	0.425	46.1
#60	0.250	41.2
#100	0.150	22.8
#200	0.075	18.8

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	35.9
Sand (%):	45.3
Fines (%):	18.8
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-110, SS-11 (55.0-56	13J195	13.1	18.8				

Note(s):

11-06-13
TR, NSR

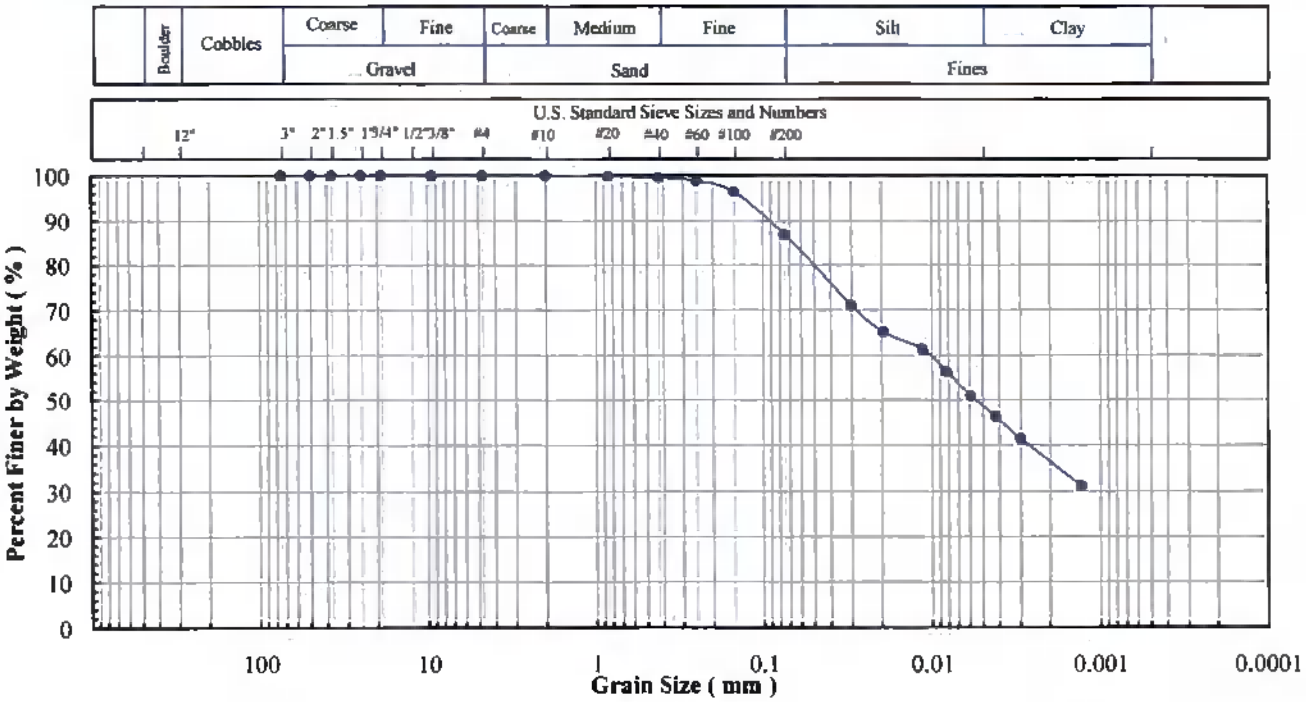
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, SS-13 (65.0-66.5')
Lab Sample No: 13J197

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.,
Eng. Classification, Atterberg Limits



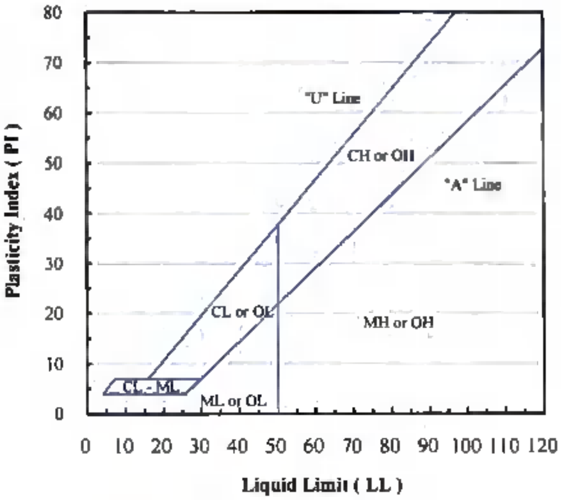
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.5
#60	0.250	98.9
#100	0.150	96.4
#200	0.075	87.0

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0302	71.0
0.0114	61.4
0.0059	51.0
0.0030	41.4
0.0013	31.1

Gravel (%):	
Sand (%):	13.0
Fines (%):	87.0
Silt (%):	38.4
Clay (%):	48.6

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
-110, SS-13 (65.0-6	13J197	50.0	87.0				

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.

11-06-13
TR, NSR

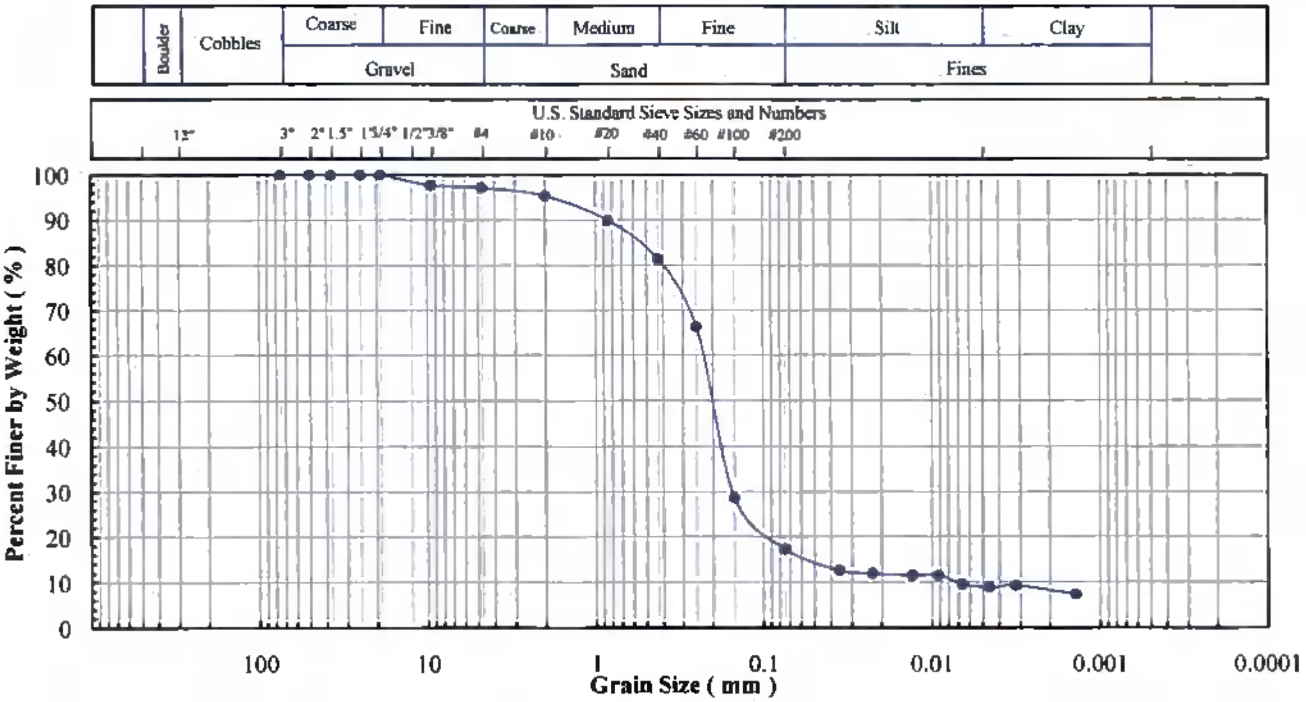
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-111, SS-01 (5.0-6.5')
Lab Sample No:	13J198

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.,
Eng. Classification, Atterberg Limits



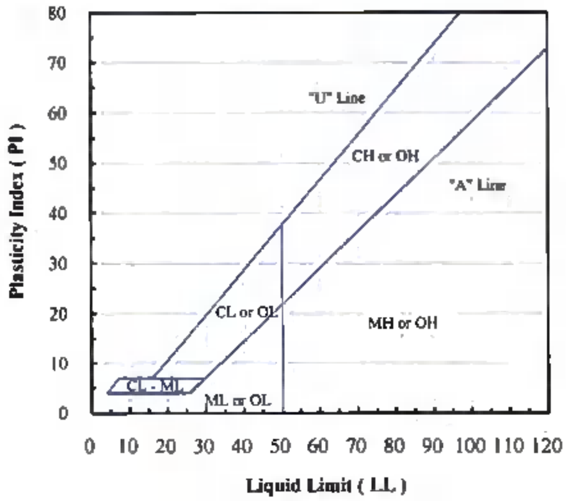
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	97.7
#4	4.75	97.1
#10	2.00	95.3
#20	0.850	89.9
#40	0.425	81.3
#60	0.250	66.3
#100	0.150	28.6
#200	0.075	17.3

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0357	12.6
0.0131	11.5
0.0066	9.6
0.0032	9.3
0.0014	7.4

Gravel (%):	2.9
Sand (%):	79.8
Fines (%):	17.3
Silt (%):	8.3
Clay (%):	9.0

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-111, SS-01 (5.0-6	13J198	13.7	17.3				

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.

11-06-13
TR, NJA

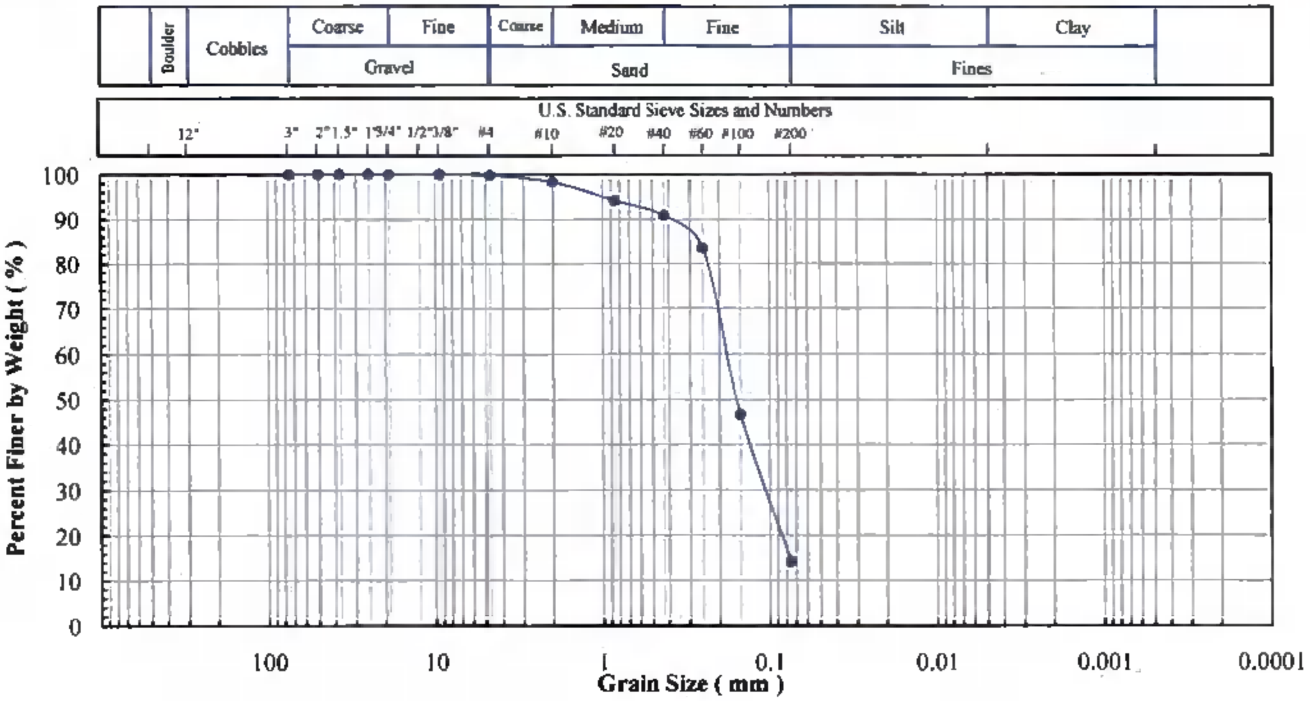
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7637 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-111, SS-05 (25.0-26.5')
Lab Sample No:	13J202

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



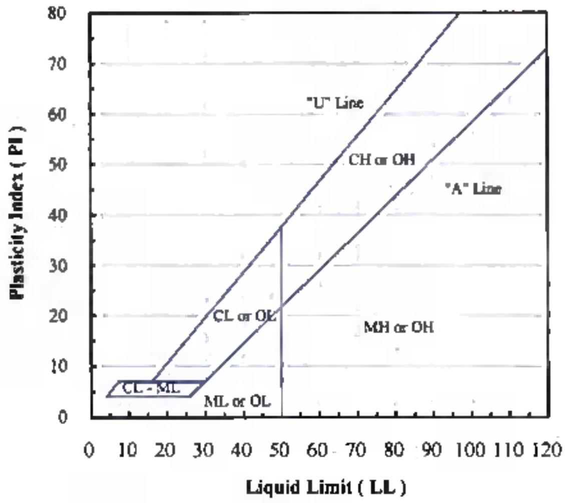
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.8
#10	2.00	98.4
#20	0.850	94.2
#40	0.425	90.9
#60	0.250	83.5
#100	0.150	46.7
#200	0.075	14.2

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	0.2
Sand (%):	85.6
Fines (%):	14.2
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-111, SS-05 (25.0-26	13J202	19.1	14.2				

Note(s):

11-06-13
TR, NSR

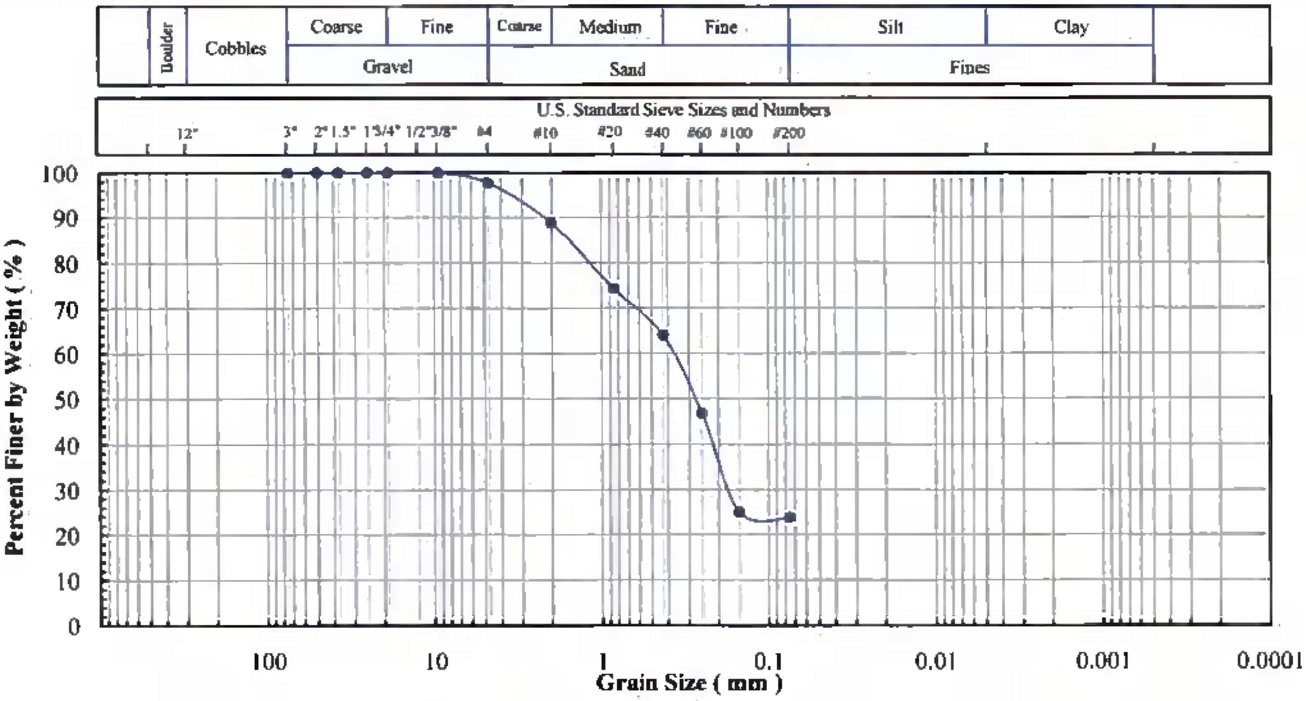
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-111, SS-07 (35.0-36.5')
Lab Sample No:	13J204

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content, Eng. Classification, Atterberg Limits



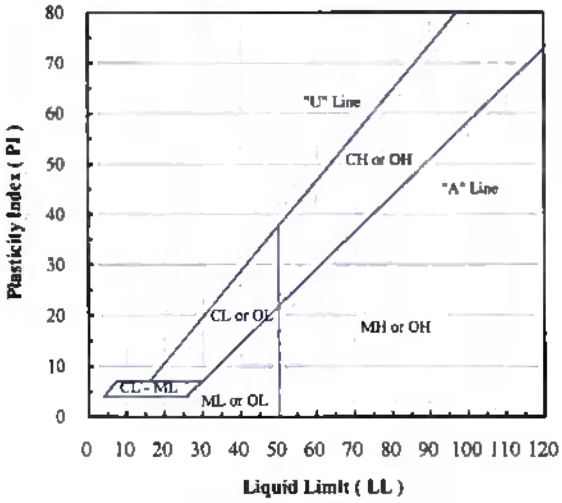
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	97.7
#10	2.00	88.7
#20	0.850	74.3
#40	0.425	64.0
#60	0.250	46.7
#100	0.150	25.0
#200	0.075	23.7

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	2.3
Sand (%):	74.0
Fines (%):	23.7
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	Atterberg Limits PL (-)	Atterberg Limits PI (-)	Engineering Classification
T-111, SS-07 (35.0-36	13J204	34.6	23.7				

Note(s):

11-06-13
TR, NSR

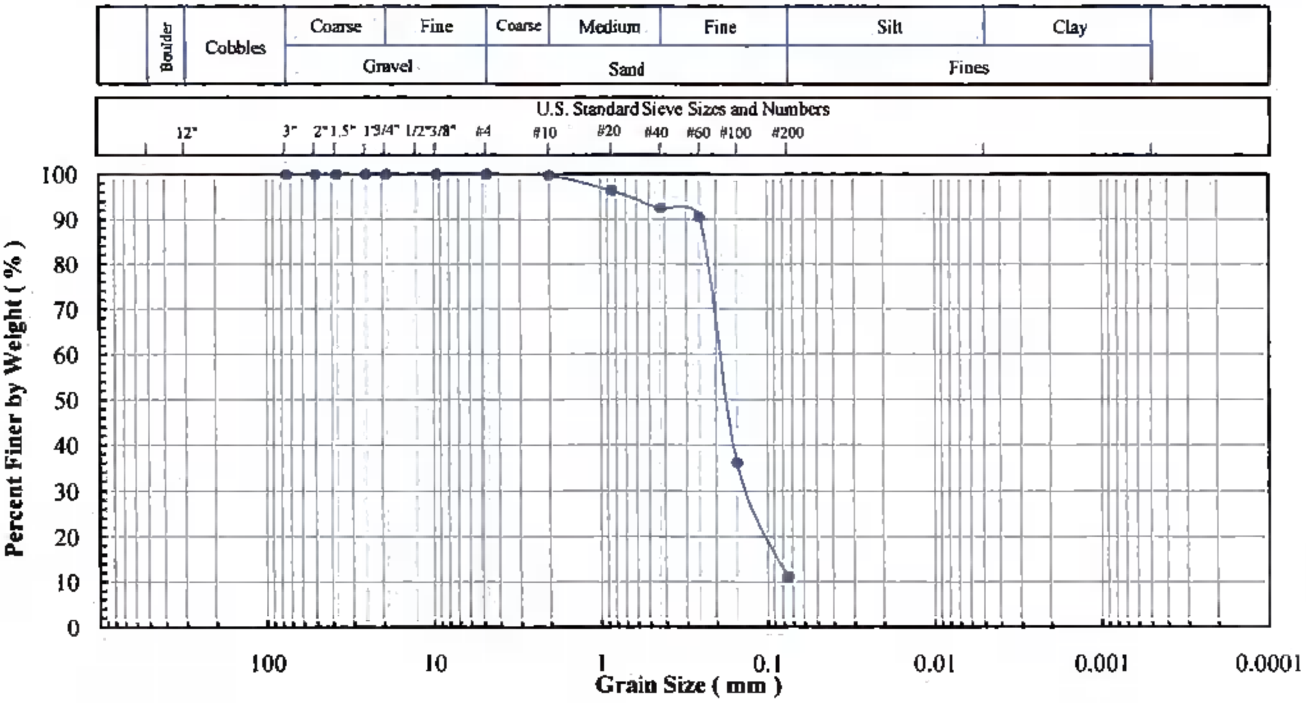
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-01 (8.0-10.0')
Lab Sample No: 13J366

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



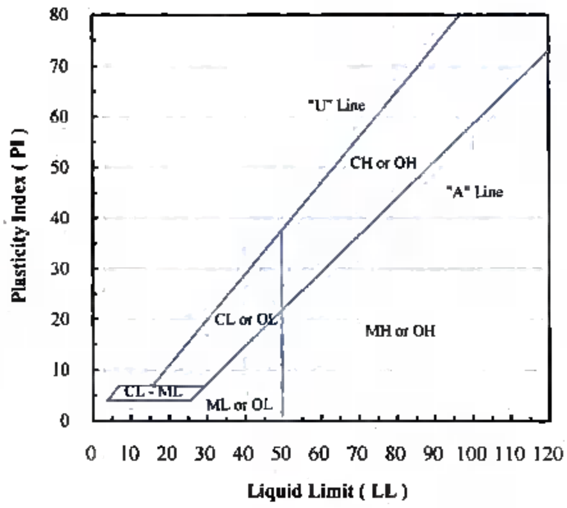
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	96.4
#40	0.425	92.5
#60	0.250	90.6
#100	0.150	36.1
#200	0.075	11.2

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	88.8
Fines (%):	11.2
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
SPT-112, ST-01 (8.0-10.0')	13J366	21.1	11.2				

Note(s):

2-18-14
PD

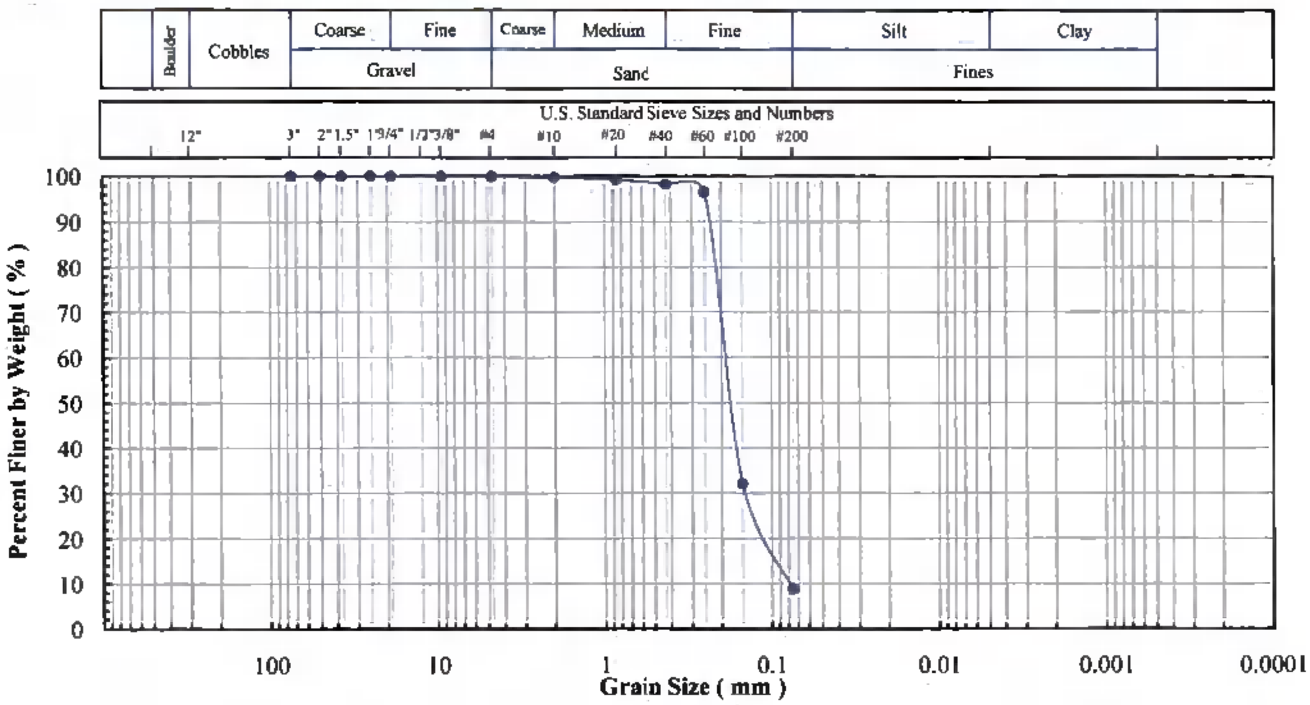
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, SS-07 (25.0-26.5')
Lab Sample No: 13J214

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content, Eng. Classification, Atterberg Limits



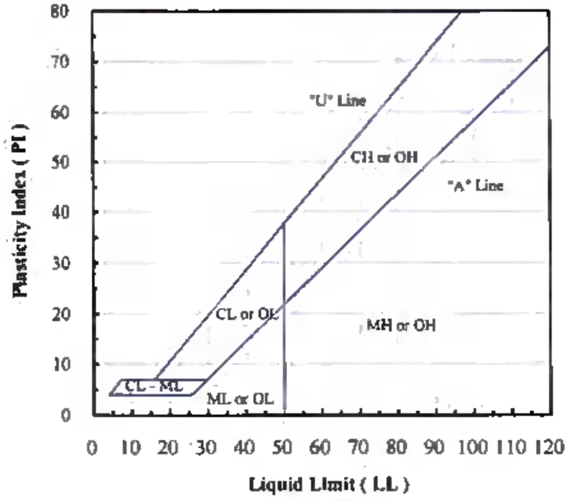
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.9
#10	2.00	99.7
#20	0.850	99.2
#40	0.425	98.2
#60	0.250	96.5
#100	0.150	32.0
#200	0.075	8.8

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	0.1
Sand (%):	91.1
Fines (%):	8.8
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	

Specific Gravity (-):



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
T-112, SS-07 (25.0-26	13J214	29.2	8.8				

Note(s):

11-06-13
TR. NSR

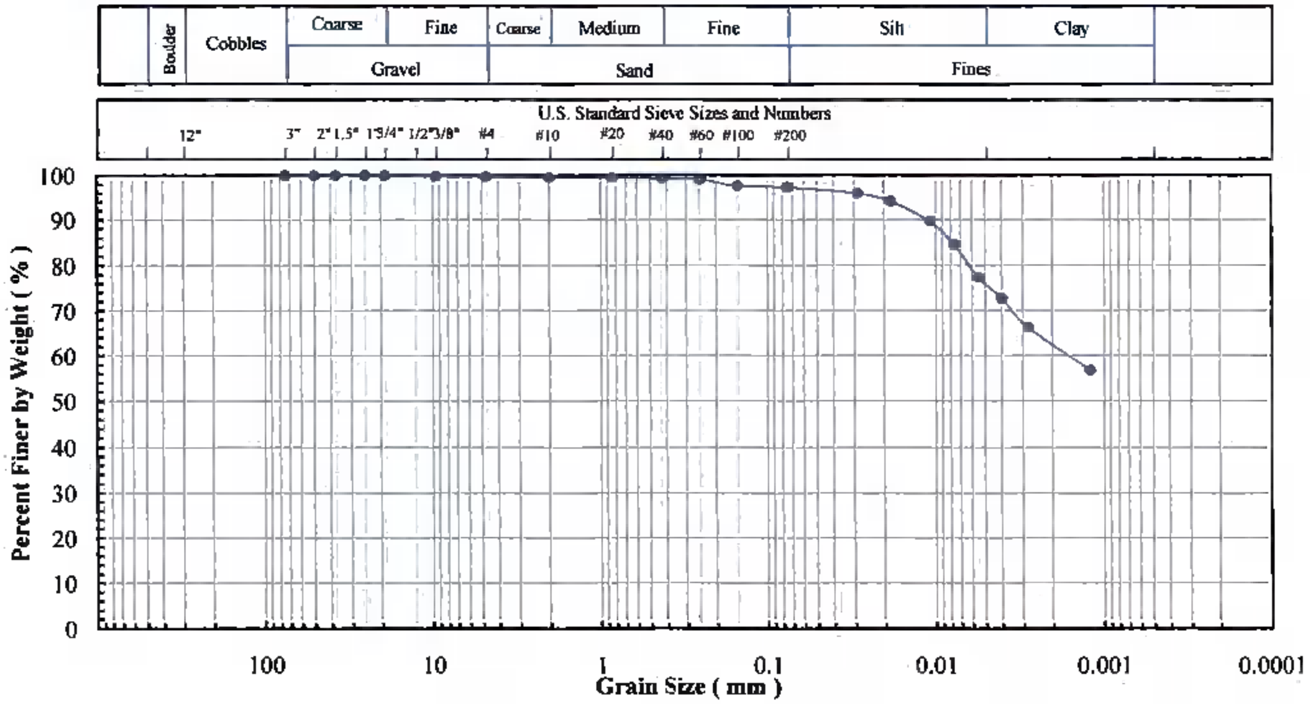
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-112, ST-03 (46.5-48.5')
Lab Sample No:	13J367

ASTM C 136, D 422, D 854, D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont., Eng. Classification, Atterberg Limits



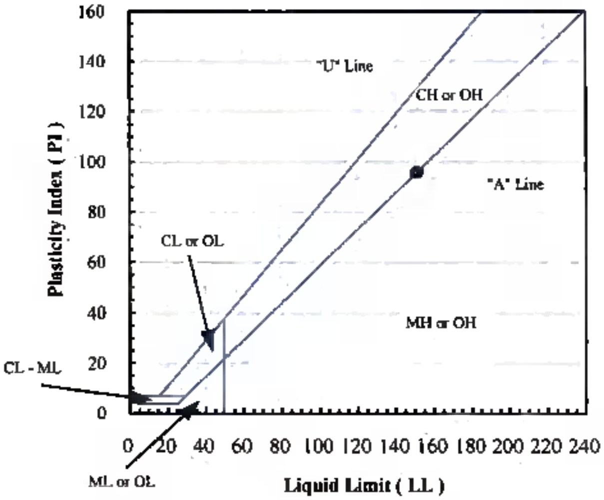
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	99.8
#4	4.75	99.7
#10	2.00	99.6
#20	0.850	99.5
#40	0.425	99.4
#60	0.250	99.2
#100	0.150	97.7
#200	0.075	97.4

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0288	96.0
0.0107	89.7
0.0055	77.4
0.0028	66.4
0.0012	56.9

Gravel (%):	0.3
Sand (%):	2.3
Fines (%):	97.4
Silt (%):	21.5
Clay (%):	75.9

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
SPT-112, ST-03 (46.5-48.5')	13J367		97.4	151	55	96	MH - Elastic silt

Note(s): An assumed specific gravity of 2.7 was used when analyzing the hydrometer test results.

2-18-14
PD

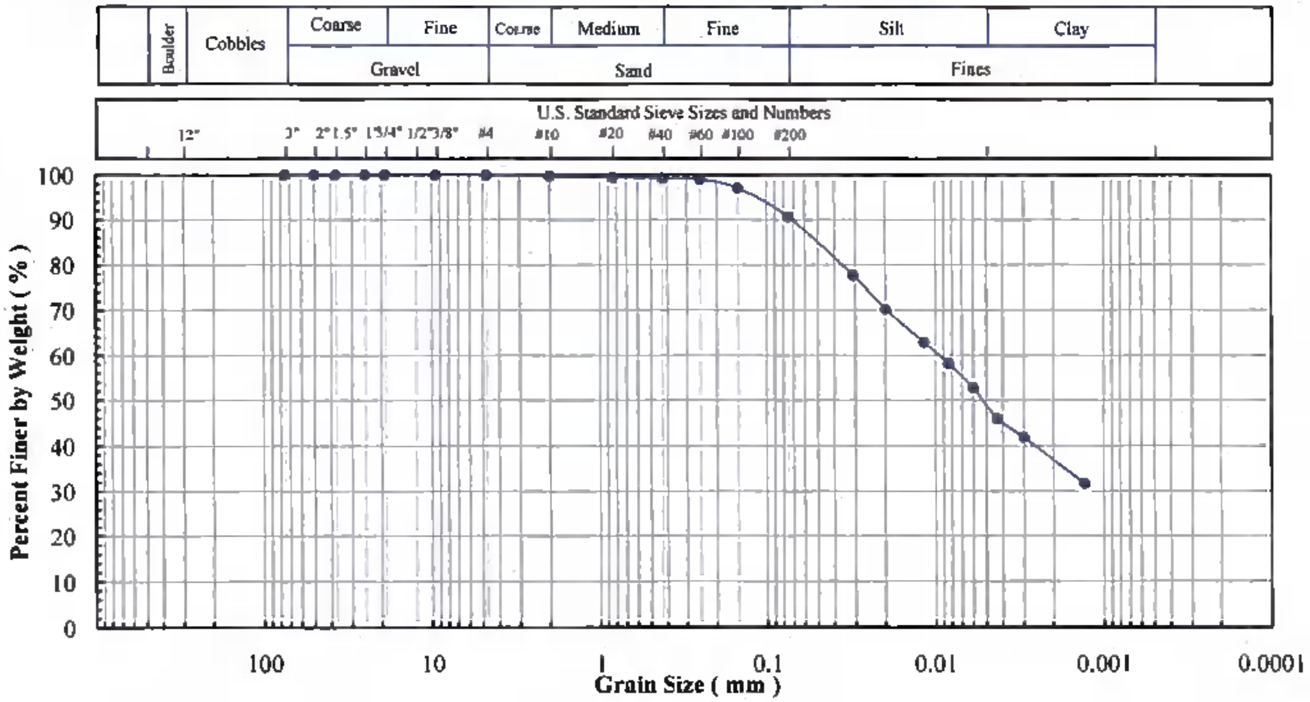
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-112, SS-13 (60.0-61.5')
Lab Sample No:	13J220

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.
Eng. Classification, Atterberg Limits



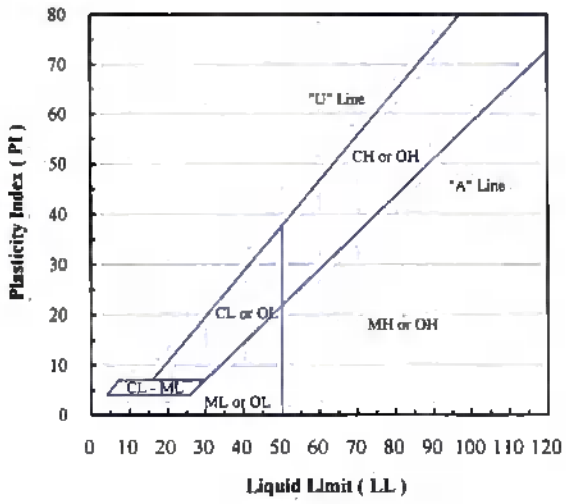
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.5
#40	0.425	99.3
#60	0.250	99.1
#100	0.150	97.1
#200	0.075	90.7

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0308	77.6
0.0117	62.9
0.0060	52.8
0.0030	41.8
0.0013	31.7

Gravel (%):	
Sand (%):	9.3
Fines (%):	90.7
Silt (%):	42.0
Clay (%):	48.7

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
T-112, SS-13 (60.0-6	13J220	48.9	90.7				

Note(s): An assumed specific gravity of 2.70 was used when analyzing the hydrometer test results.

11-06-13
TK.

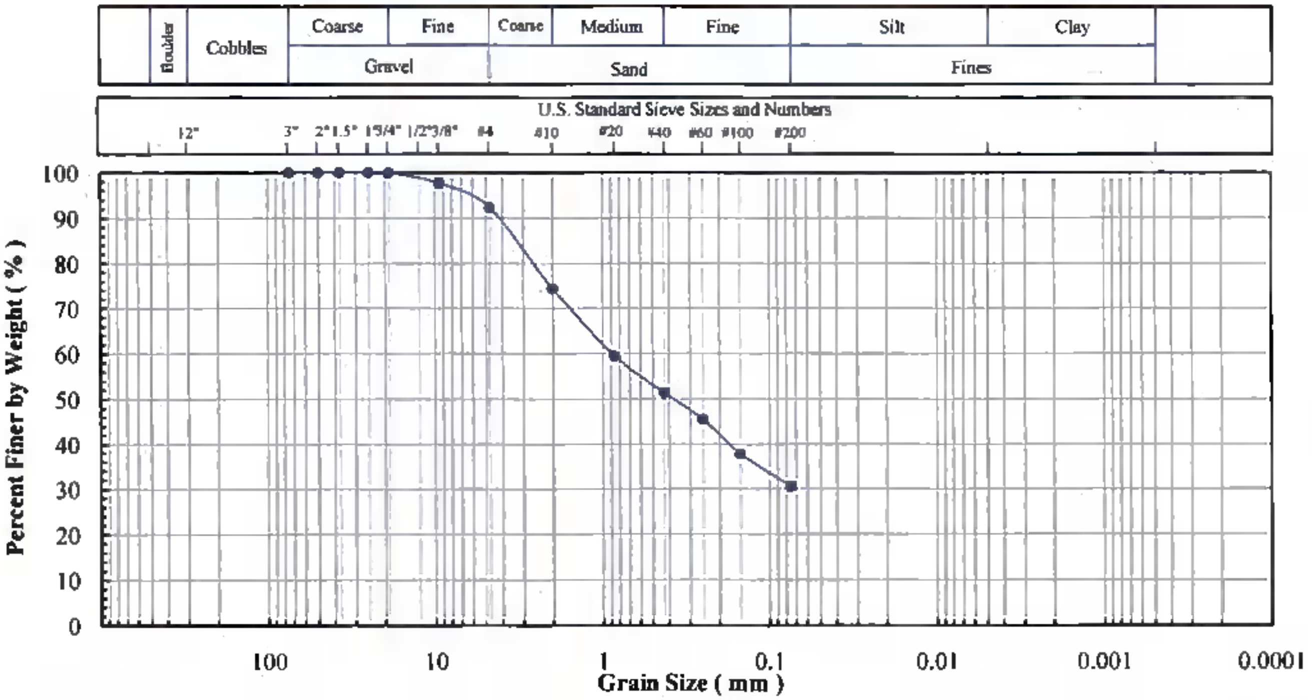
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-113, SS-02 (10.0-11.5')
Lab Sample No: 13J222

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



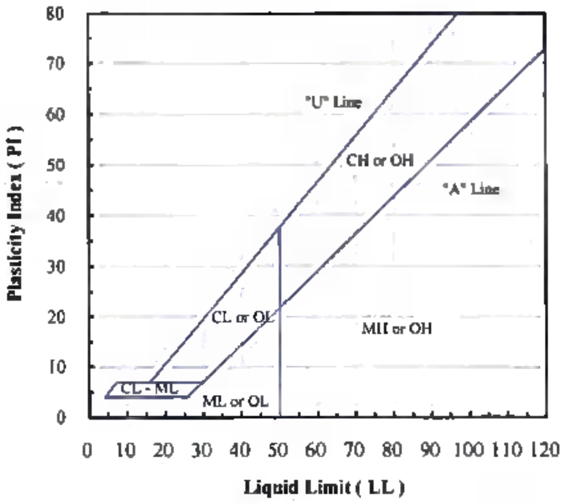
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	97.7
#4	4.75	92.3
#10	2.00	74.4
#20	0.850	59.4
#40	0.425	51.4
#60	0.250	45.5
#100	0.150	37.9
#200	0.075	30.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	7.7
Sand (%):	61.8
Fines (%):	30.5
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
T-113, SS-02 (10.0-11	13J222	31.8	30.5				

Note(s):

11-06-13
TR, NSR

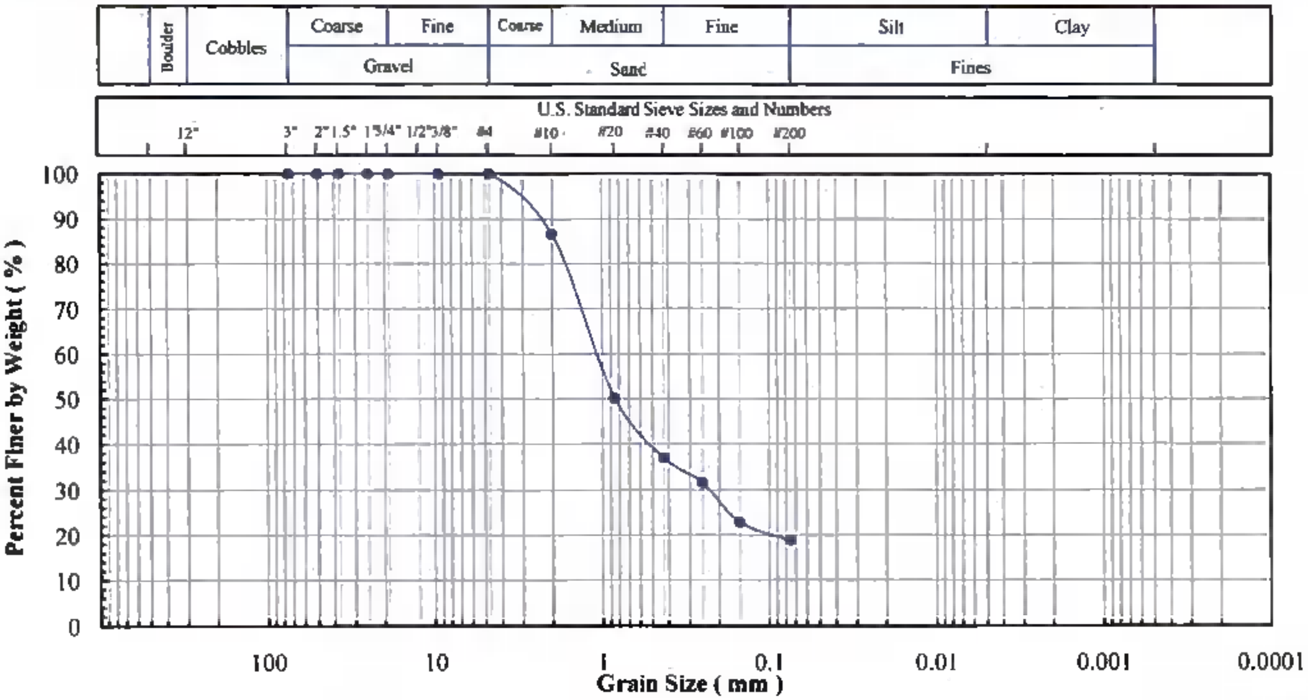
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7637 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-113, SS-03 (15.0-16.5')
Lab Sample No:	13J223

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



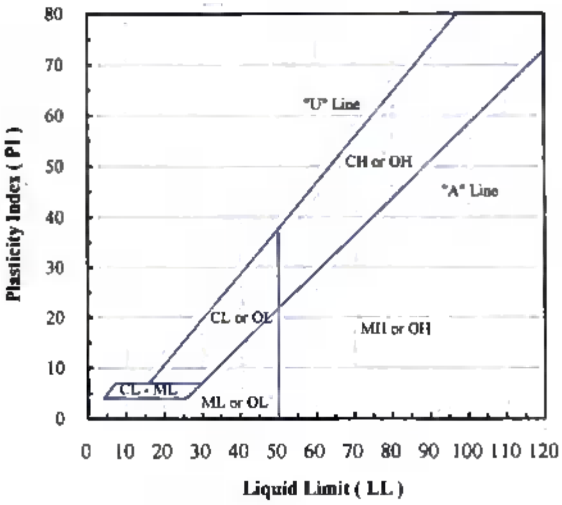
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	86.6
#20	0.850	50.1
#40	0.425	36.9
#60	0.250	31.5
#100	0.150	22.8
#200	0.075	18.9

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	81.1
Fines (%):	18.9
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
T-113, SS-03 (15.0-16	13J223	43.8	18.9				

Note(s):

11-06-17
TE, NSR

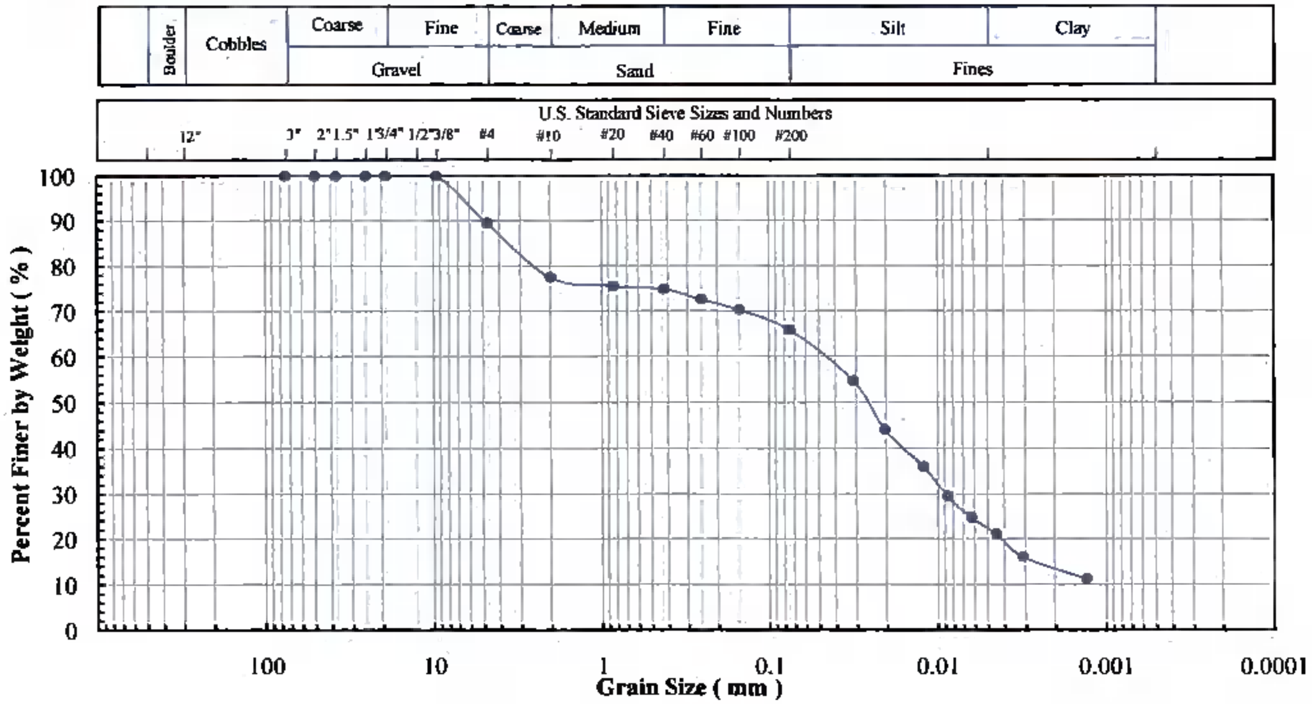
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-113, ST-02 (26.5-28.5')
Lab Sample No:	13J369

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.,
Eng. Classification, Atterberg Limits



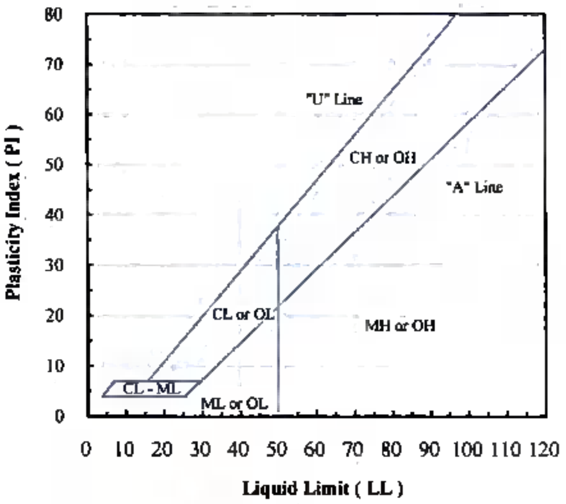
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	89.6
#10	2.00	77.5
#20	0.850	75.6
#40	0.425	74.9
#60	0.250	72.6
#100	0.150	70.3
#200	0.075	66.0

Specific Gravity (-):	2.7

Hydrometer Particle Diameter (mm)	% Finer
0.0314	54.8
0.0121	36.1
0.0063	24.8
0.0031	16.1
0.0013	11.3

Gravel (%):	10.4
Sand (%):	23.6
Fines (%):	66.0
Silt (%):	43.8
Clay (%):	22.2

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-113, ST-02 (26.5-28.5')	13J369	40.5	66.0	NP	NP	NP	ML - Sandy silt

Note(s): An assumed specific gravity of 2.7 was used when analyzing the hydrometer test results.

2-18-14
PD, NSA

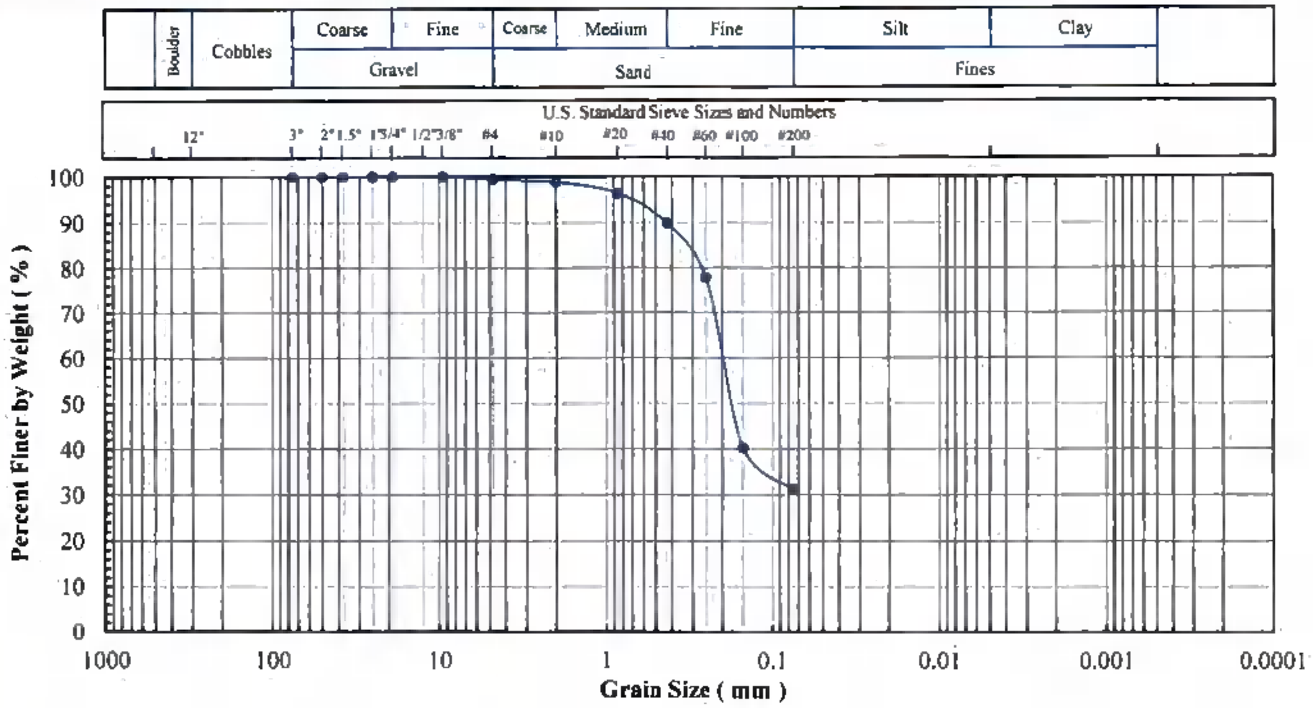
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 771
Client Sample ID: SPT-302-S-3 (4-6')
Lab Sample No: 16C022

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content, Eng. Classification, Atterberg Limits



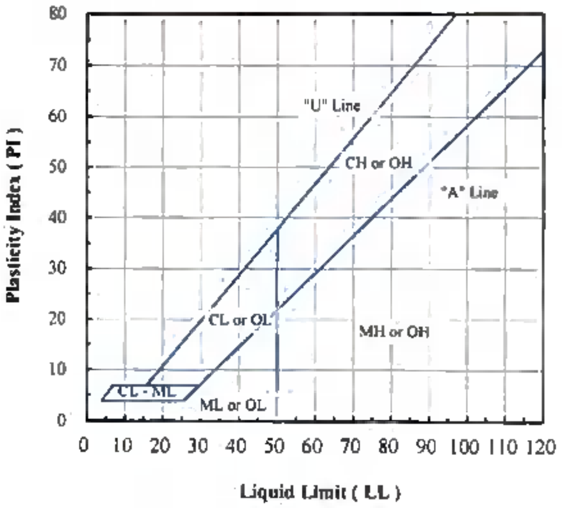
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.5
#10	2.00	98.9
#20	0.850	96.4
#40	0.425	89.9
#60	0.250	77.9
#100	0.150	40.3
#200	0.075	31.3

Specific Gravity (-):

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	0.5
Sand (%):	68.2
Fines (%):	31.3
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-302-S-3 (4-6')	16C022	28.4	31.3				

Note(s):

4-8-16
NSR

Excel Geotechnical Testing, Inc.
"Excellence in Testing"
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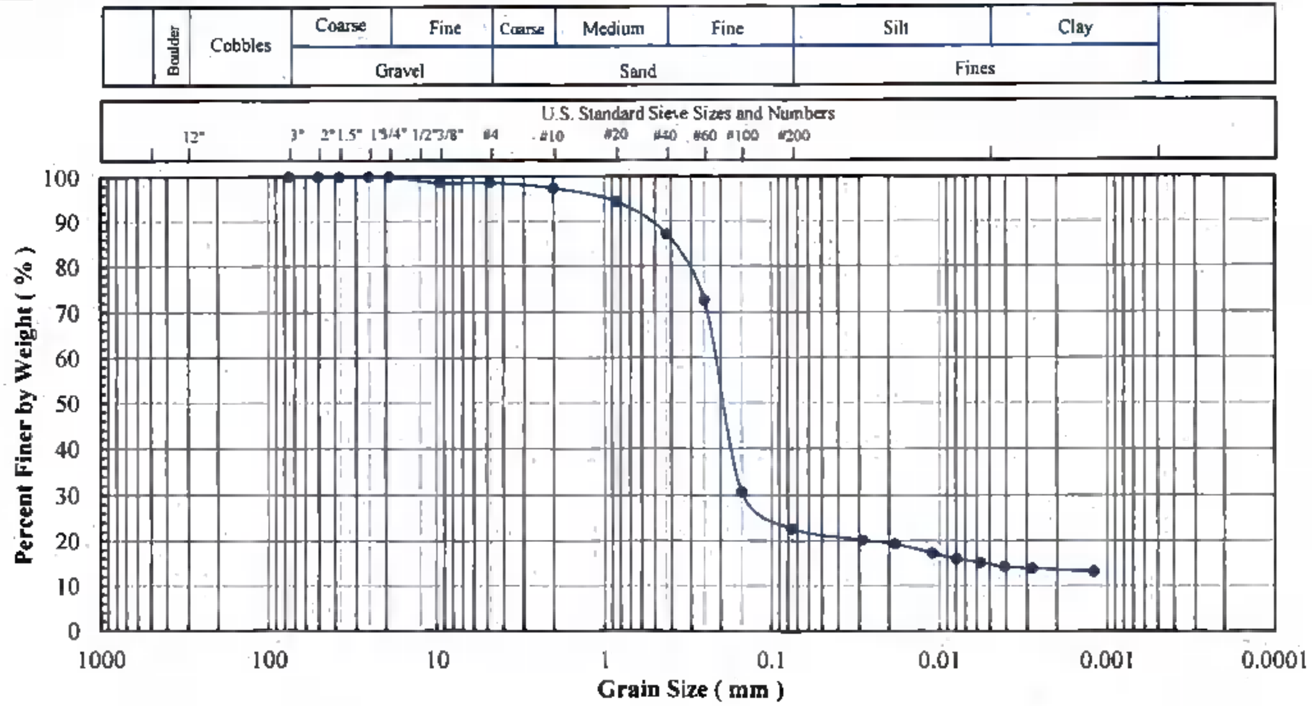
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	771
Client Sample ID:	SPT-302-S-11 (23-25')
Lab Sample No:	16C024

ASTM C 136, D 422, D 854,
D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont.
Eng. Classification, Atterberg Limits



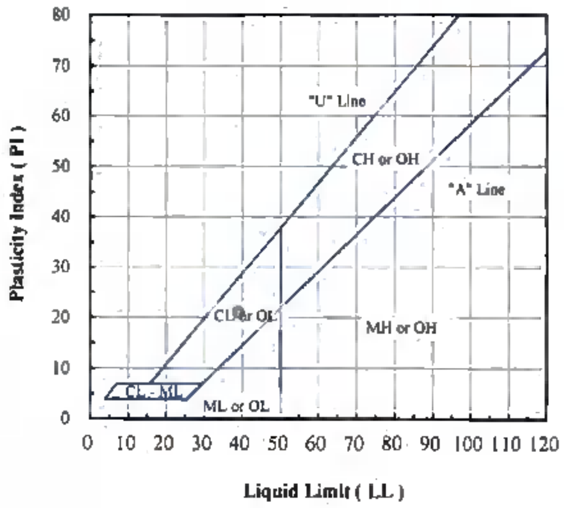
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	98.7
#4	4.75	98.7
#10	2.00	97.5
#20	0.850	94.5
#40	0.425	87.1
#60	0.250	72.6
#100	0.150	30.7
#200	0.075	22.7

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0287	20.2
0.0110	17.3
0.0057	15.2
0.0028	13.9
0.0012	13.2

Gravel (%):	1.3
Sand (%):	76.0
Fines (%):	22.7
Silt (%):	7.9
Clay (%):	14.8

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
SPT-302-S-11 (23-25')	16C024		22.7	39	18	21	SC - Clayey sand

Note(s): An assumed specific gravity of 2.7 was used when analyzing the hydrometer test results.
Engineering classification is based on the assumption that the fines are either CL or CH.

11-08-16
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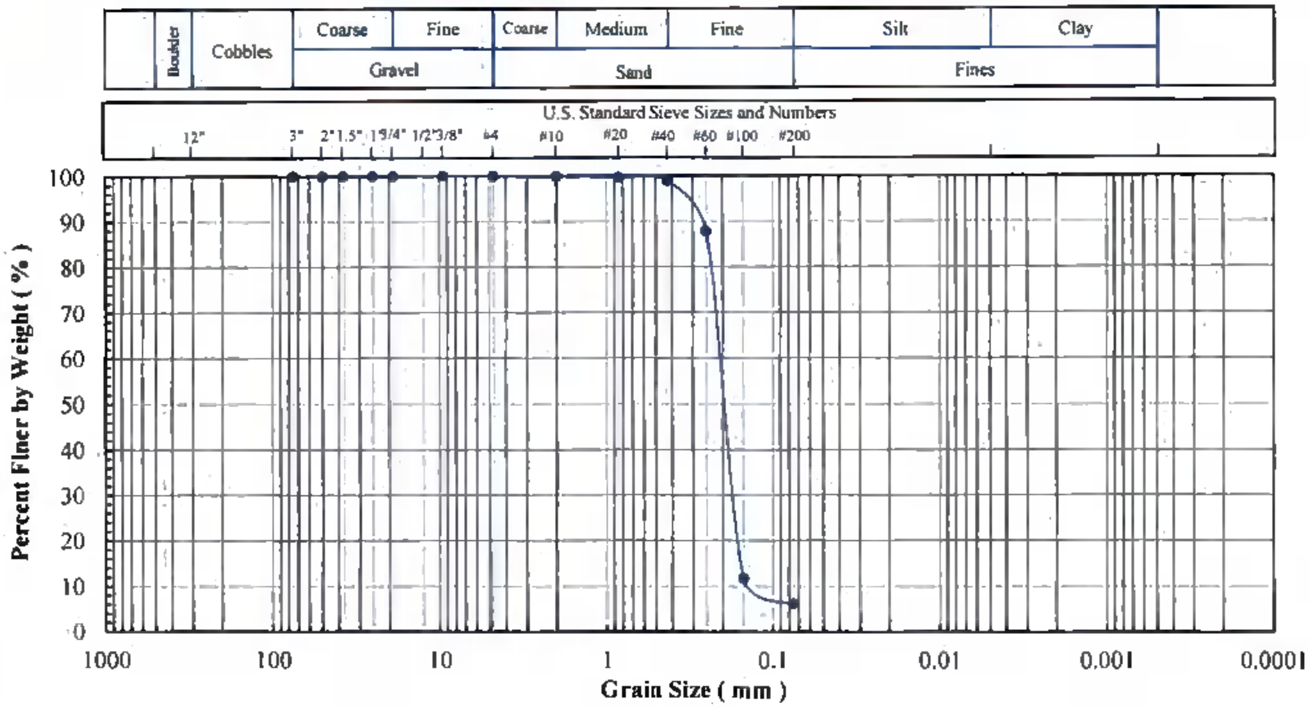
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	771
Client Sample ID:	SPT-302-S-13 (33-35')
Lab Sample No:	16C026

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content, Eng. Classification, Atterberg Limits



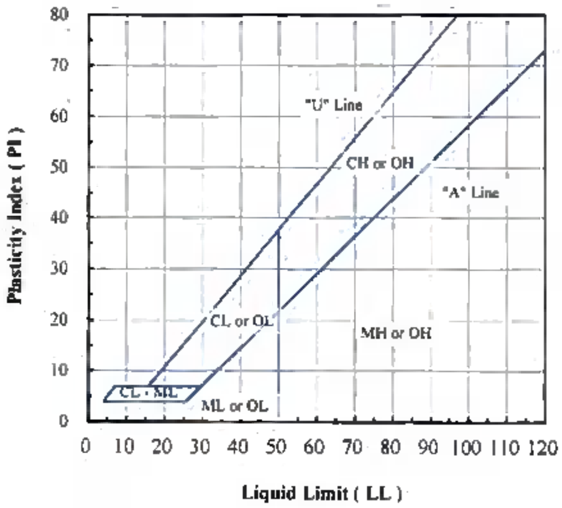
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.7
#40	0.425	98.9
#60	0.250	87.9
#100	0.150	11.8
#200	0.075	6.2

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	93.8
Fines (%):	6.2
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-302-S-13 (33-35')	16C026	22.9	6.2				

Note(s):

4-08-16
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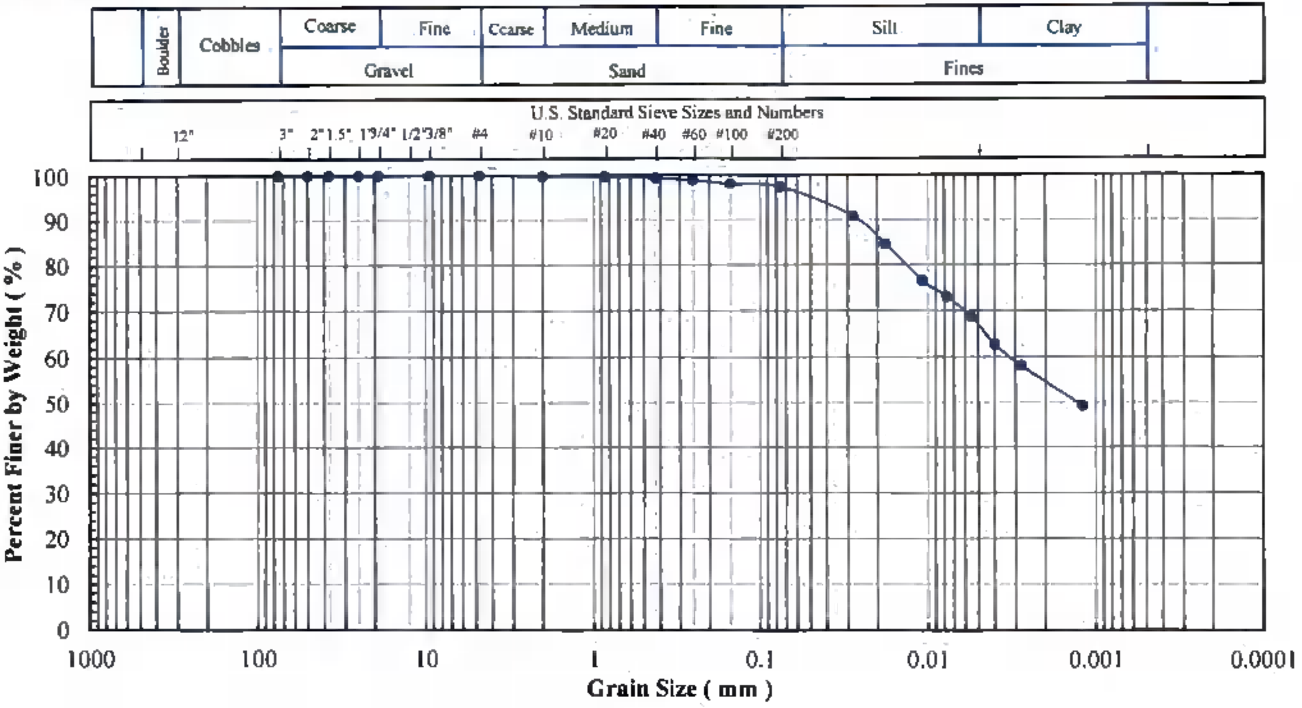
Project Name:	Winyah Generating Station
Project No:	771
Client Sample ID:	SPT-303A-S-15 (43-45')
Lab Sample No:	16C053

ASTM C 136, D 422, D 854, D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Cont., Eng. Classification, Atterberg Limits



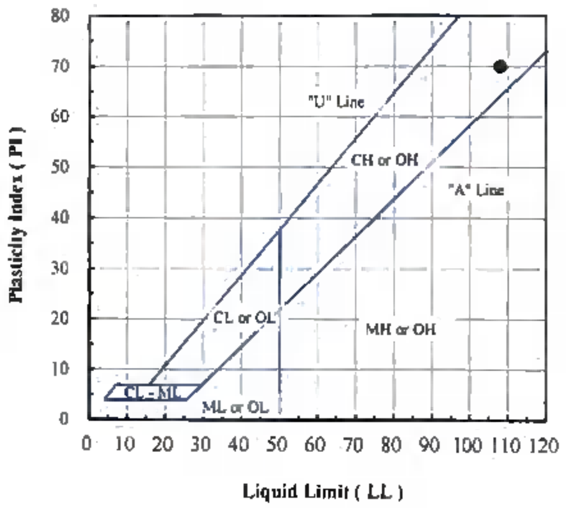
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.7
#40	0.425	99.4
#60	0.250	98.9
#100	0.150	98.1
#200	0.075	97.4

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0279	90.9
0.0108	76.7
0.0055	68.7
0.0028	58.2
0.0012	49.4

Gravel (%):	
Sand (%):	2.6
Fines (%):	97.4
Silt (%):	30.6
Clay (%):	66.8

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits			Engineering Classification
				LL (-)	PL (-)	PI (-)	
SPT-303A-S-15 (43-45')	16C053		97.4	108	38	70	CH - Fat clay

Note(s): An assumed specific gravity of 2.7 was used when analyzing the hydrometer test results.

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Excel Geotechnical Testing, Inc.
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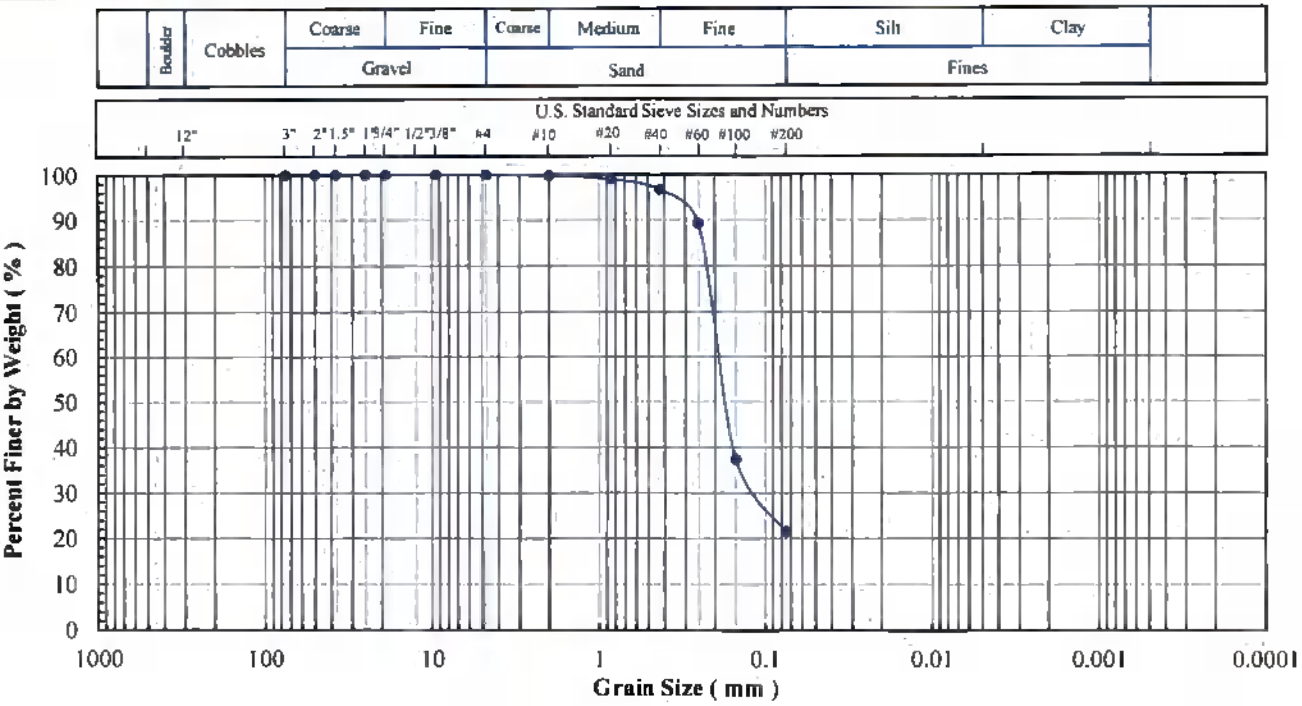
Project Name:	Winyah Generating Station
Project No:	771
Client Sample ID:	SPT-303A-S-5 (8-10')
Lab Sample No:	16C059

ASTM C 136, D 422, D 854,
D 1140, D2216, D 2487, D4318

SOIL INDEX PROPERTIES

Grain Size, Spec. Gravity, Moist. Content,
Eng. Classification, Atterberg Limits



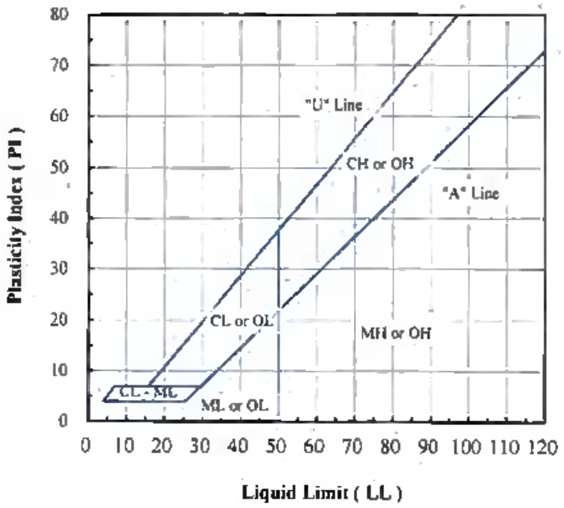
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.0
#40	0.425	96.6
#60	0.250	89.2
#100	0.150	37.4
#200	0.075	21.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	78.5
Fines (%):	21.5
Silt (%):	
Clay (%):	

Coeff. Unif. (Cu):	
Coeff. Curv. (Cc):	



Client Sample ID.	Lab Sample No:	Moisture Content (%)	Fines Content < No. 200 (%)	Atterberg Limits LL (-)	PL (-)	PI (-)	Engineering Classification
SPT-303A-S-5 (8-10')	16C059	17.6	21.5				

Note(s):

4-08-16
NSA

Triaxial Testing

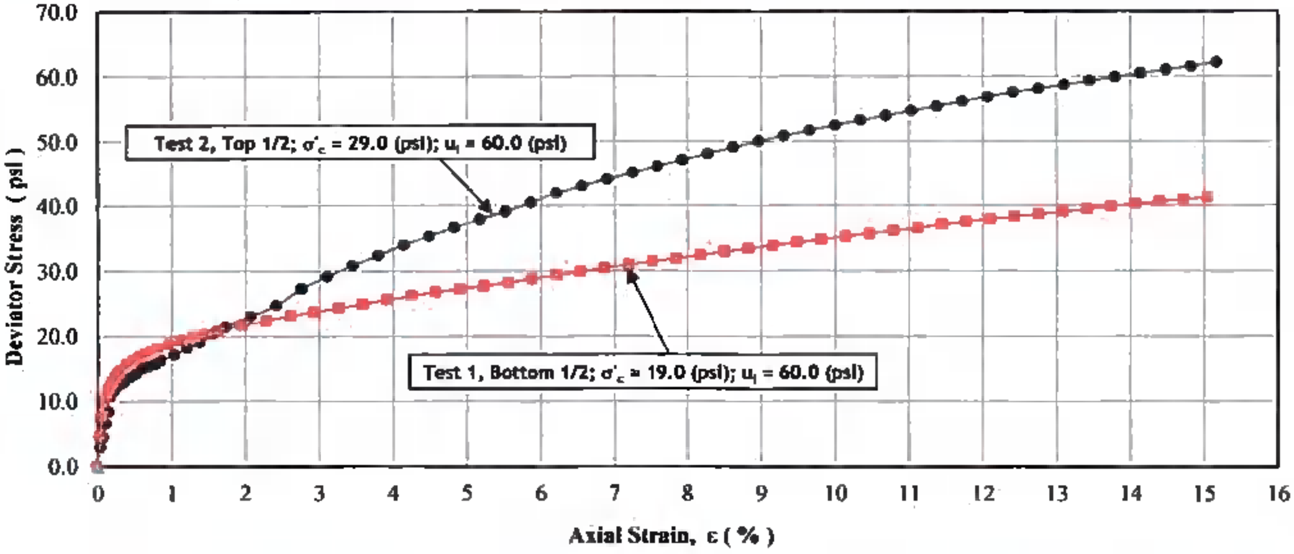
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

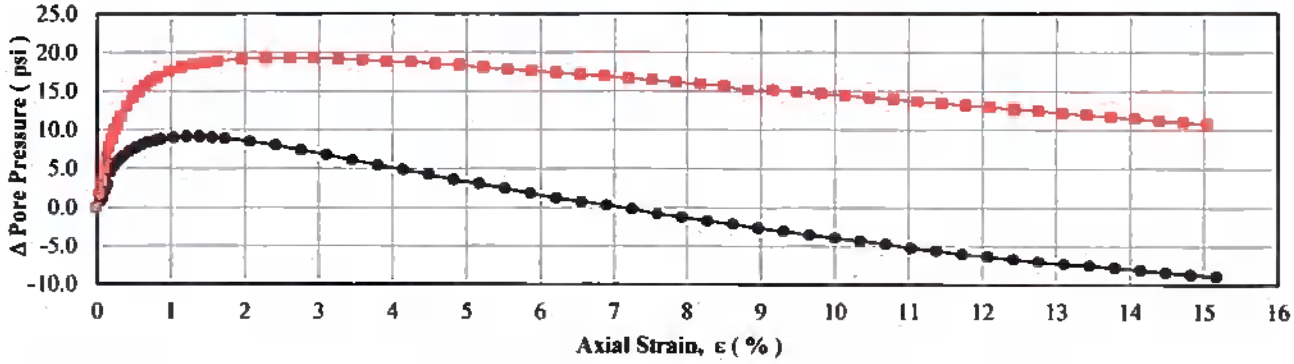
Project Name: Winyah Generating Station
Project No: 618
Site Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 1





Test Specimen No.	Maximum Strength				
	Deviator Stress $(\sigma'_1 - \sigma'_3)$ (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	62.1	90.0	27.9	51.1	15.2
2	41.2	59.4	18.2	70.8	15.1

Test Specimen No.	Strength at App. 15% Axial Strain				
	Deviator Stress $(\sigma'_1 - \sigma'_3)$ (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	62.1	90.0	27.9	51.1	15.2
2	41.2	59.4	18.2	70.8	15.1

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi)

11-21-13
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Excel Geotechnical Testing, Inc.
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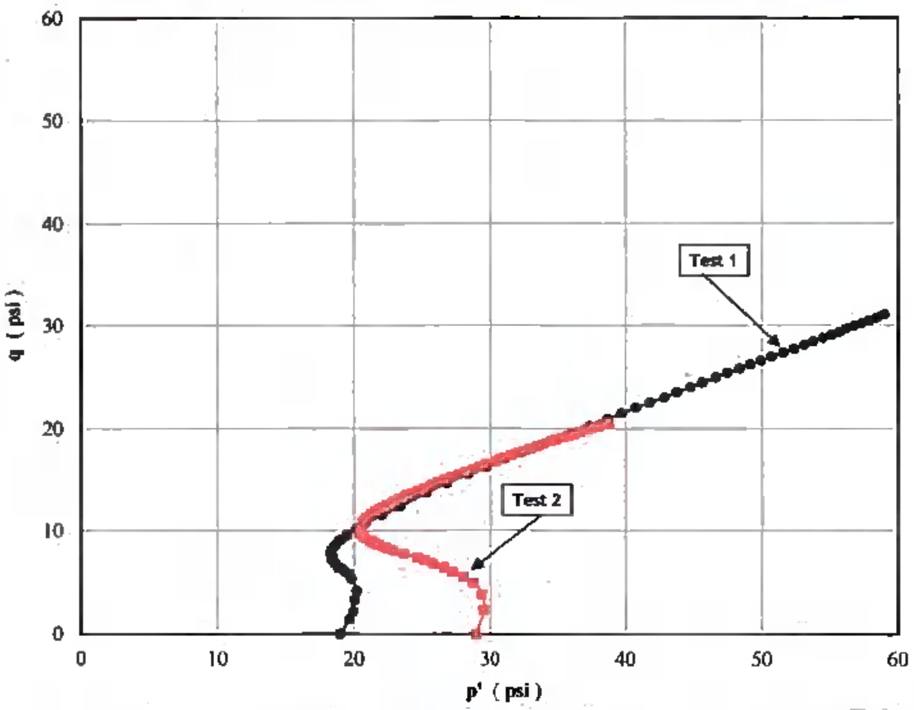
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Site Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

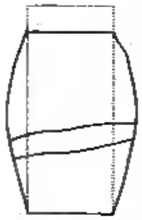
Figure 2



Test Specimen No.	Initial Conditions							Axial	Specimen
	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	Initial Pore Pressure (u_o) (psi)	Consolidation Pressure (σ'_c) (psi)	Strain (% / min)	Quality Bad to Good (1 to 10)
1	5.87	2.87	14.7	115.8	0.95	60.0	19.0	0.068	8
2	6.23	2.86	19.4	104.8	0.96	60.0	29.0	0.064	8

Specimen No. 1

Tan grey sandy clay



Specimen No. 2

Tan brown sandy clay

Specimen No. 3

Notes:

11-21-13
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"Excellence In Testing"

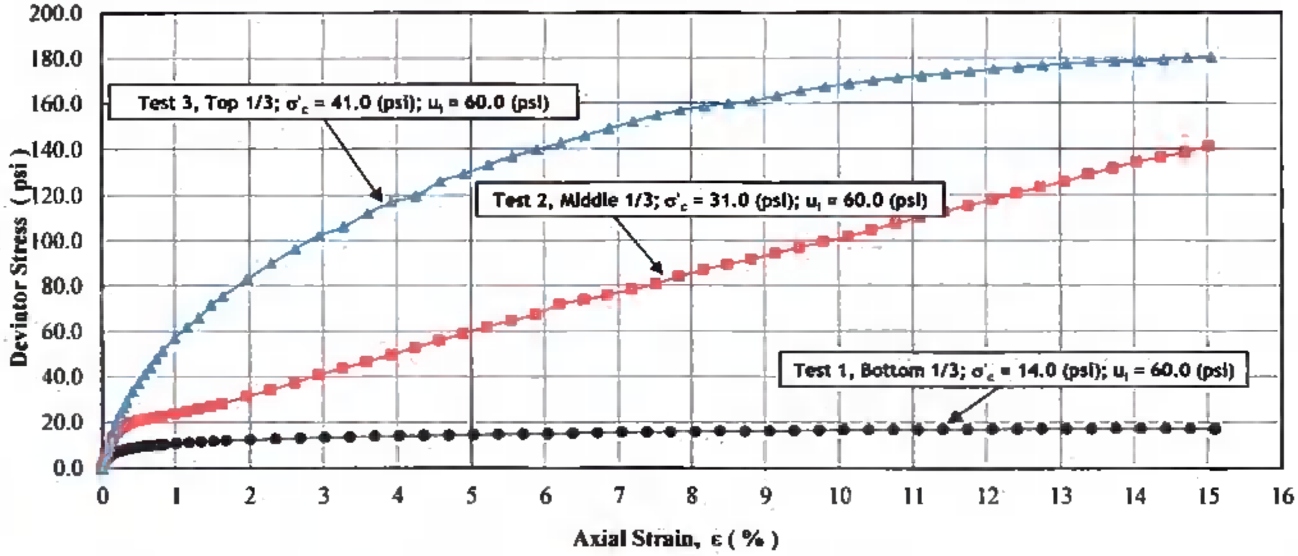
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

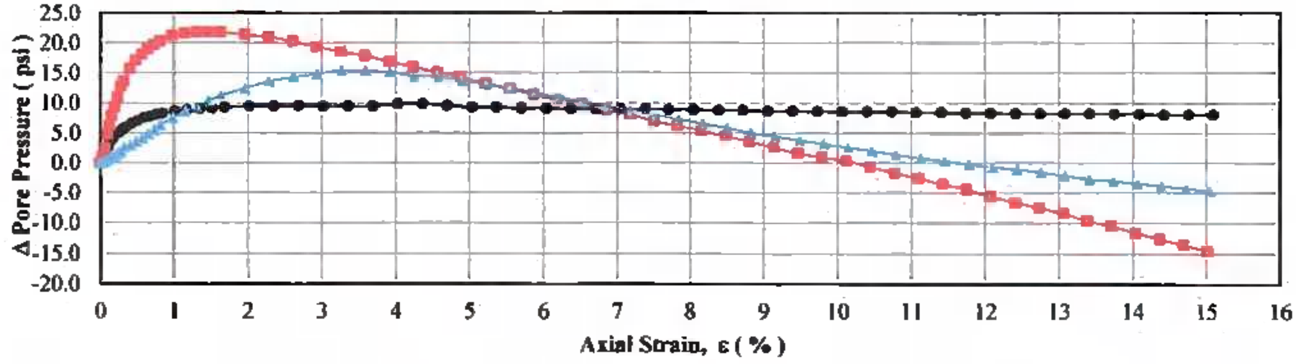
Project Name: Winyah Generating Station
Project No: 618
Site Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 1





Test Specimen No.	Maximum Strength				
	Deviator Stress $(\sigma'_1 - \sigma'_3)$ (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	17.0	22.9	5.9	68.1	15.1
2	141.1	186.6	45.5	45.5	15.0
3	180.9	226.2	45.3	55.7	15.1

Test Specimen No.	Strength at App. 15% Axial Strain				
	Deviator Stress $(\sigma'_1 - \sigma'_3)$ (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	17.0	22.9	5.9	68.1	15.1
2	141.1	186.6	45.5	45.5	15.0
3	180.9	226.2	45.3	55.7	15.1

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi)

11-23-13
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Excel Geotechnical Testing, Inc.
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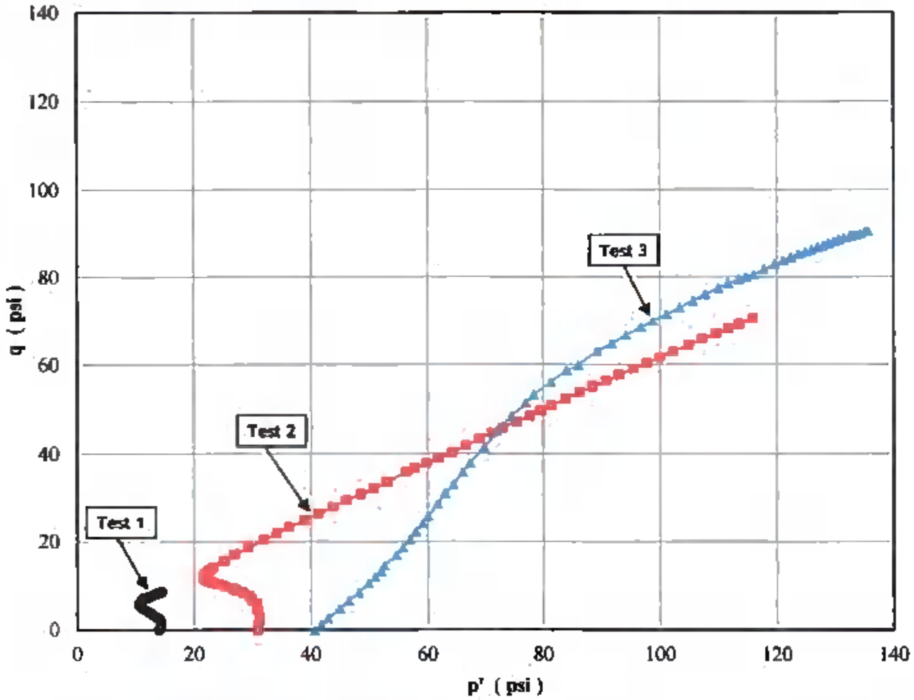
Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-110, ST-02 (36.5-38.5')
Lab Sample No:	13J364

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

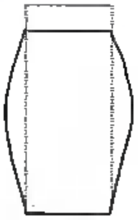
Figure 2



Test Specimen No.	Initial Conditions							Axial	Specimen
	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	Initial Pore Pressure (u_c) (psi)	Consolidation Pressure (σ'_c) (psi)	Strain (% / min)	Quality Bad to Good (1 to 10)
1	6.06	2.85	36.1	87.1	0.98	60.0	14.0	0.066	8
2	6.19	2.84	27.2	95.4	0.98	60.0	31.0	0.065	2
3	6.19	2.85	16.2	108.6	0.97	60.3	40.7	0.065	5

Specimen No.1

Gray sandy clay with shells



Specimen No. 2

Gray sandy clay with shells

Specimen No. 3

Gray sandy clay with shells

Notes:

11-23-13
NSR

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"Excellence in Testing"

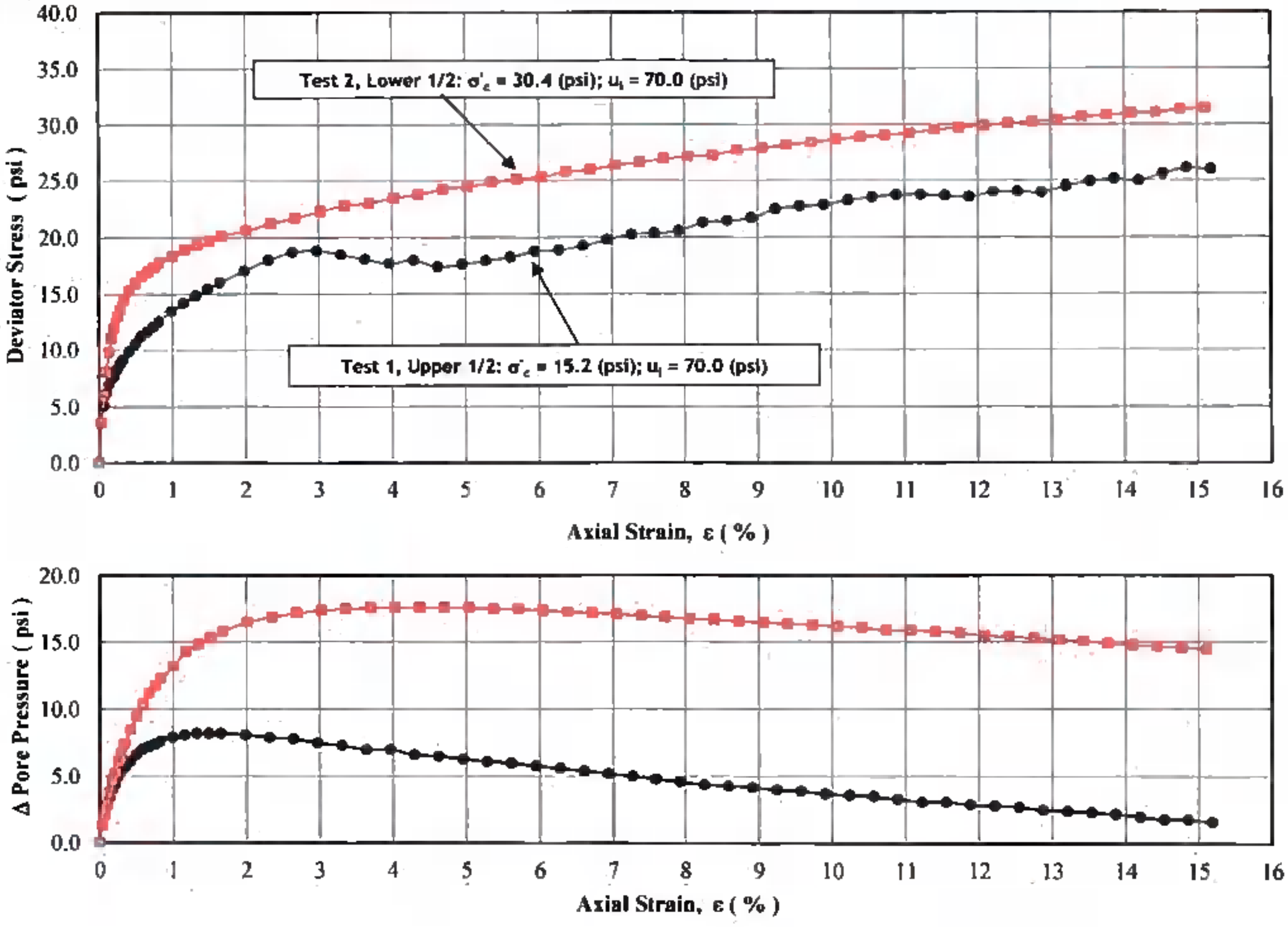
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	771
Site Sample ID:	SPT-302-S-11 (23-25')
Lab Sample No:	16C024

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 1



Test Specimen No.	Maximum Strength				
	Deviator Stress ($\sigma'_1 - \sigma'_3$) (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	25.9	39.5	13.6	71.6	15.2
2	31.4	47.3	15.9	84.5	15.1

Test Specimen No.	Strength at App. 15% Axial Strain				
	Deviator Stress ($\sigma'_1 - \sigma'_3$) (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	25.9	39.5	13.6	71.6	15.2
2	31.4	47.3	15.9	84.5	15.1

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi)

4-15-16
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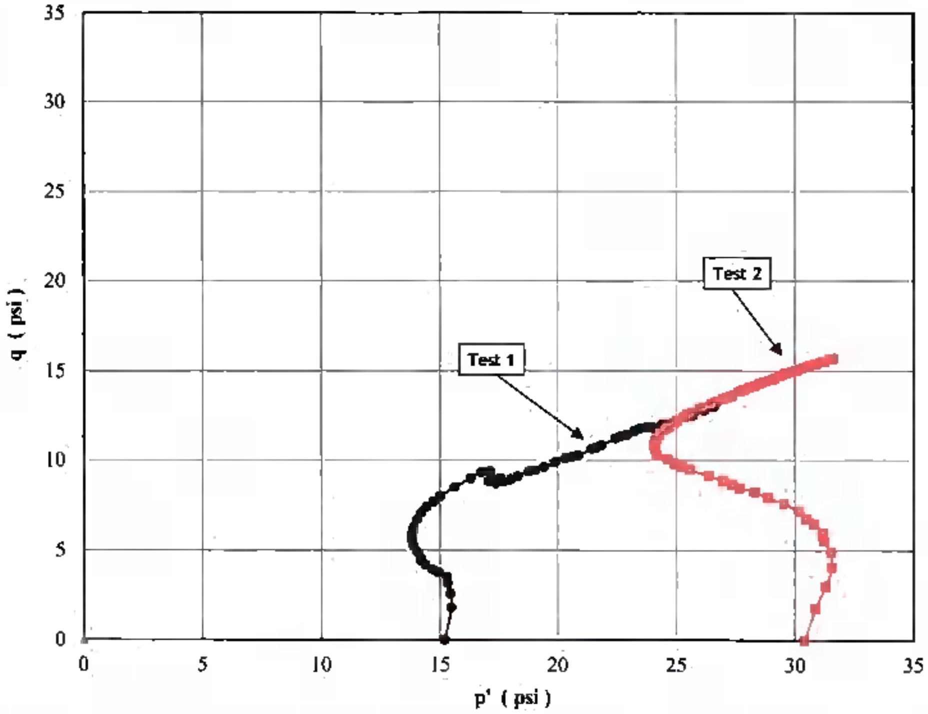
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	771
Site Sample ID:	SPT-302-S-11 (23-25')
Lab Sample No:	16C024

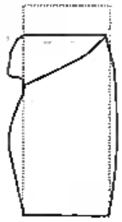
ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 2



Test	Specimen	Initial Conditions							Consolidation Stage		Loading
Specimen Number	Quality Bad to Good	Height	Diameter	Moisture Content	Dry Unit Weight	B Parameter	Initial Pore Pressure (u_i)	Consolidation Pressure (σ'_c)	Axial Strain	Volumetric Strain	Axial Rate
(-)	(1 to 10)	(in.)	(in.)	(%)	(pcf)	(-)	(psi)	(psi)	(%)	(%)	(% / min)
1	7	6.14	2.85	18.8	108.5	0.96	70.0	15.2	1.26	3.26	0.033
2	4	6.13	2.86	20.8	101.0	0.96	70.0	30.4	2.84	5.40	0.033



Specimen No.1

Tan brown silty clay

Specimen No. 2

Tan brown silty clayey sand

Specimen No. 3

Notes:

4-13-16
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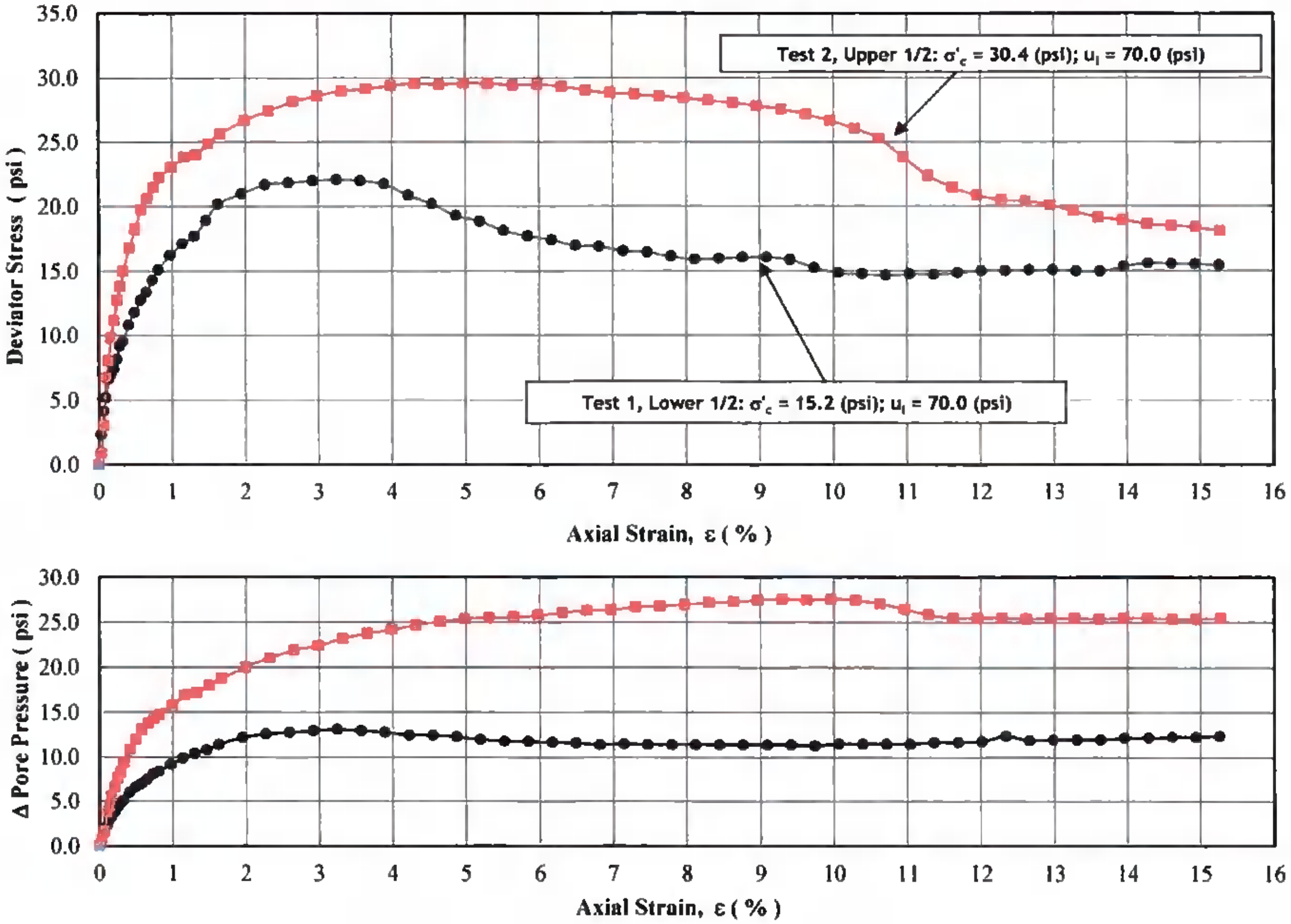
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 771
Site Sample ID: SPT-303A-S-15 (43-45')
Lab Sample No: 16C053

ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 1



Test 2, Upper 1/2: σ'_c = 30.4 (psi); u_i = 70.0 (psi)

Test 1, Lower 1/2: σ'_c = 15.2 (psi); u_i = 70.0 (psi)

Test Specimen No.	Maximum Strength				
	Deviator Stress ($\sigma'_1 - \sigma'_3$) (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	22.1	30.0	7.9	83.1	3.2
2	29.6	46.2	16.6	95.4	5.0

Test Specimen No.	Strength at App. 15% Axial Strain				
	Deviator Stress ($\sigma'_1 - \sigma'_3$) (psi)	Effective Axial Stress (σ'_1) (psi)	Effective Radial Stress (σ'_3) (psi)	Pore Pressure (u) (psi)	Axial Strain (ε_a) (%)
1	15.5	24.1	8.6	82.4	15.3
2	18.1	34.6	16.5	95.5	15.3

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi)

4-13-16
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"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
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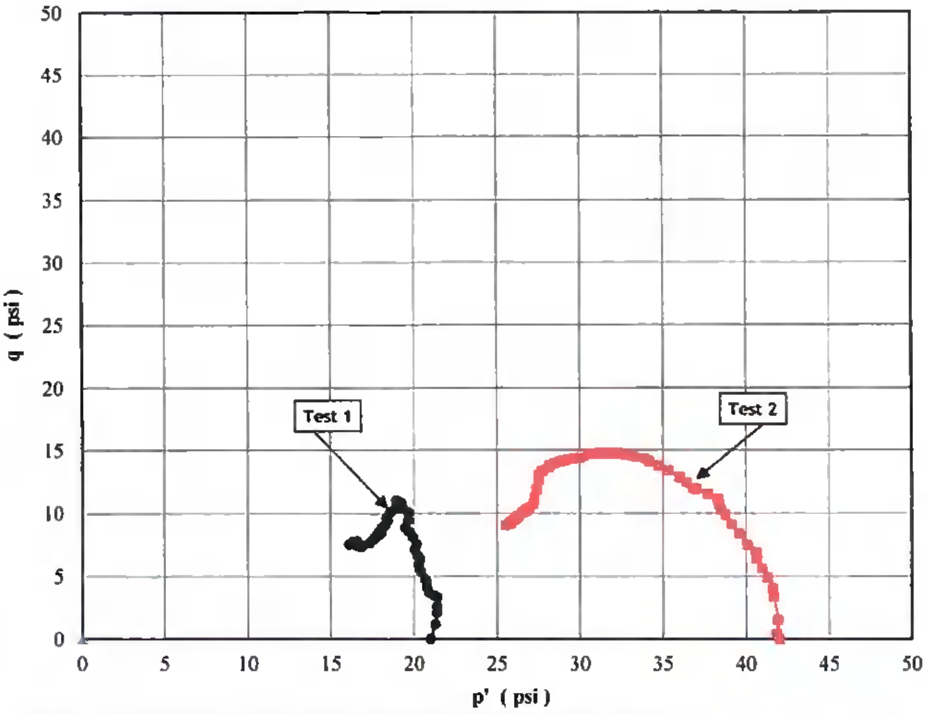
Project Name: Winyah Generating Station
Project No: 771
Site Sample ID: SPT-303A-S-15 (43-45')
Lab Sample No: 16C053

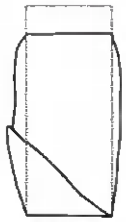
ASTM D 4767

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS

Figure 2



Test	Specimen	Initial Conditions							Consolidation Stage		Loading
Specimen Number (-)	Quality Bad to Good (1 to 10)	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	Initial Pore Pressure (u_i) (psi)	Consolidation Pressure (σ'_c) (psi)	Axial Strain (%)	Volumetric Strain (%)	Axial Rate (% / min)
1	9	6.35	2.85	82.4	51.2	0.96	70.0	21.0	2.92	7.33	0.032
2	10	6.28	2.85	85.9	50.5	0.95	70.0	42.0	4.17	9.53	0.032



Specimen No.1
Brown sandy silt

Specimen No. 2
Brown sandy silt

Specimen No. 3

Notes:

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Consolidation/Permeability Testing

Excel Geotechnical Testing, Inc
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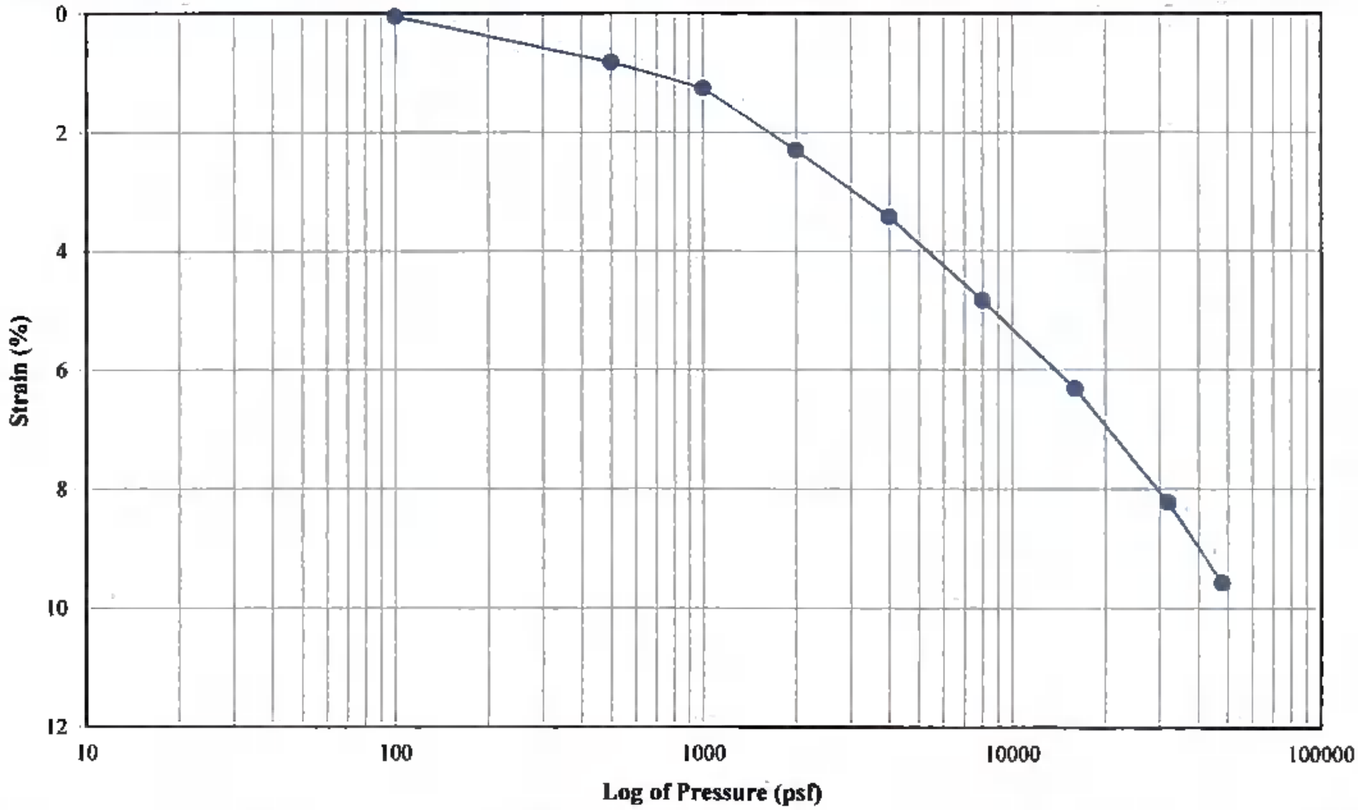
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST



Client Sample ID	Lab Sample No.	Specimen Quality 1-10 (Bad to Good)	Test Specimen Initial Conditions				Consolidation Pressure (psf)	Pressure Increment Duration (min)	Accumu.[1] Vertical Strain (%)	Figure No.	Remarks
			Height (cm)	Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)					
SPT-109, ST-01 (21.5-23.5')	13J361	5	2.54	6.35	106.2	16.7	100	47	0.05	1	
							500	120	0.82	2	
							1000	240	1.25	3	
							2000	218	2.29	4	
							4000	240	3.42	5	
							8000	1240	4.82	6	
							16000	1565	6.32	7	
							32000	300	8.22	8	
							48000	1713	9.57	9	

Notes:

For each pressure increment, the vertical strain values were calculated based on the final deformation measurements.

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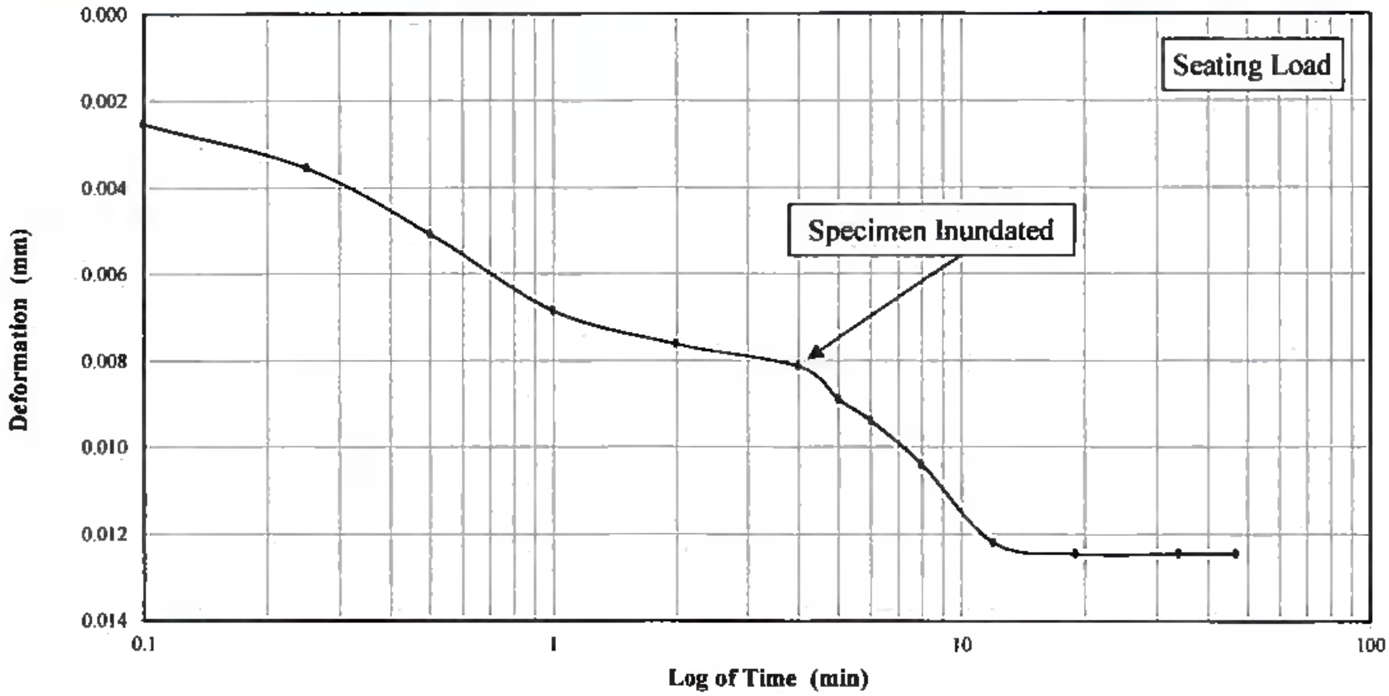
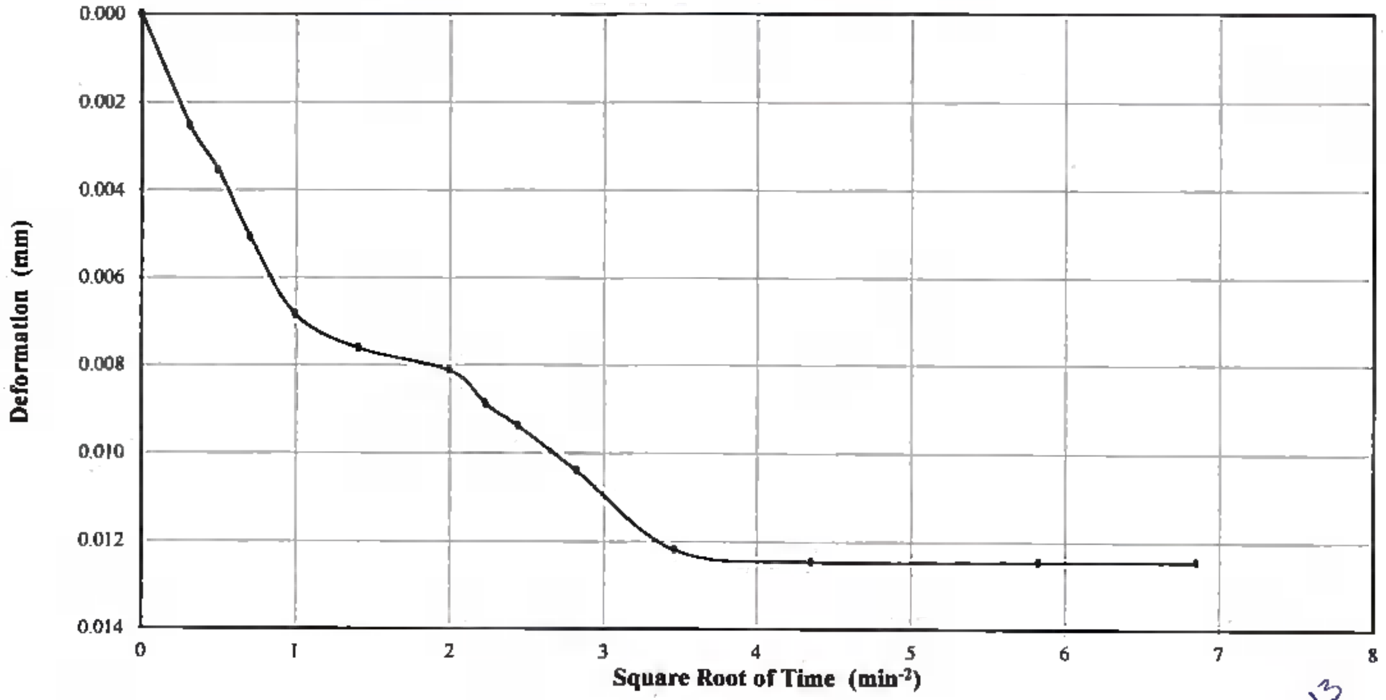
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





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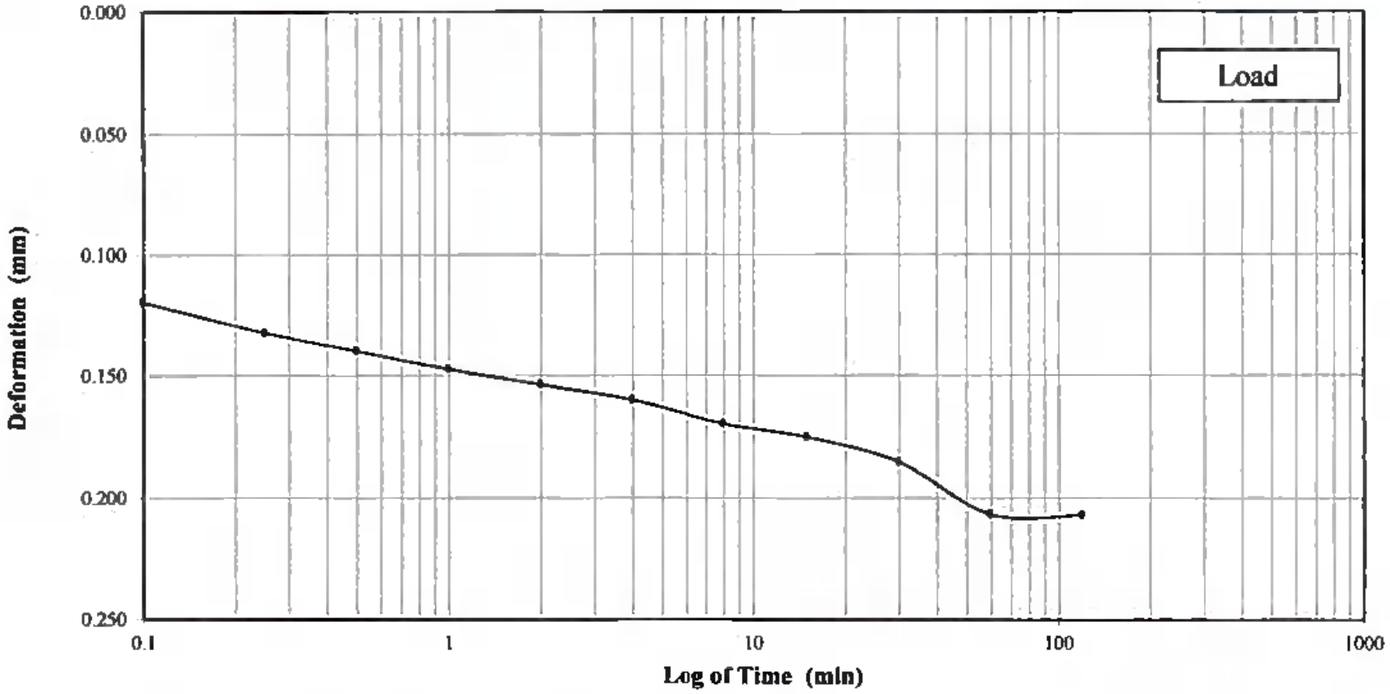
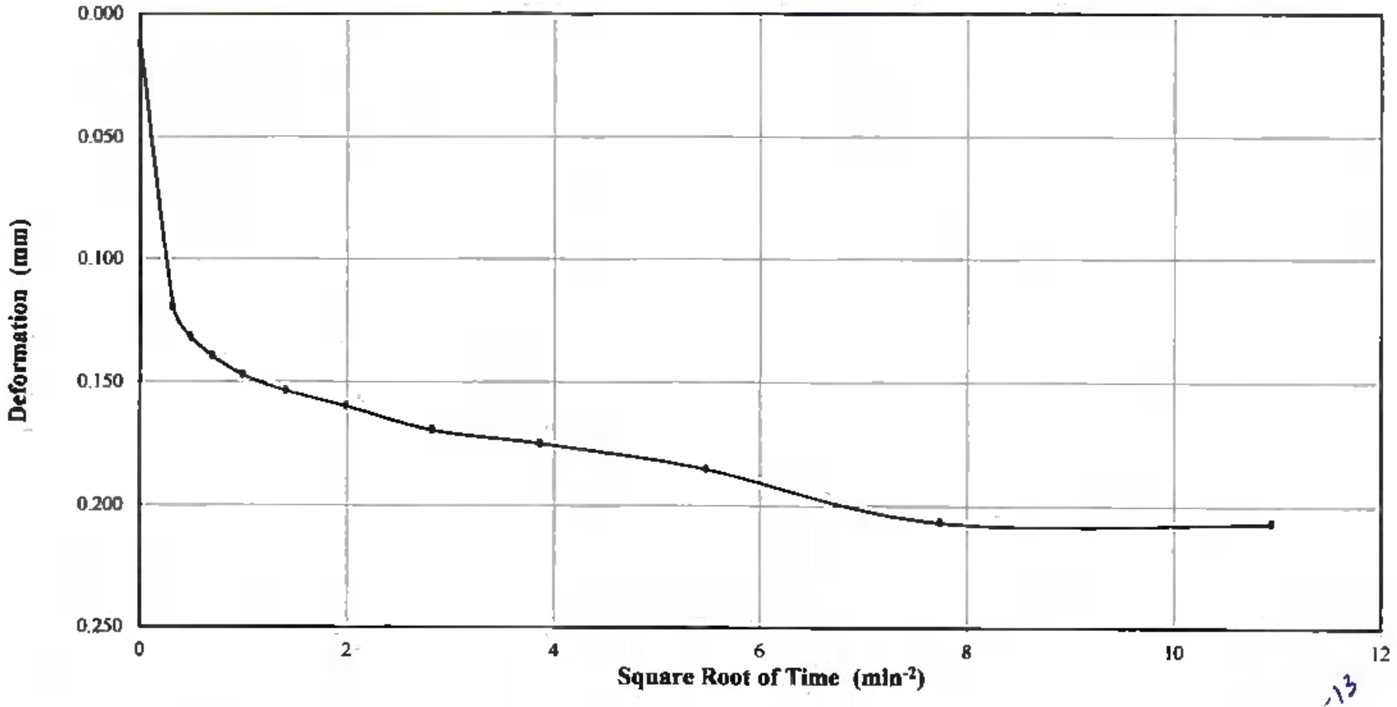
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf





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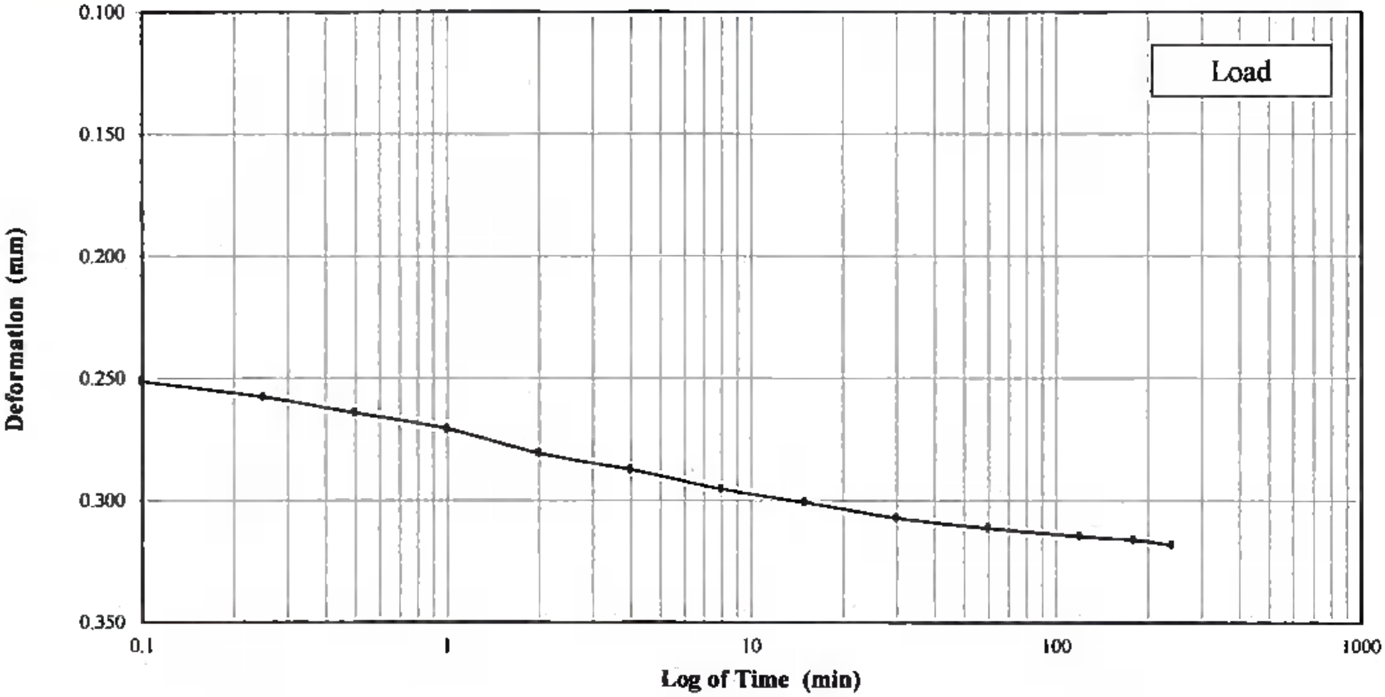
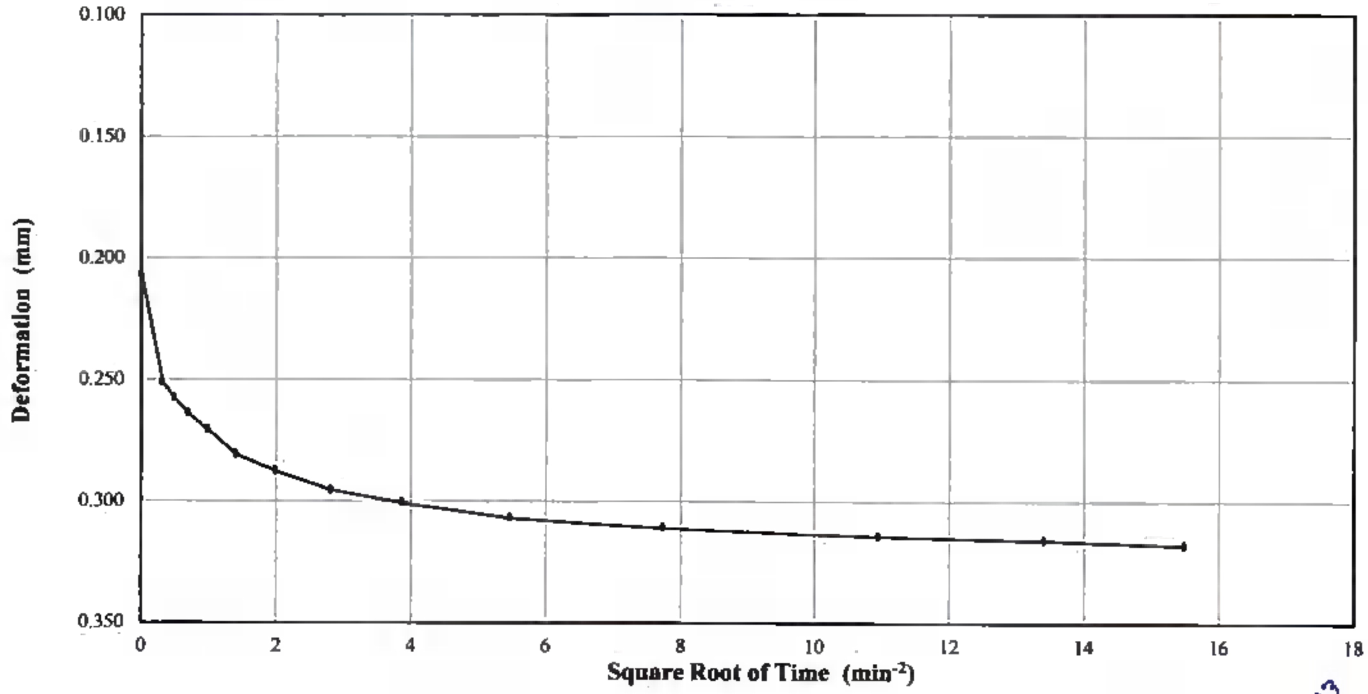
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-109, ST-01 (21.5-23.5')
Lab Sample No:	13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 3 - 1000 psf





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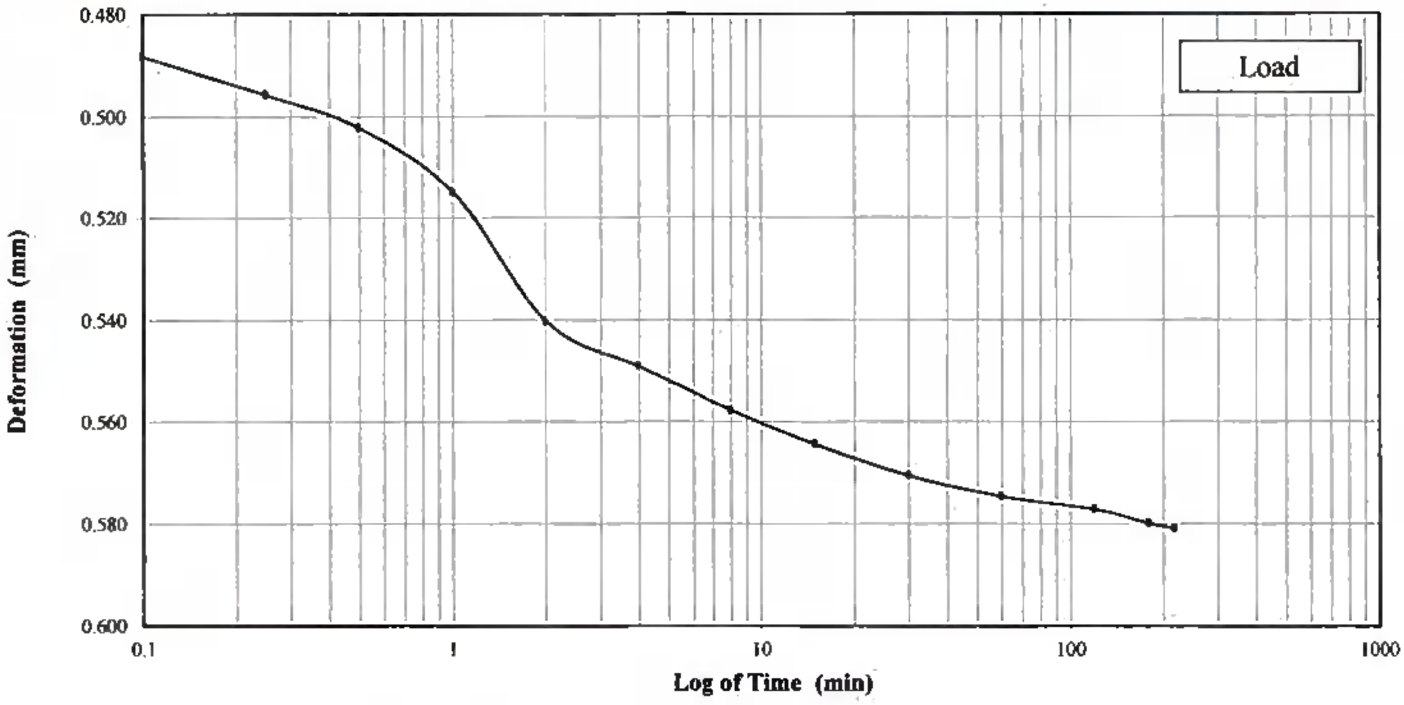
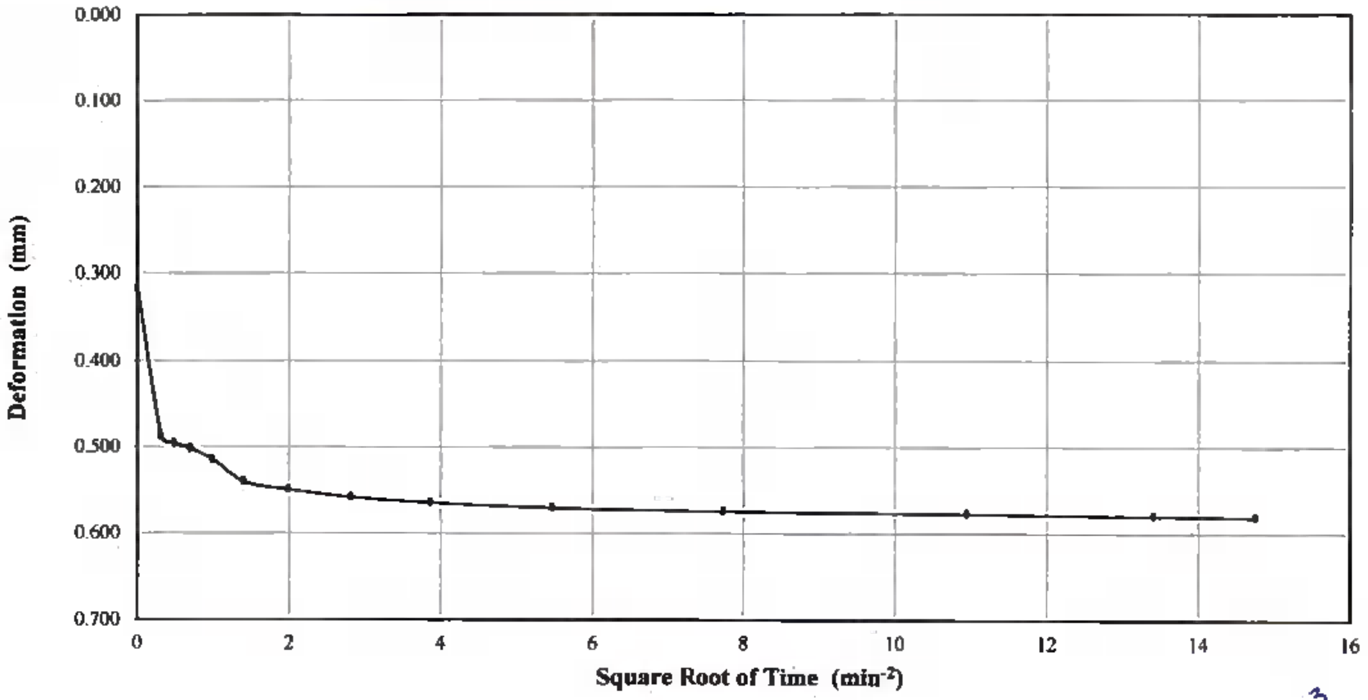
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf





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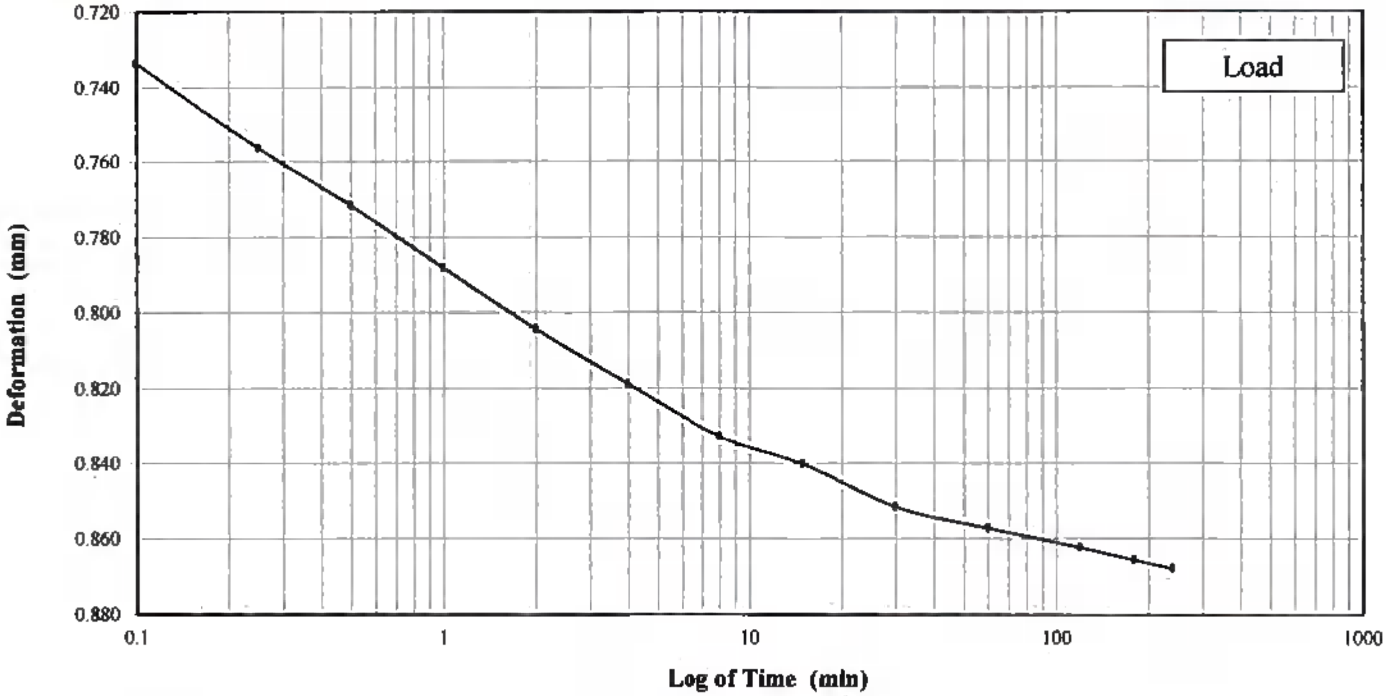
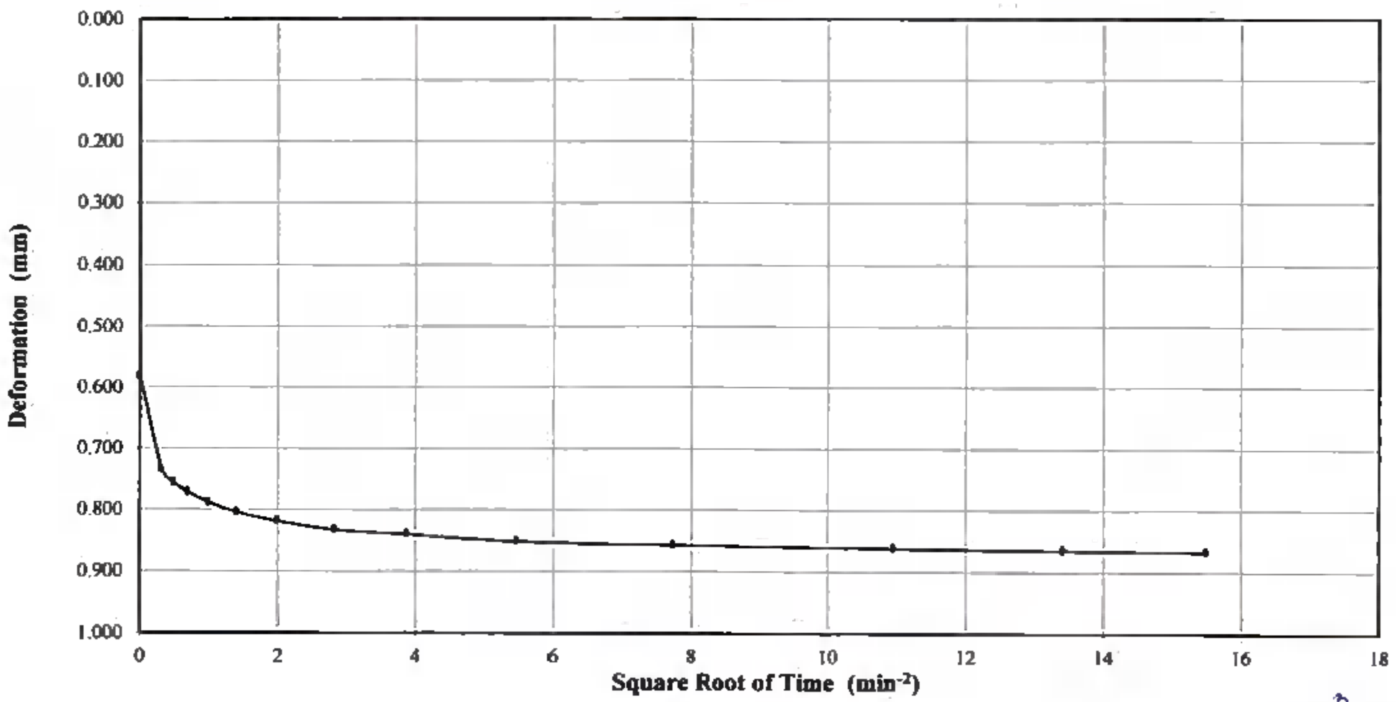
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 5 - 4000 psf





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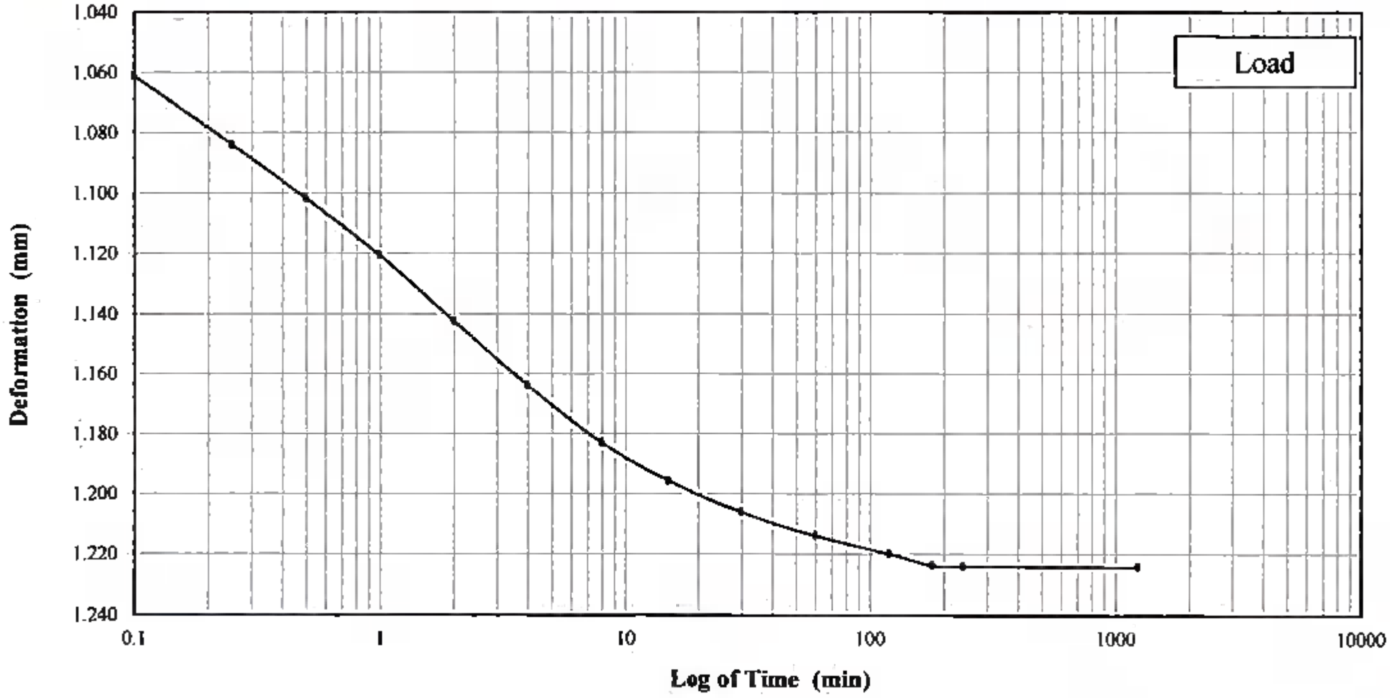
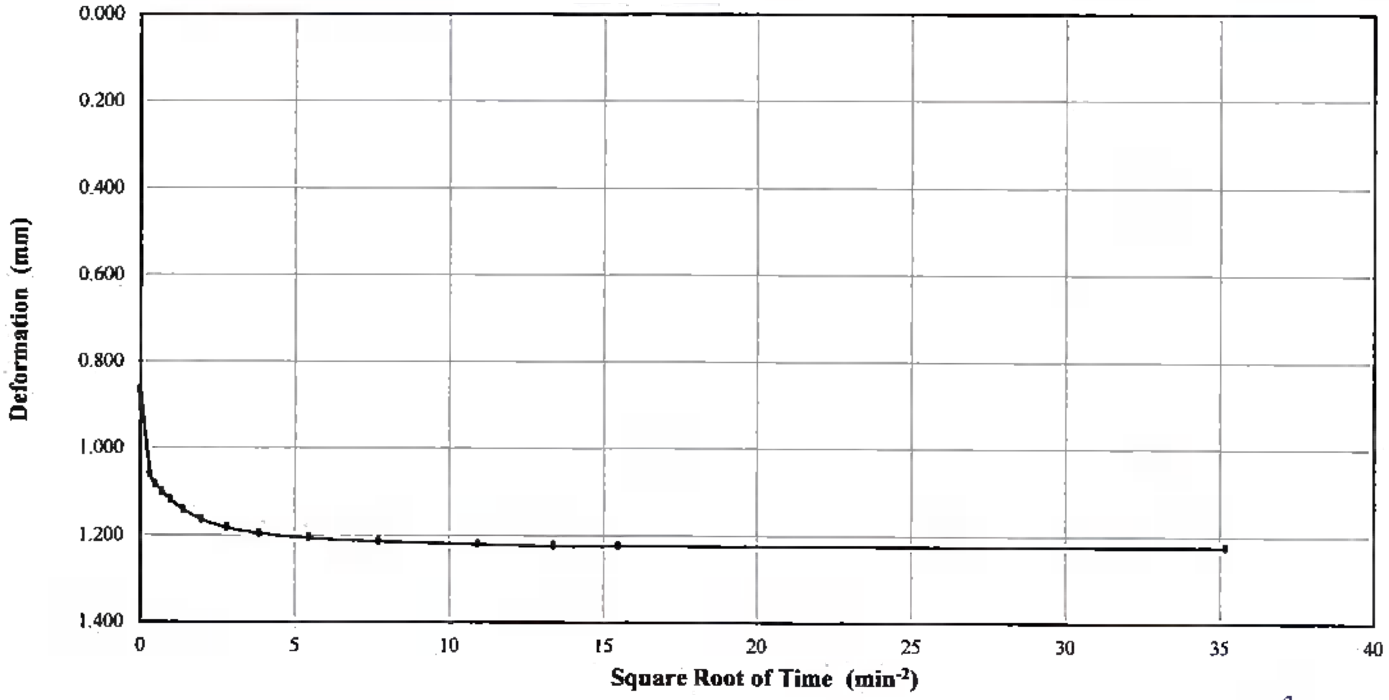
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 6 - 8000 psf





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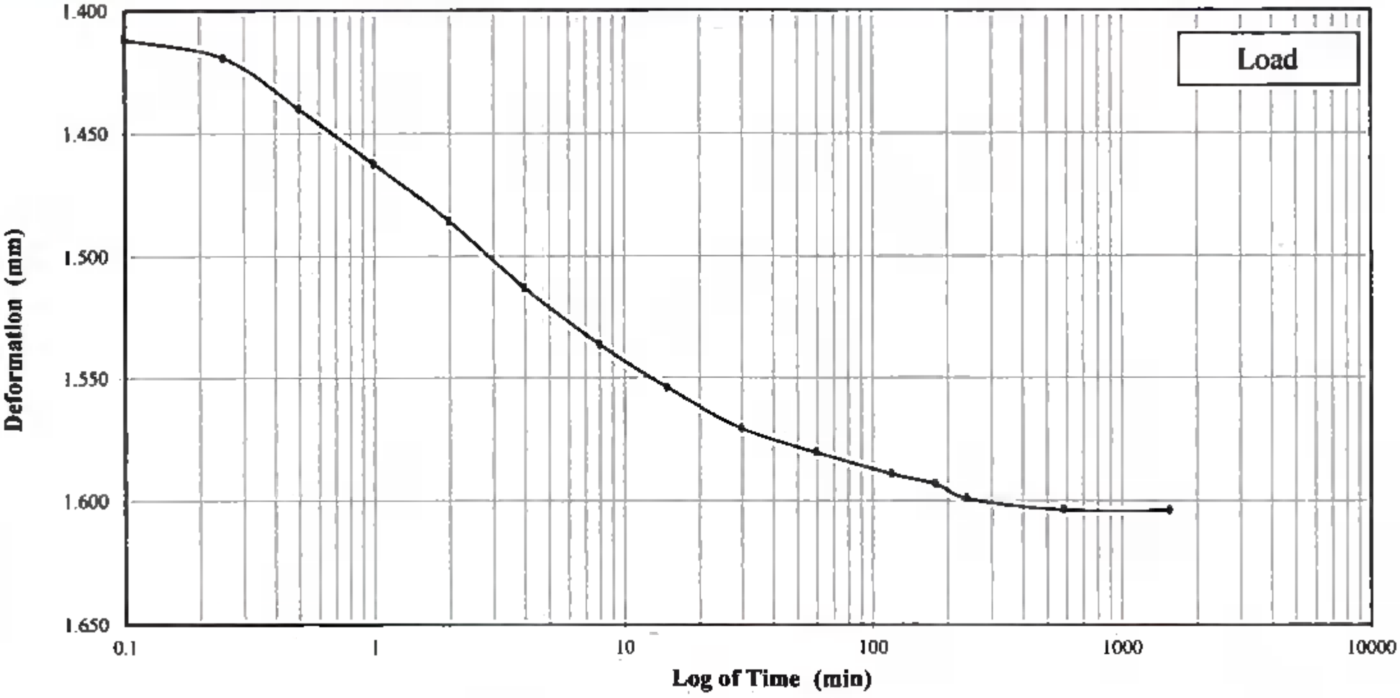
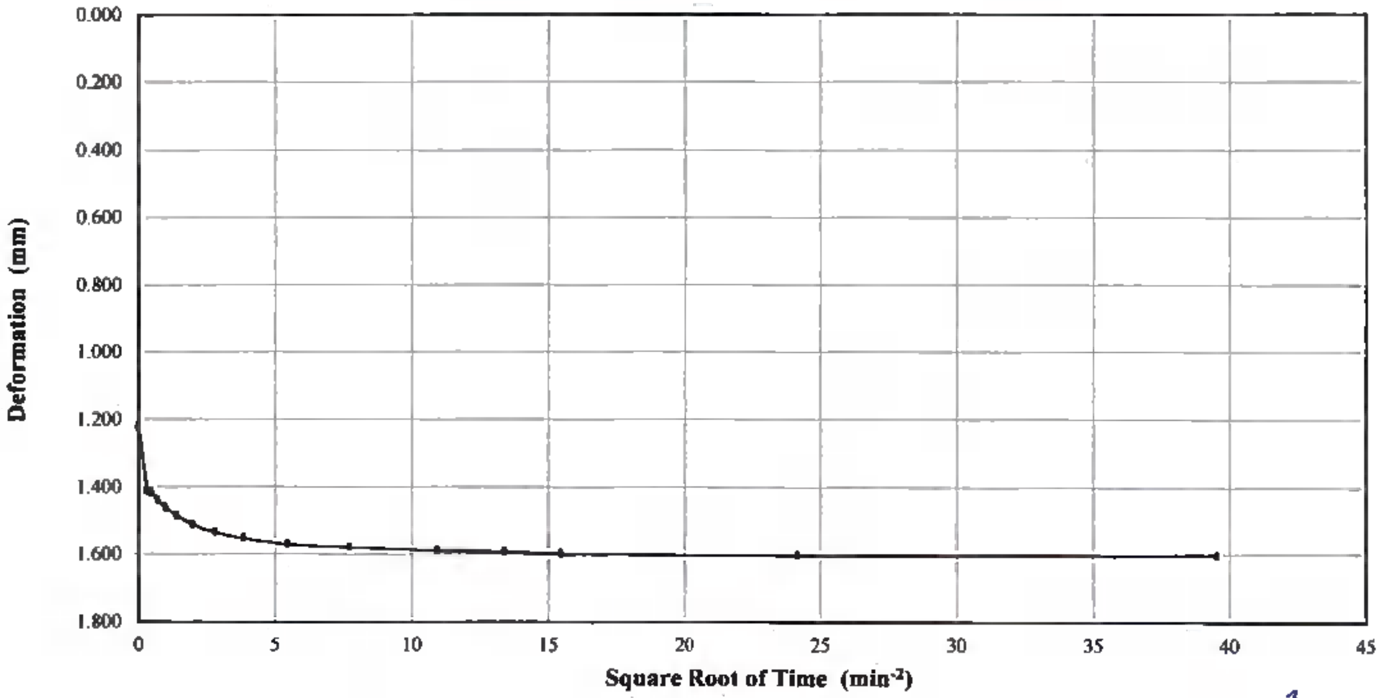
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 7 -16000 psf





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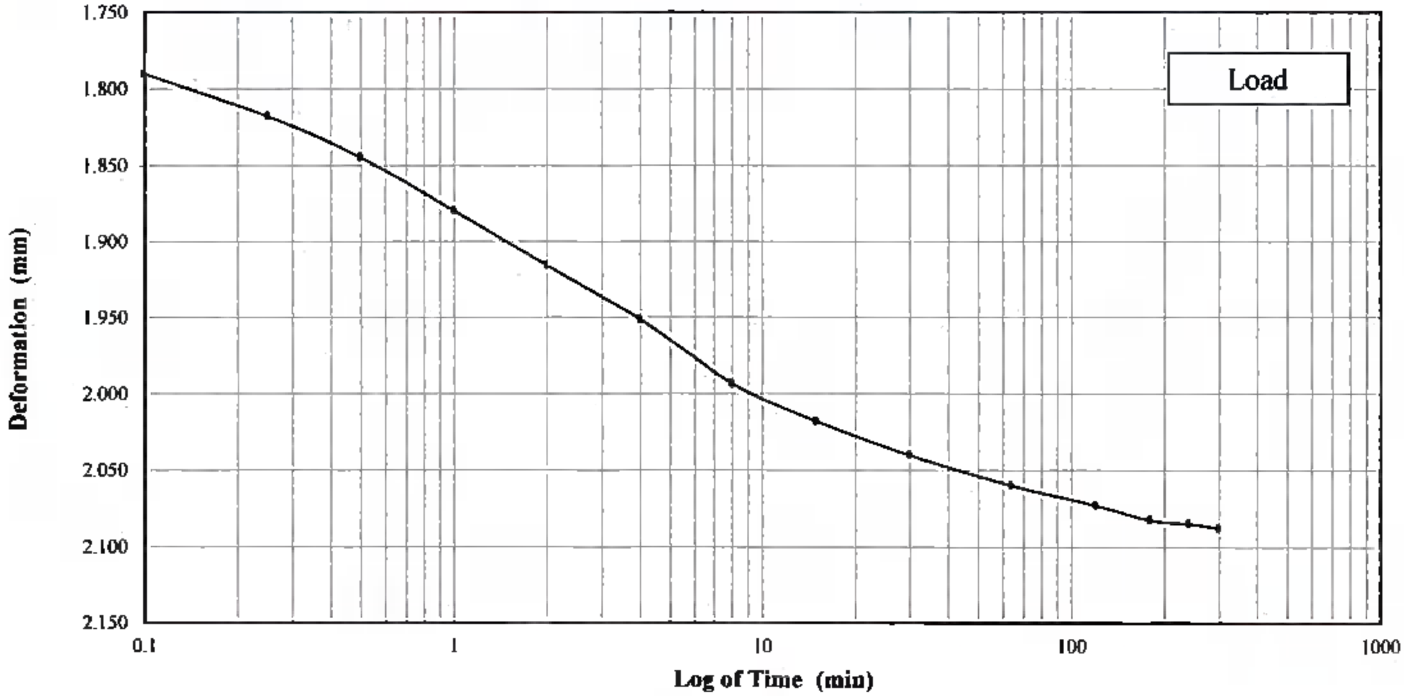
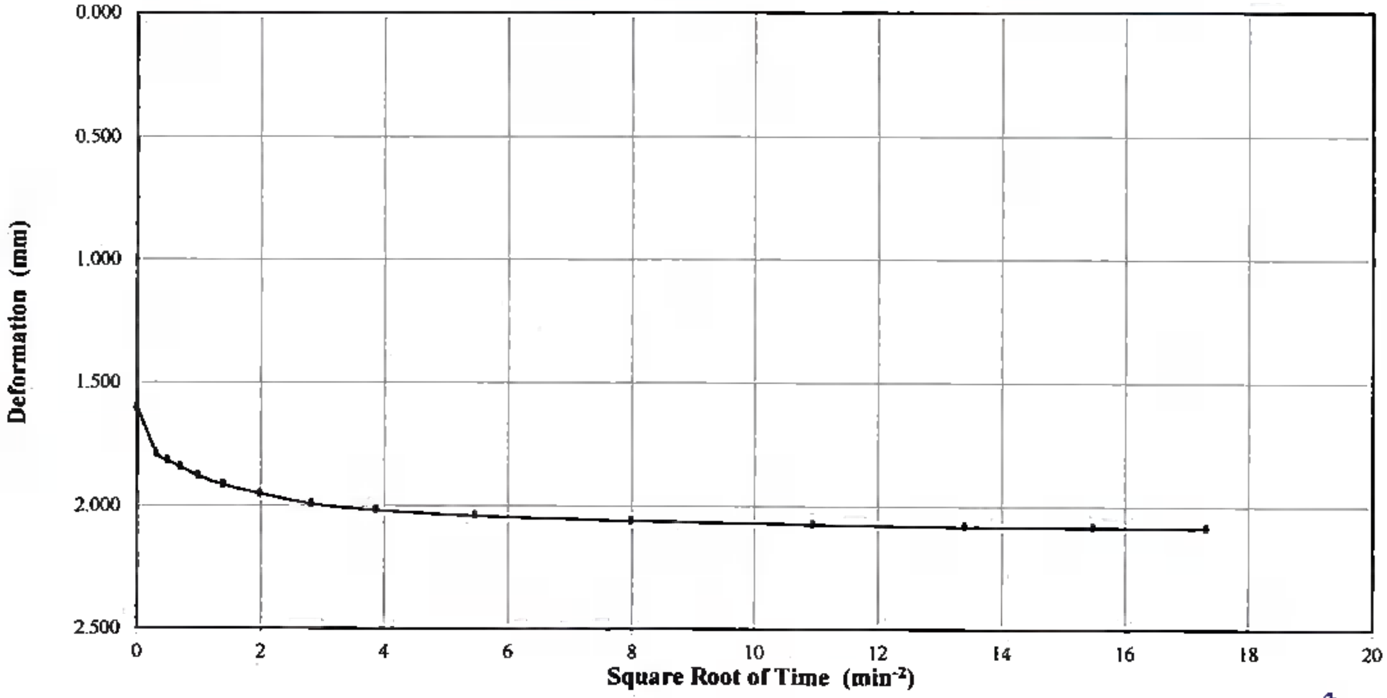
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 8 - 32000 psf





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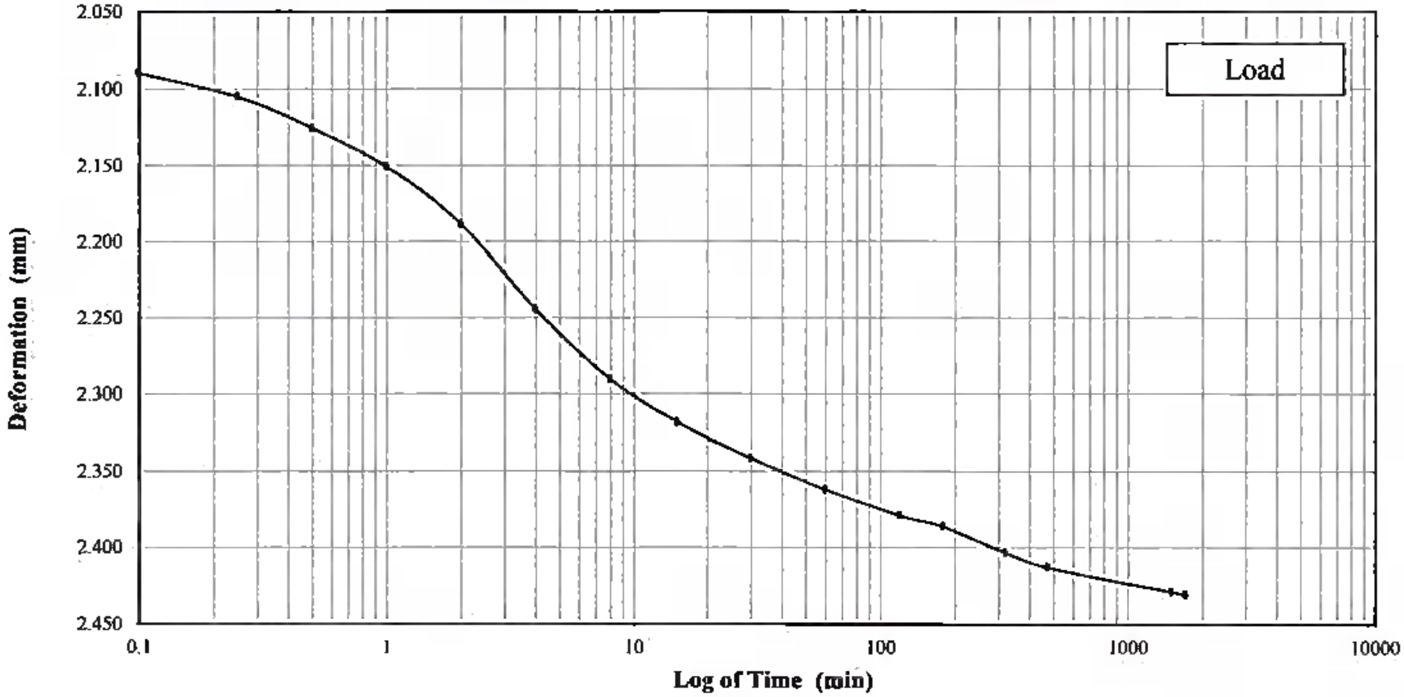
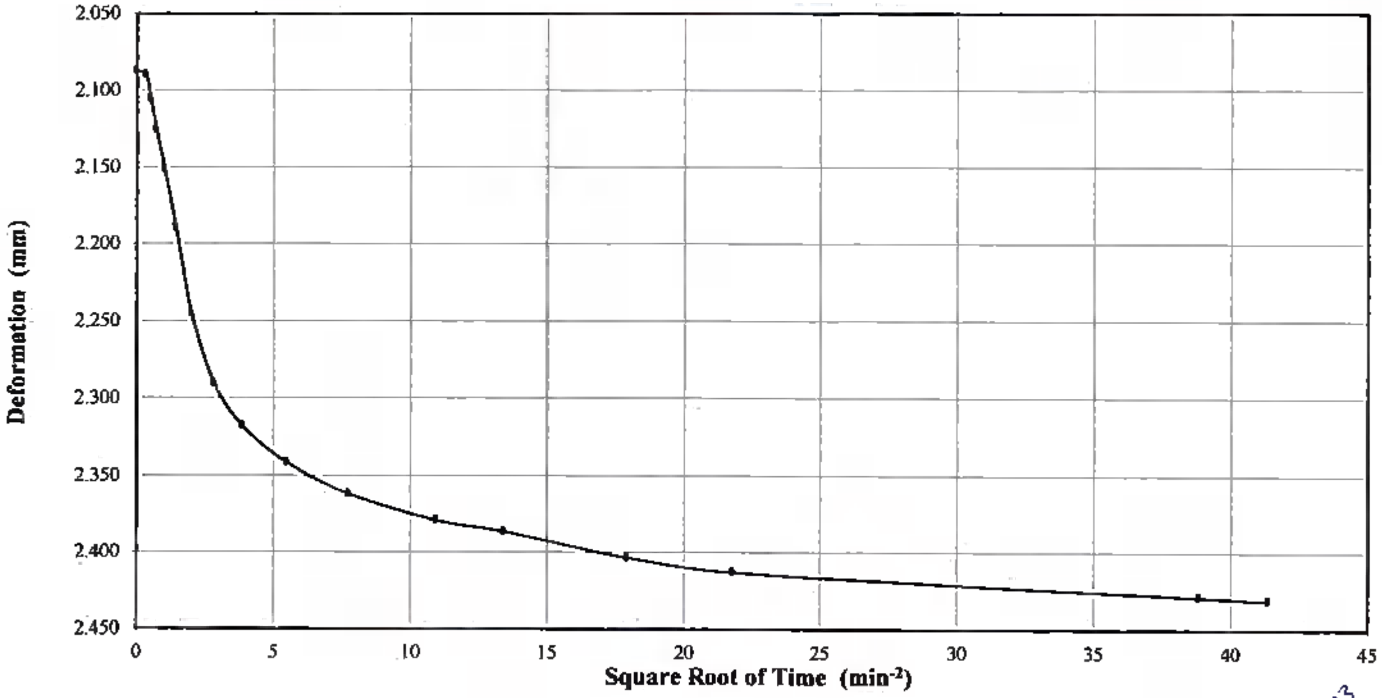
Project No: 618
Client Sample ID: SPT-109, ST-01 (21.5-23.5')
Lab Sample No: 13J361

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 9 - 48000 psf





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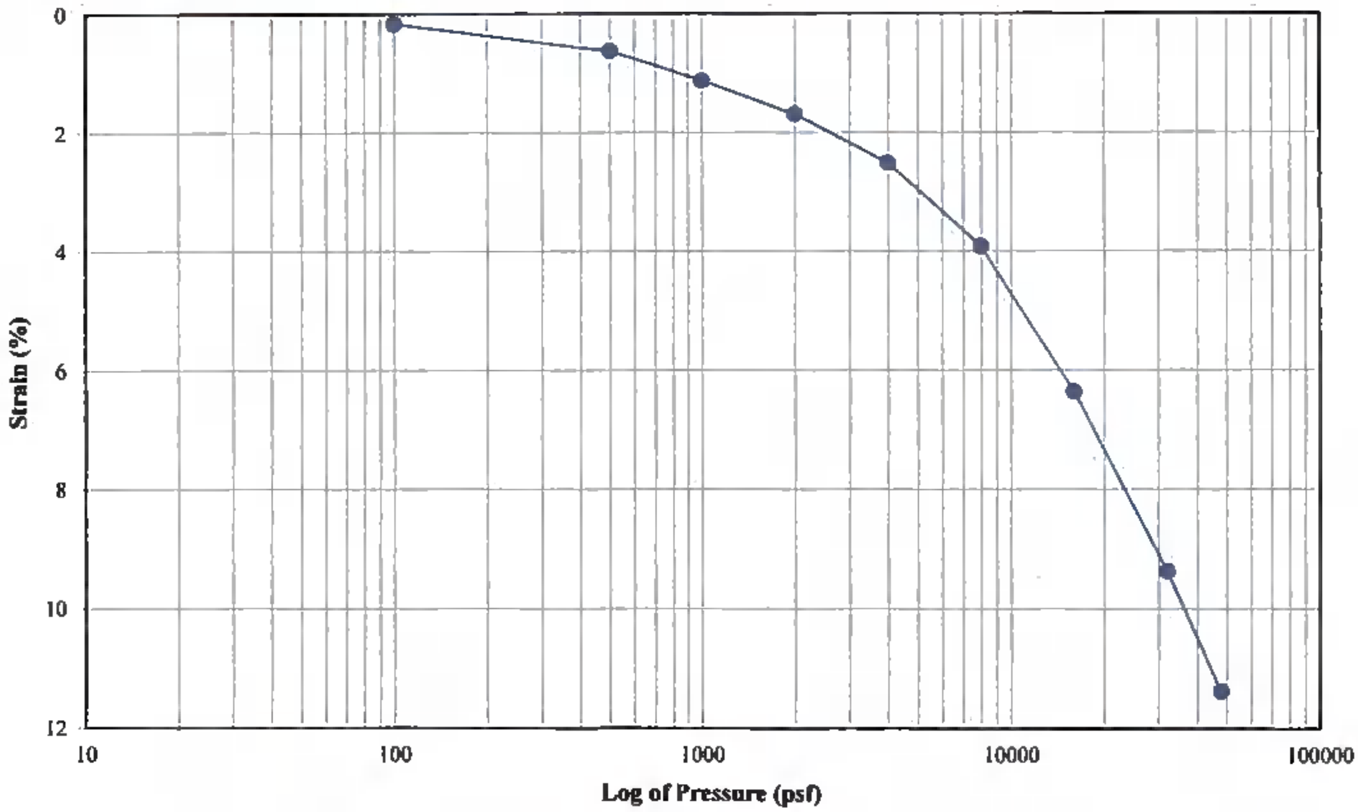
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST



Client Sample ID	Lab Sample No	Specimen Quality 1-10 (Bad to Good)	Test Specimen Initial Conditions				Consolidation Pressure (psf)	Pressure Increment Duration (min)	Accumu.[1] Vertical Strain (%)	Figure No.	Remarks
			Height (cm)	Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)					
SPT-110, ST-02 (36.5-38.5')	13J364	4	2.54	6.35	90.2	31.7	100	1247	0.16	1	
							500	240	0.62	2	
							1000	1186	1.12	3	
							2000	240	1.68	4	
							4000	1204	2.51	5	
							8000	1691	3.92	6	
							16000	1210	6.36	7	
							32000	1414	9.37	8	
							48000	1496	11.37	9	

Notes:

For each pressure increment, the vertical strain values were calculated based on the final deformation measurements.

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953 Forrest Street, Roswell, Georgia 30075
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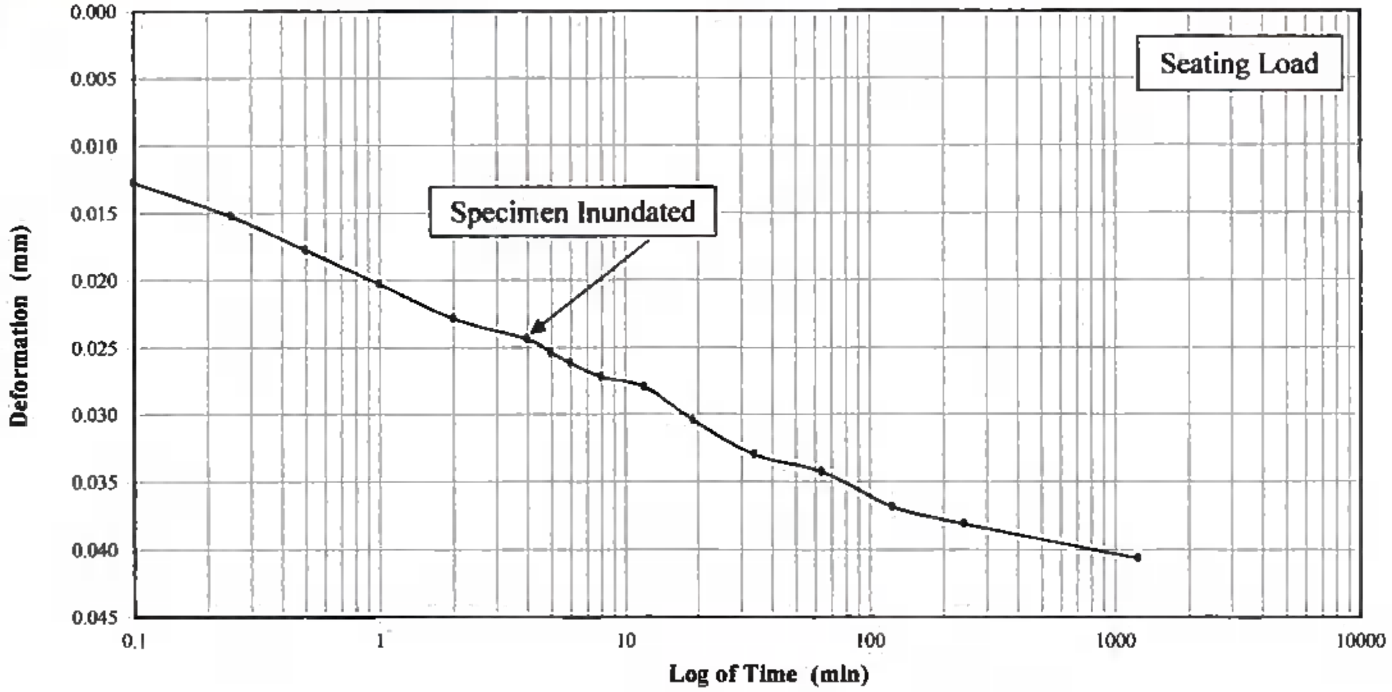
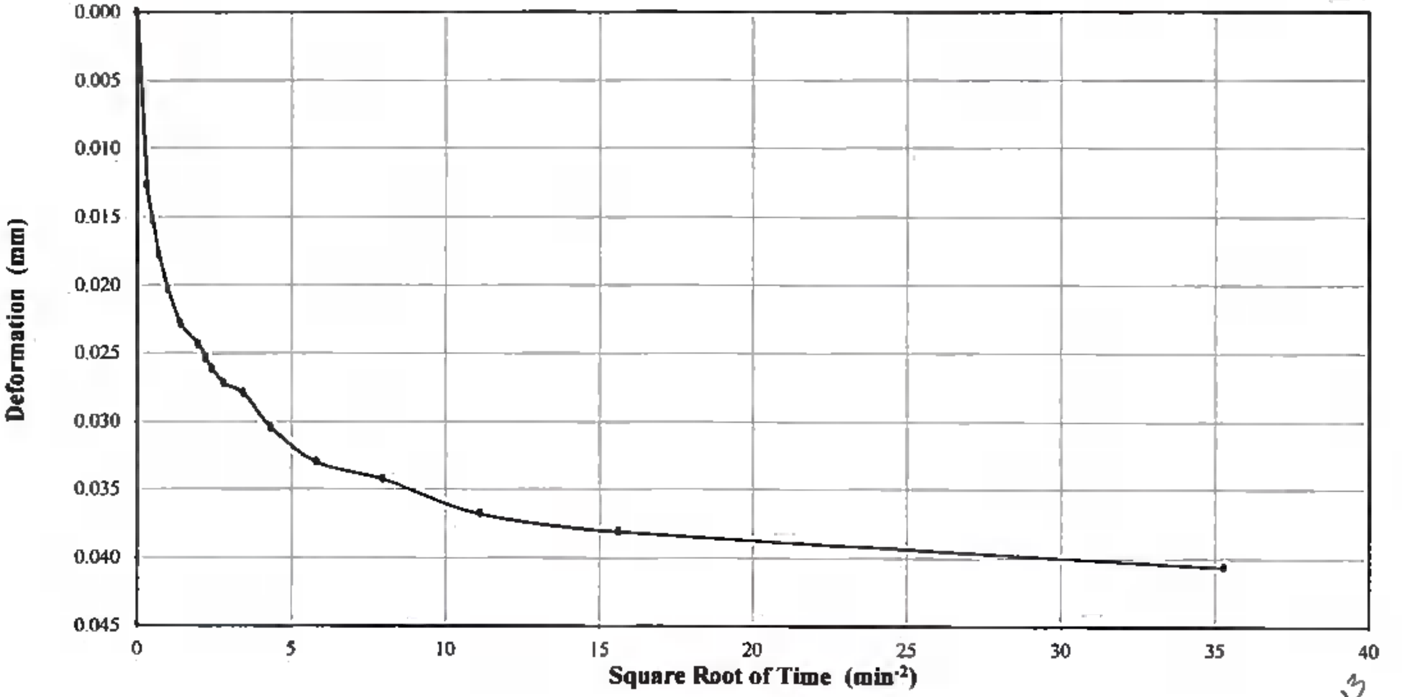
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-110, ST-02 (36.5-38.5')
Lab Sample No:	13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





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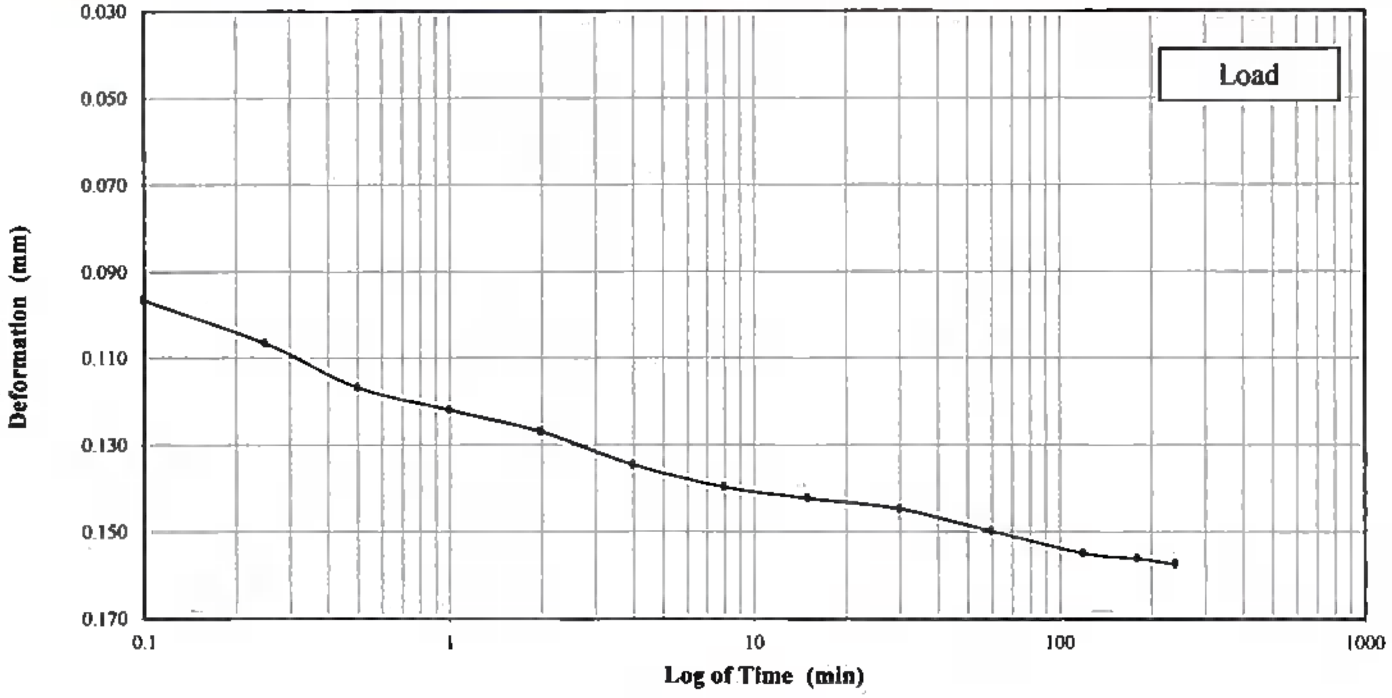
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

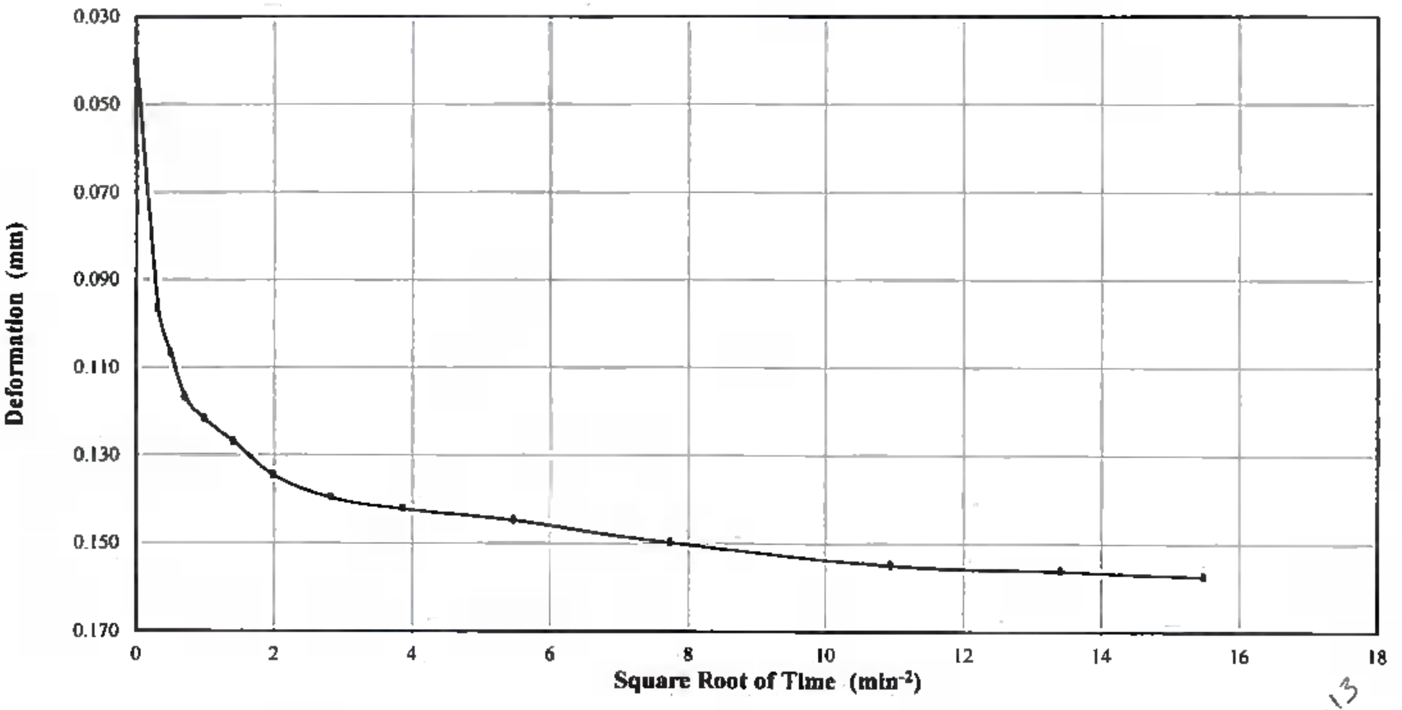
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf





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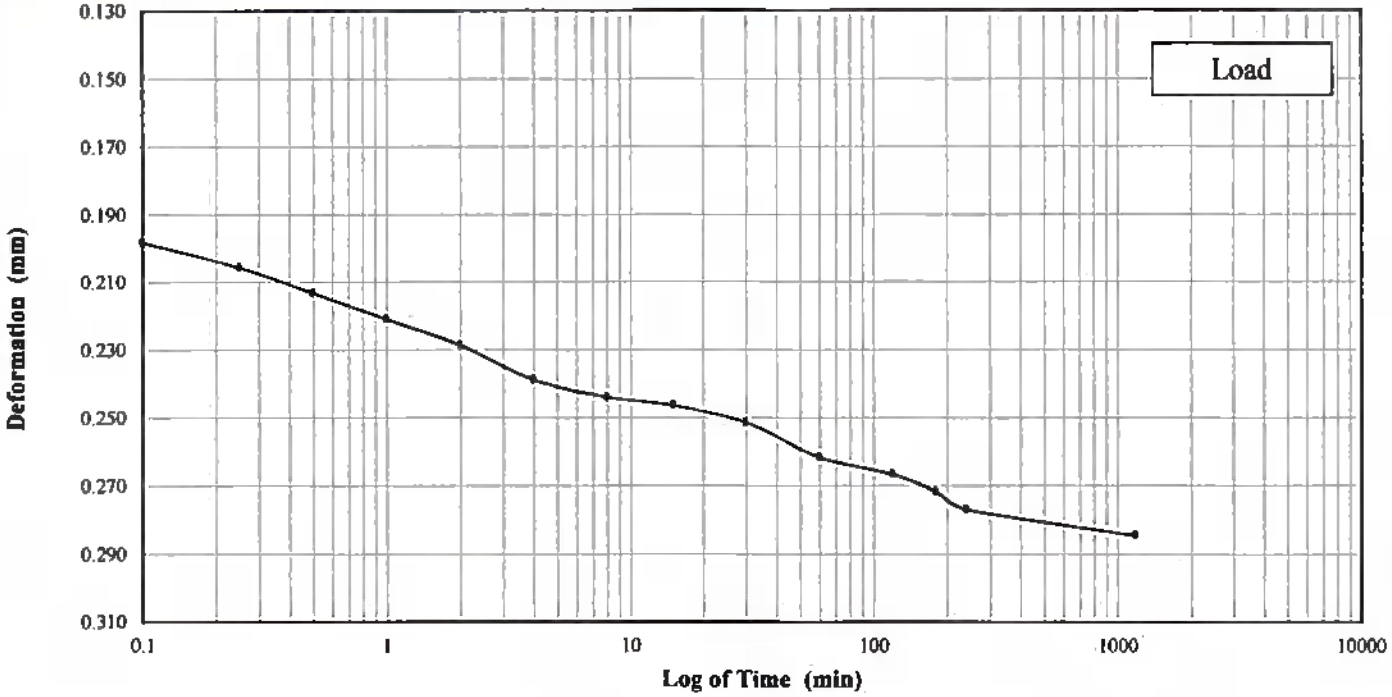
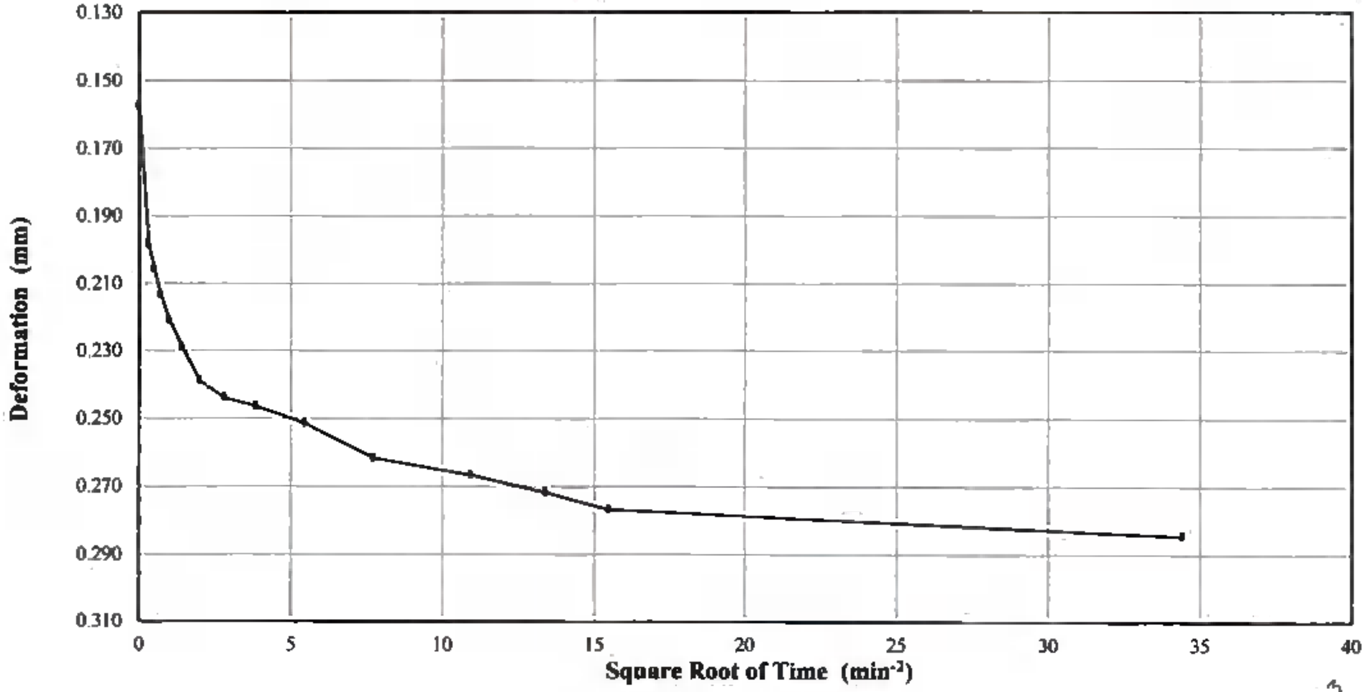
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 3 - 1000 psf





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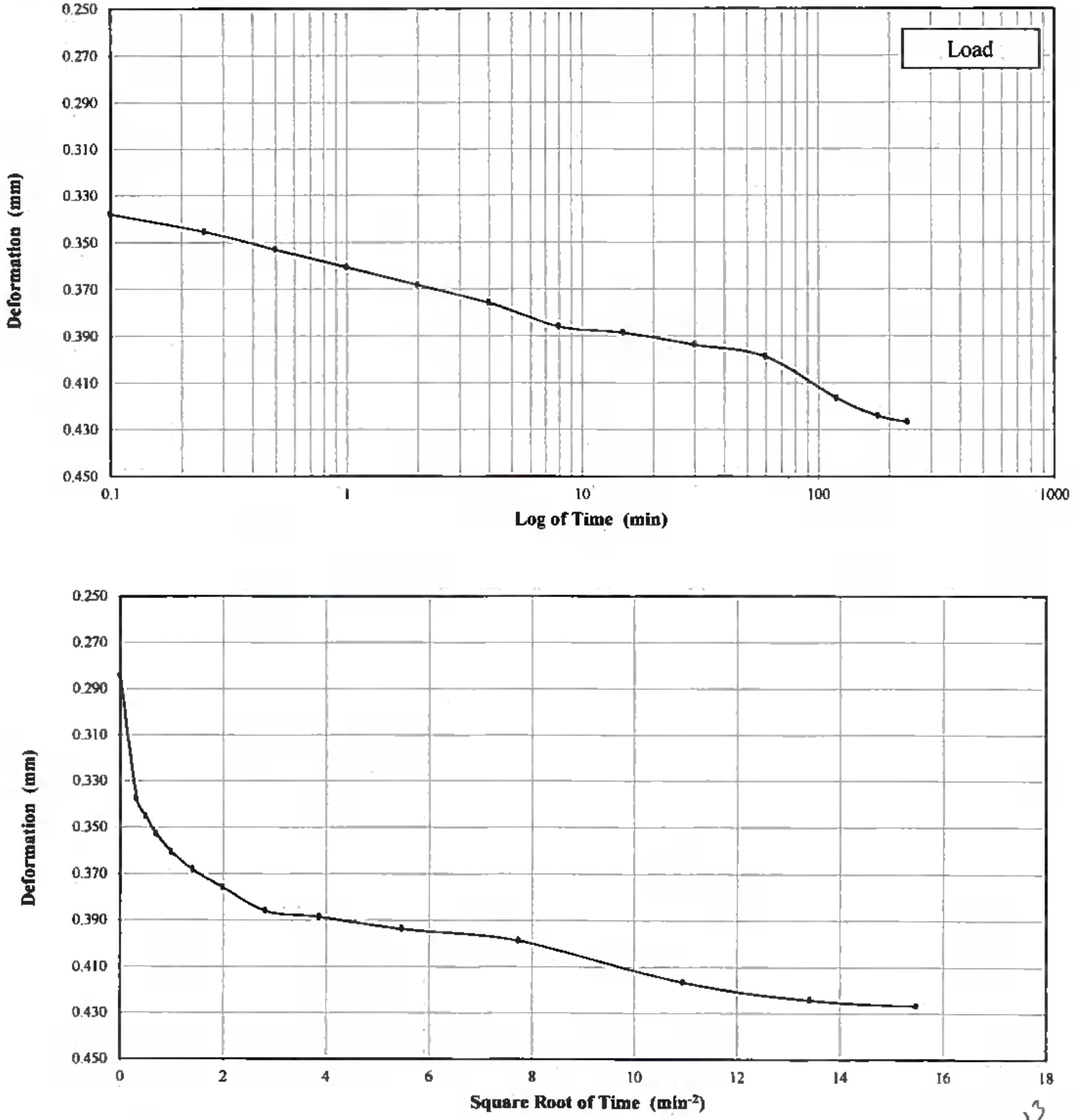
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf



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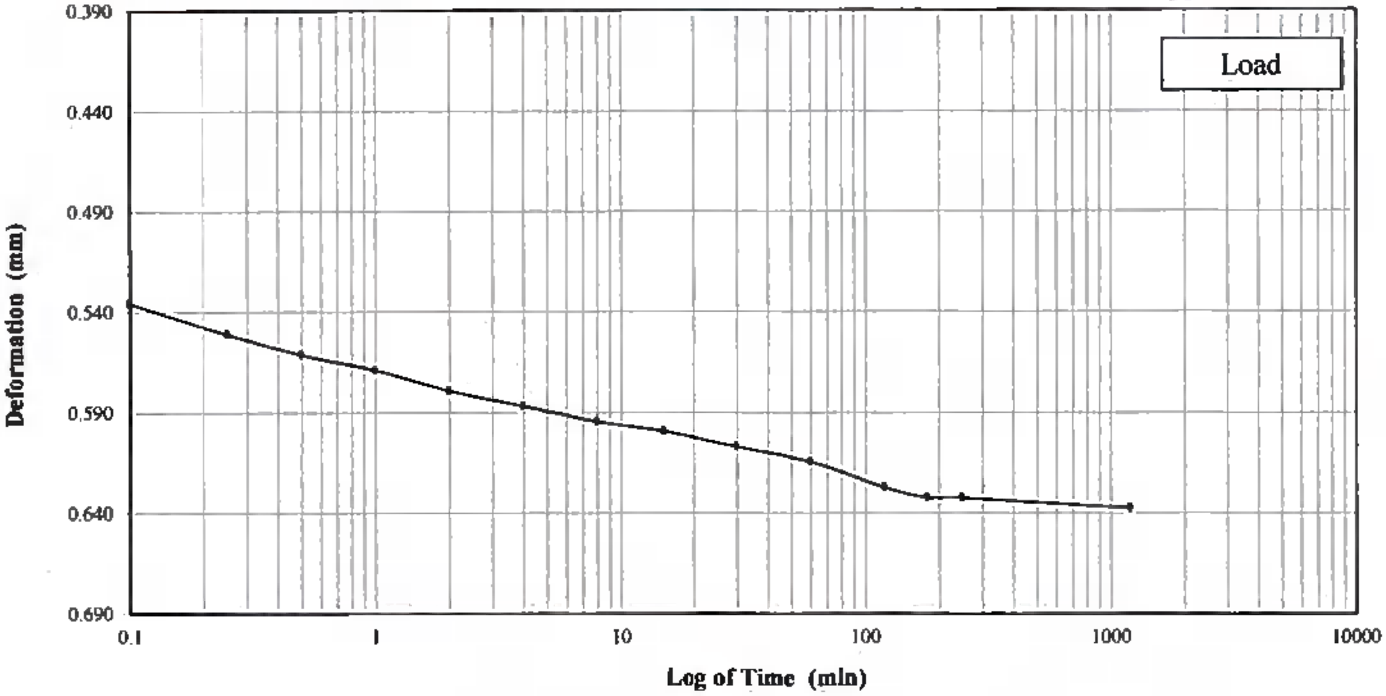
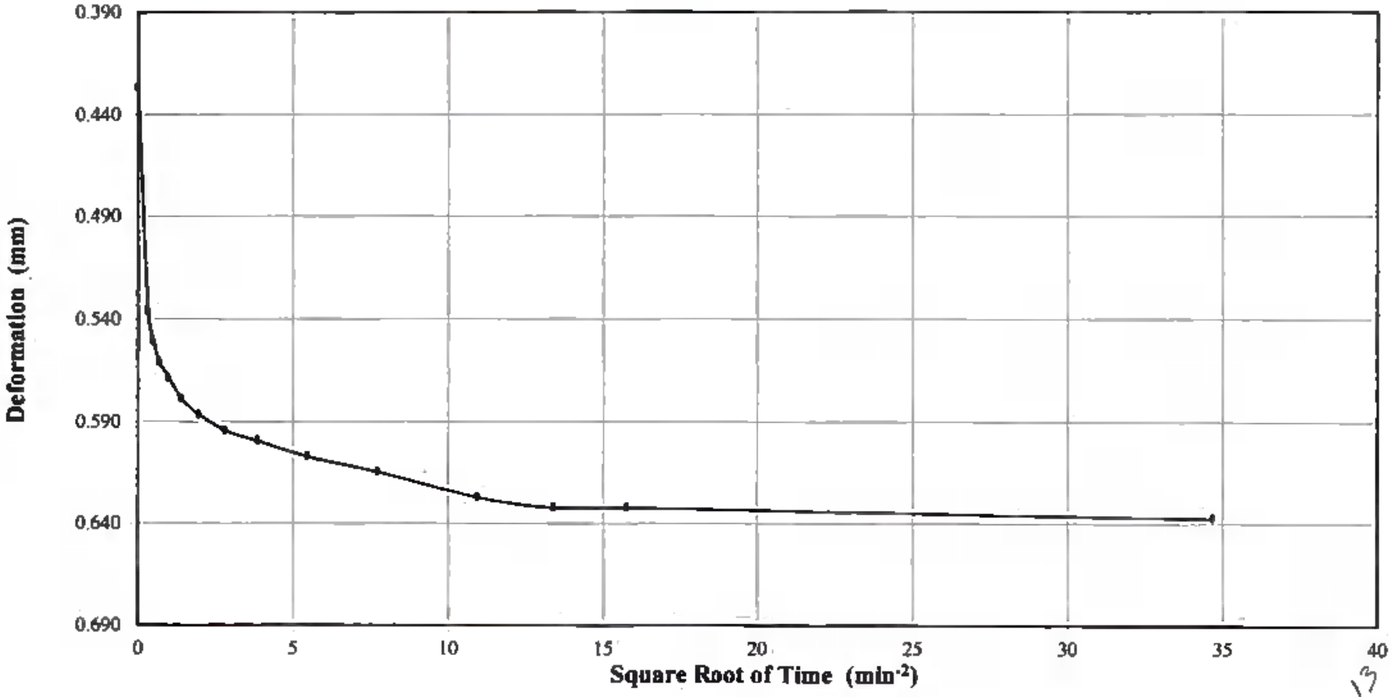
Project No:	618
Client Sample ID:	SPT-110, ST-02 (36.5-38.5')
Lab Sample No:	13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 5 - 4000 psf





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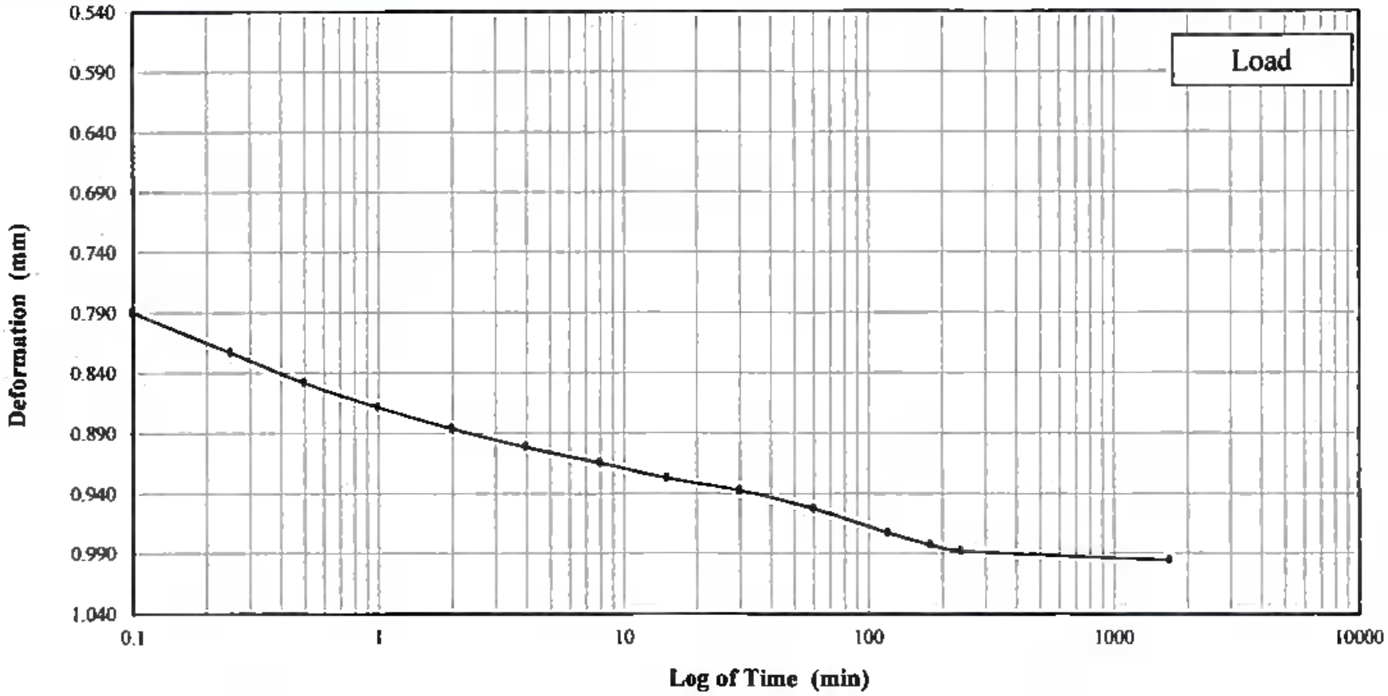
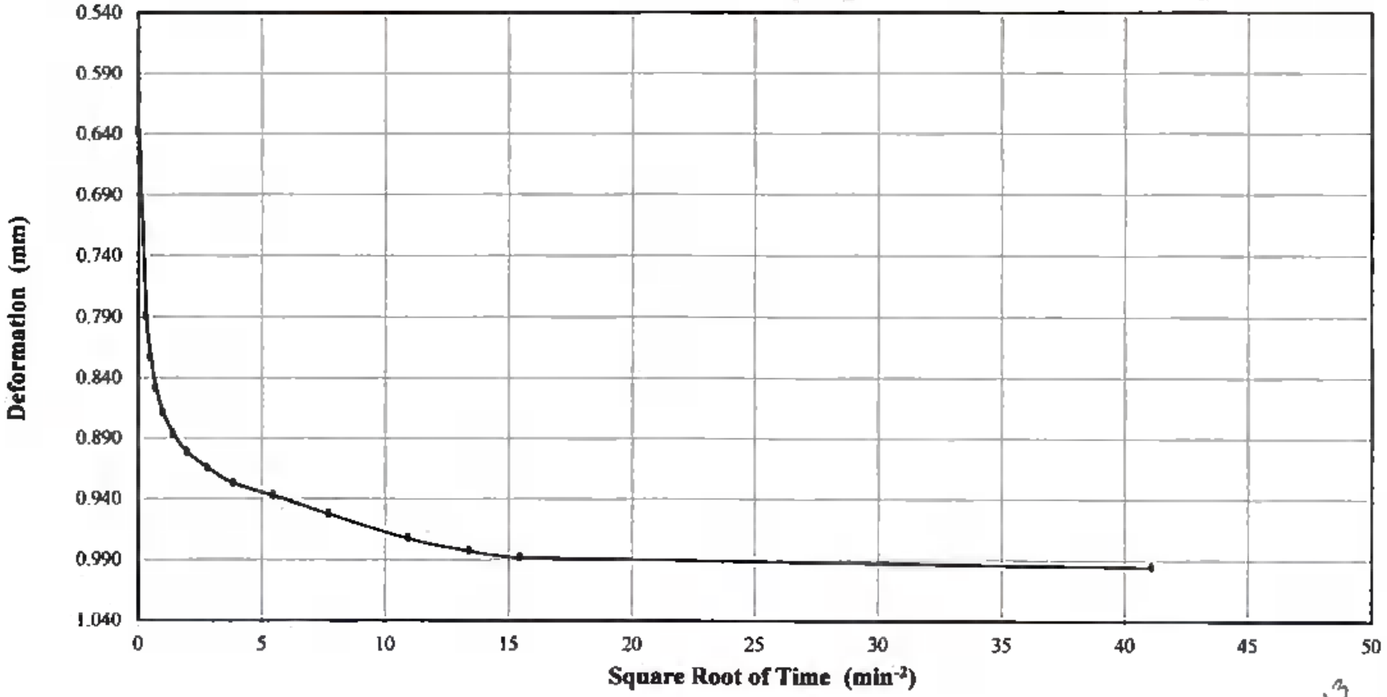
Project No:	618
Client Sample ID:	SPT-110, ST-02 (36.5-38.5')
Lab Sample No:	13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 6 - 8000 psf





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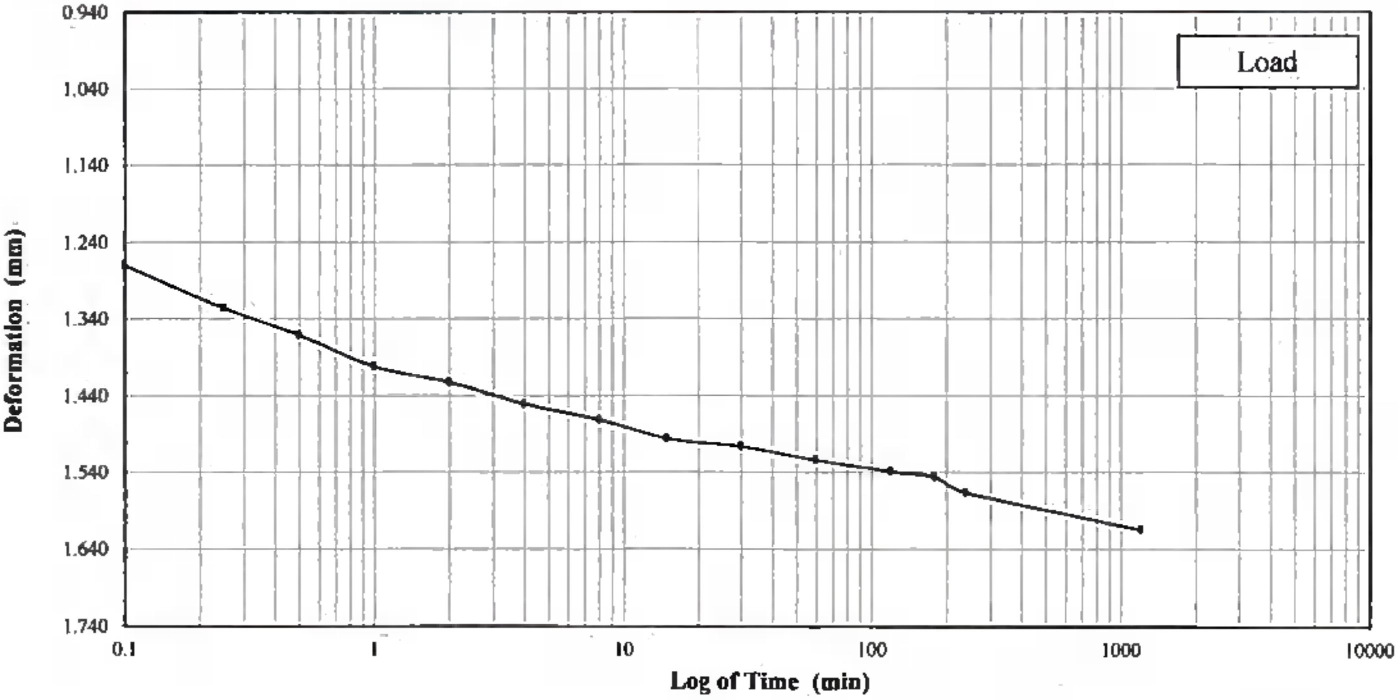
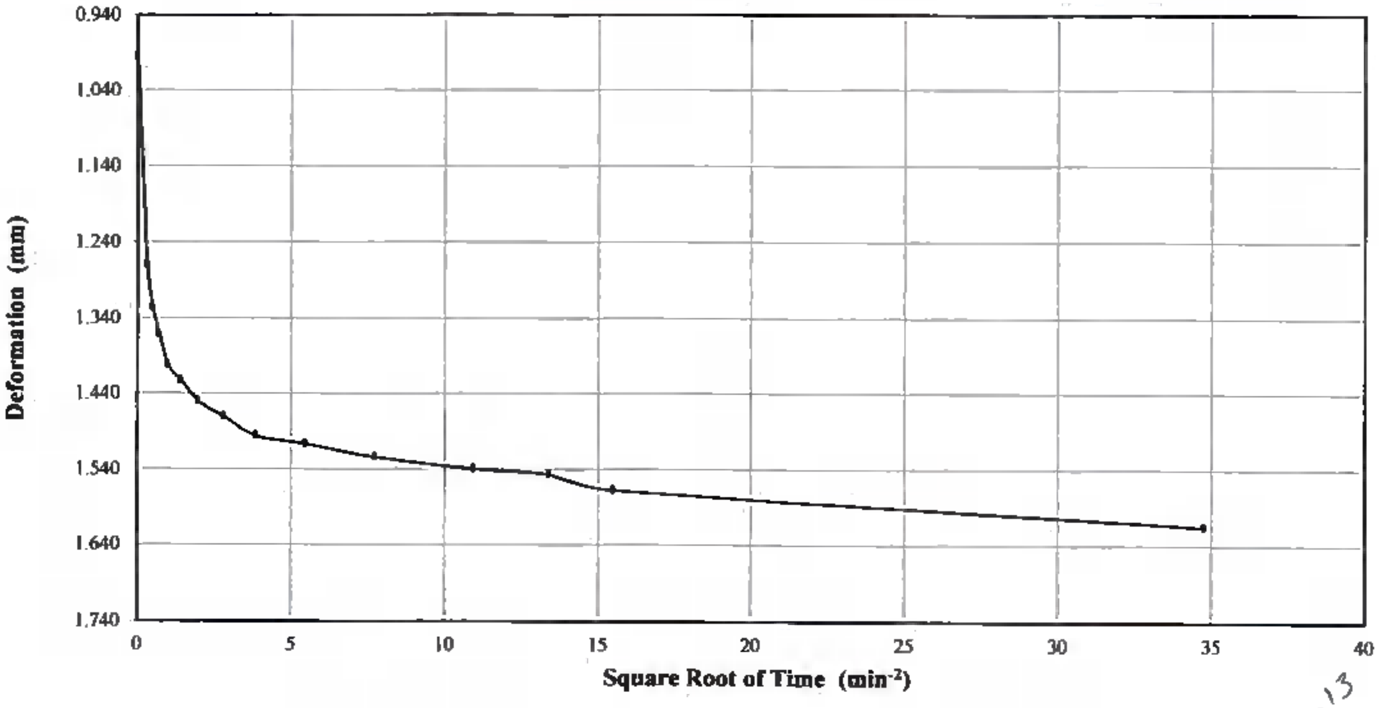
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 7 - 16000 psf





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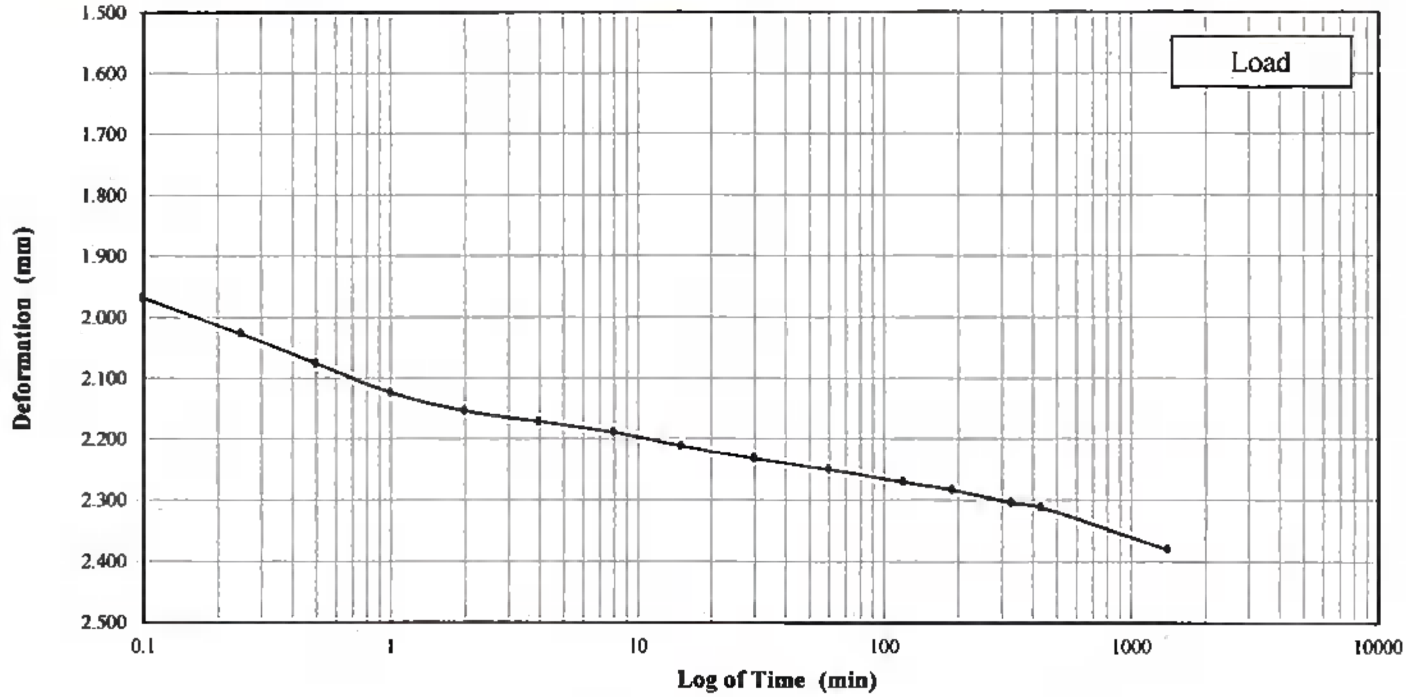
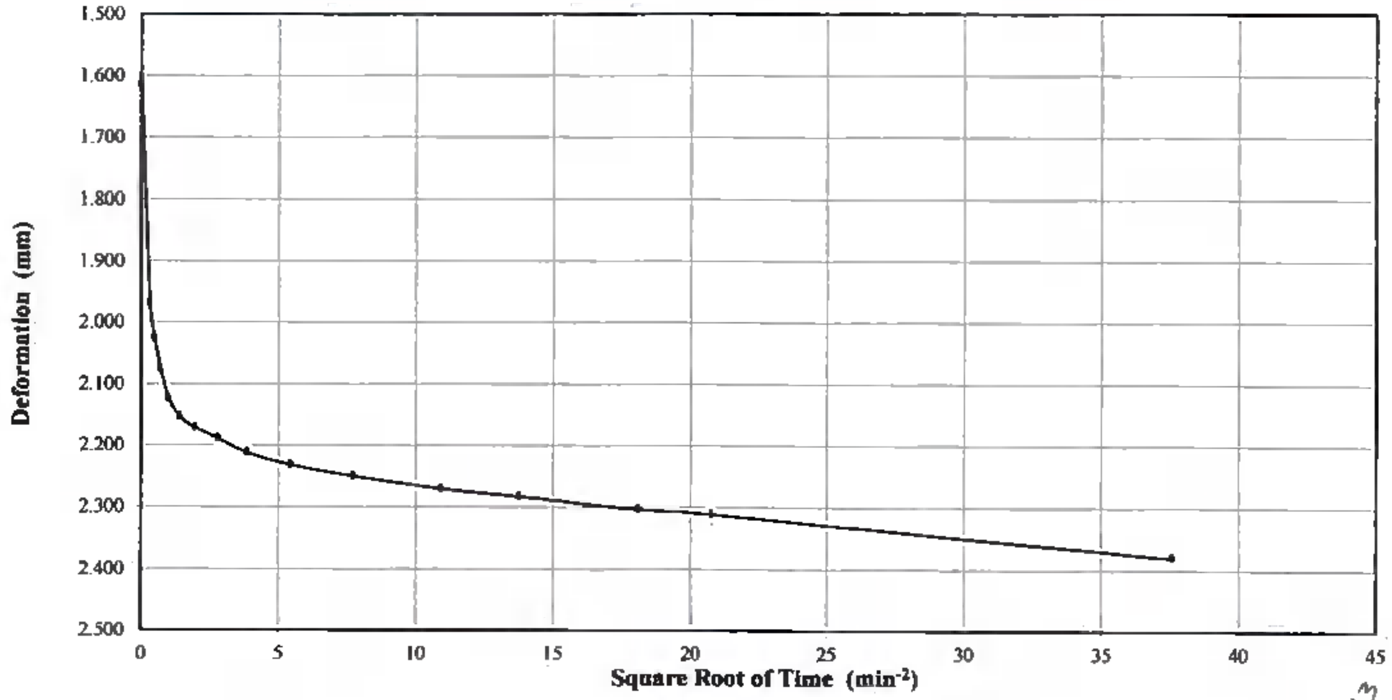
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 8 - 32000 psf





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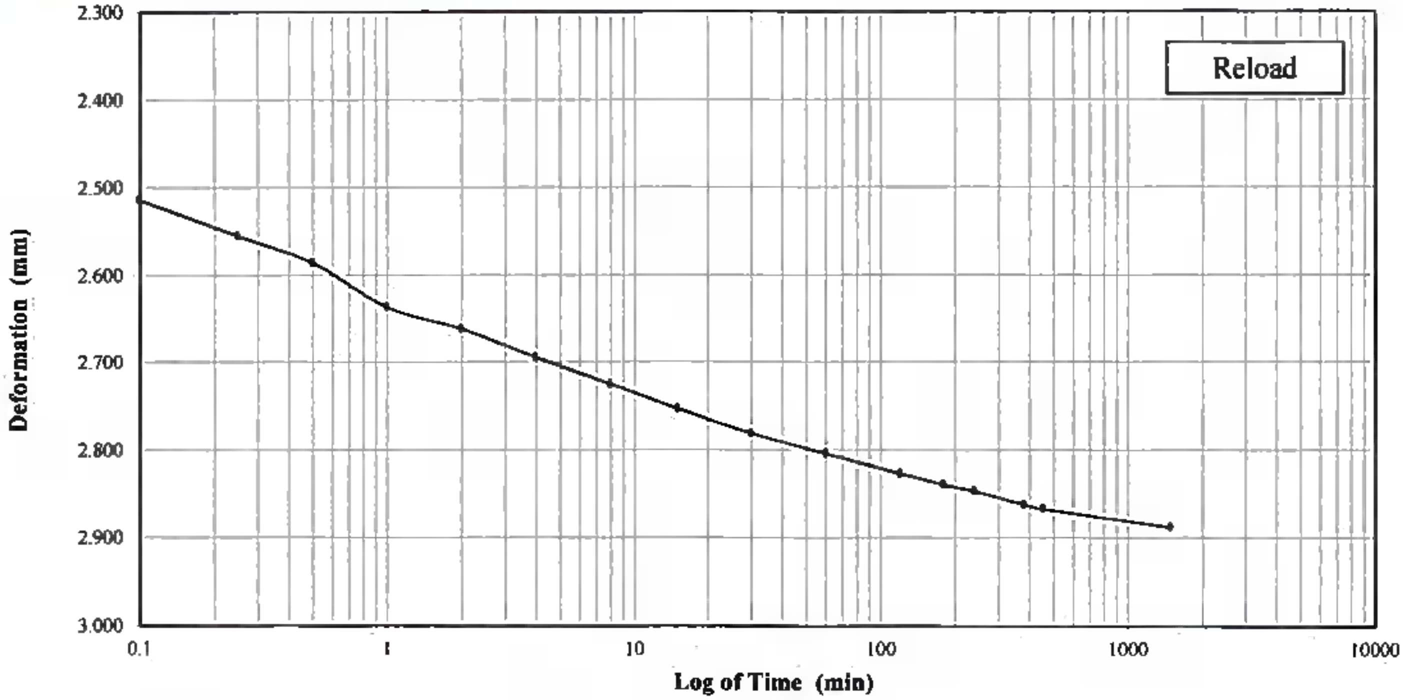
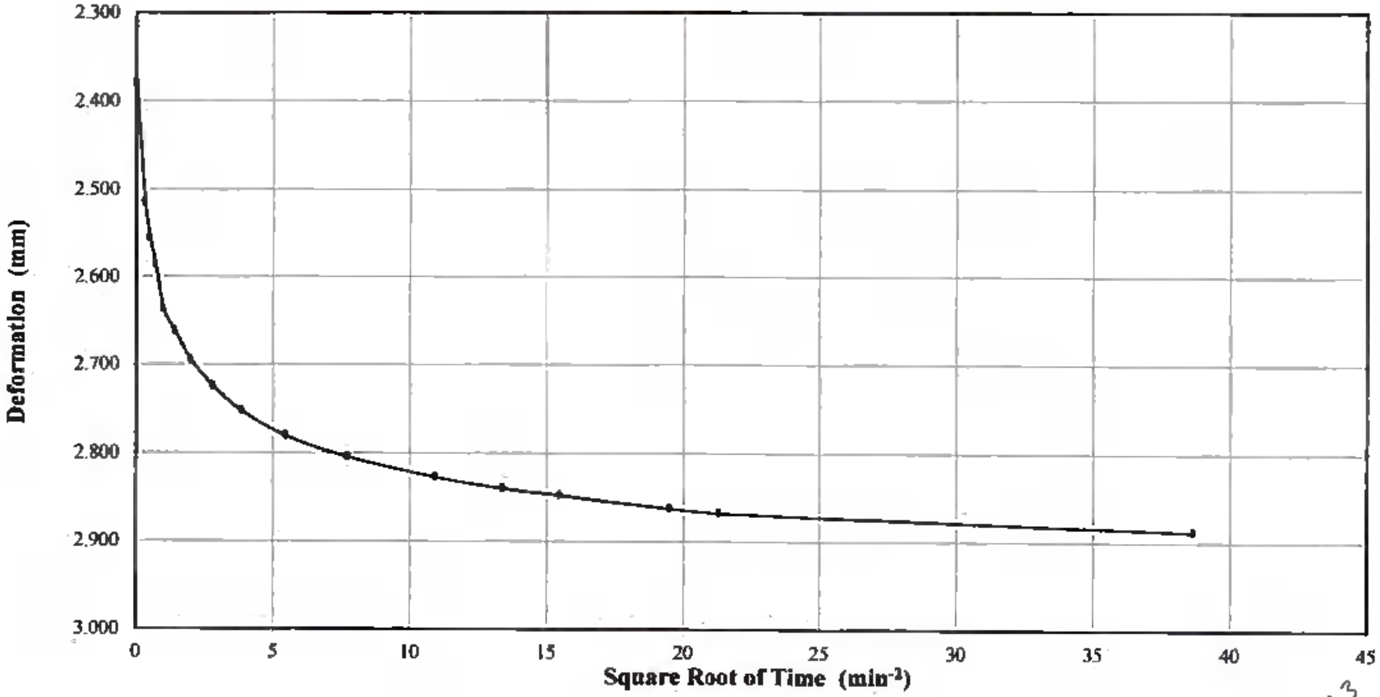
Project No: 618
Client Sample ID: SPT-110, ST-02 (36.5-38.5')
Lab Sample No: 13J364

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 9 - 48000 psf





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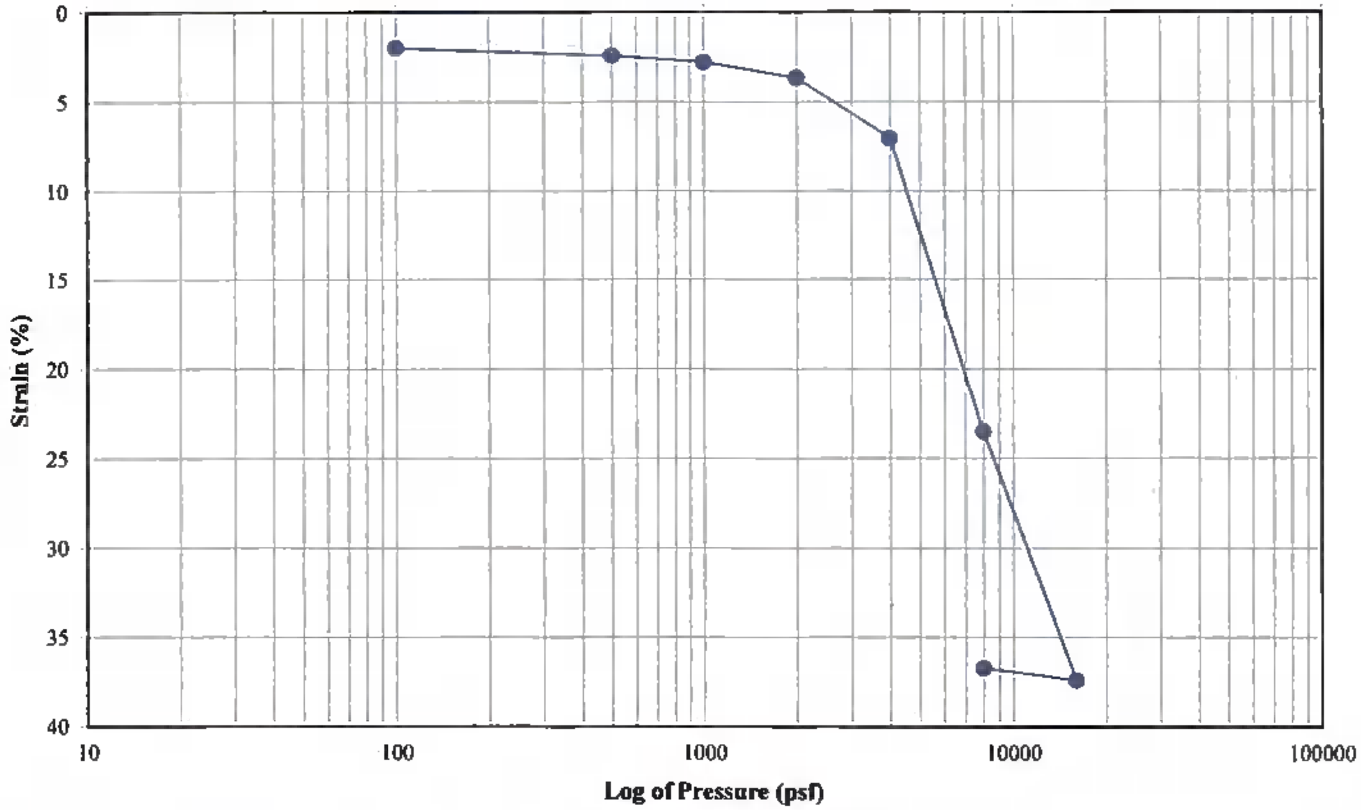
"Excellence in Testing"

953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST



Client Sample ID	Lab Sample No.	Specimen Quality 1-10 (Bad to Good)	Test Specimen Initial Conditions				Consolidation Pressure (psf)	Pressure Increment Duration (min)	Accumu.(1) Vertical Strain (%)	Figure No.	Remarks
			Height (cm)	Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)					
SPT-112, ST-03 (46.5-48.5')	13J367	5	2.54	6.35	42.1	107.0	100	1136	1.96	1	
							500	1826	2.39	2	
							1000	2966	2.76	3	
							2000	1473	3.62	4	
							4000	1295	7.02	5	
							8000	1526	23.47	6	
							16000	2808	37.38	7	
							8000	1440	36.70	8	

Notes:

For each pressure increment, the vertical strain values were calculeted based on the final deformation measurements.

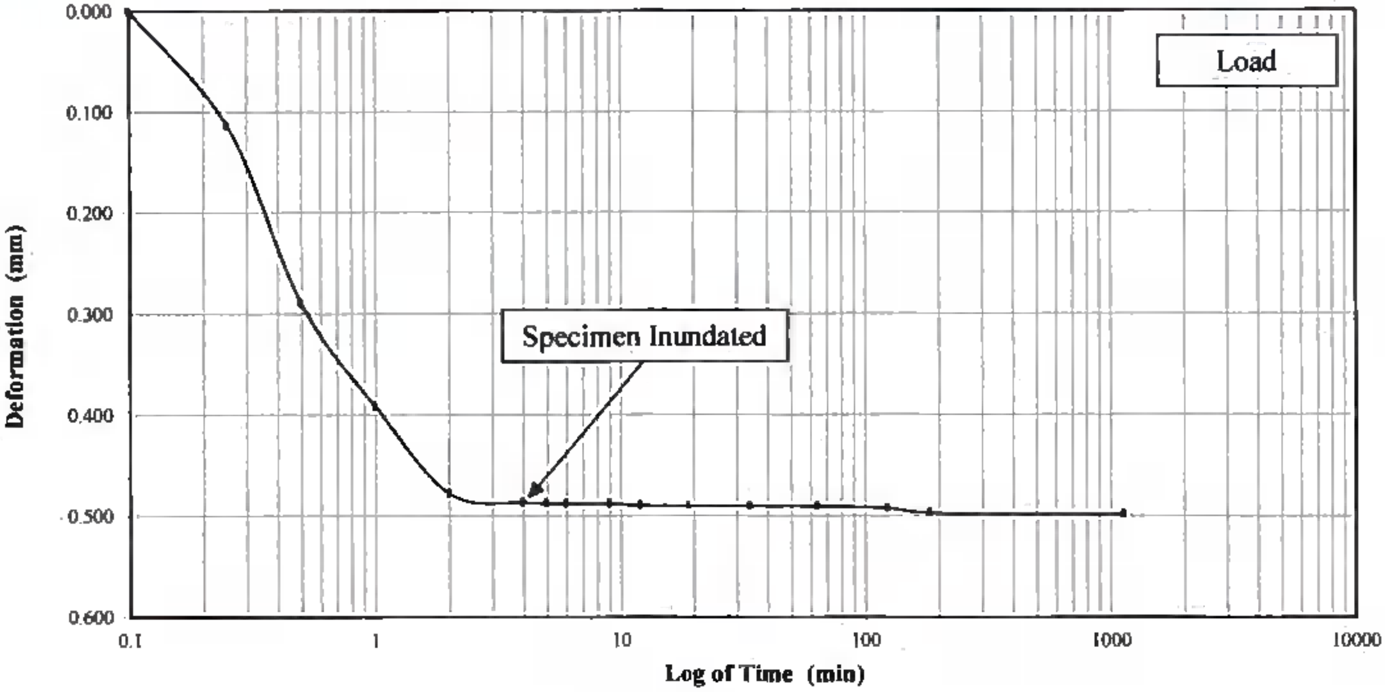
1-16-14
NJR

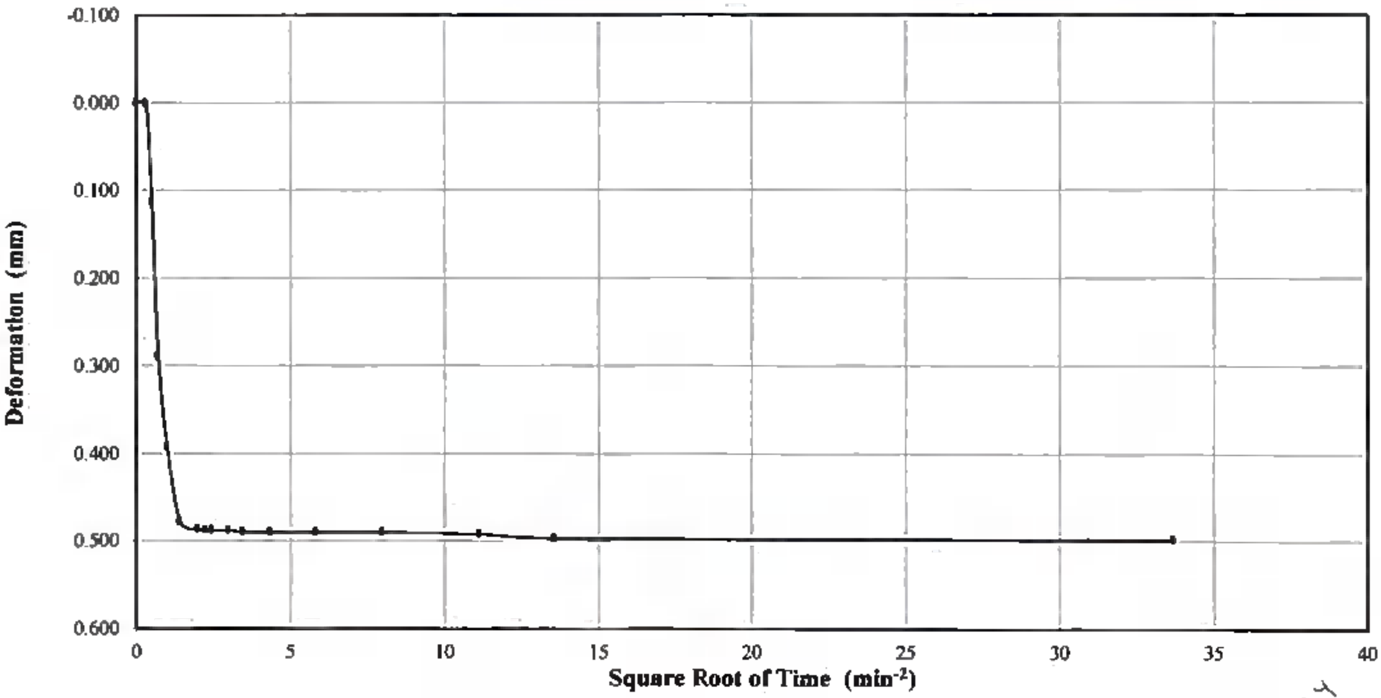
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





1-16-14
NJK

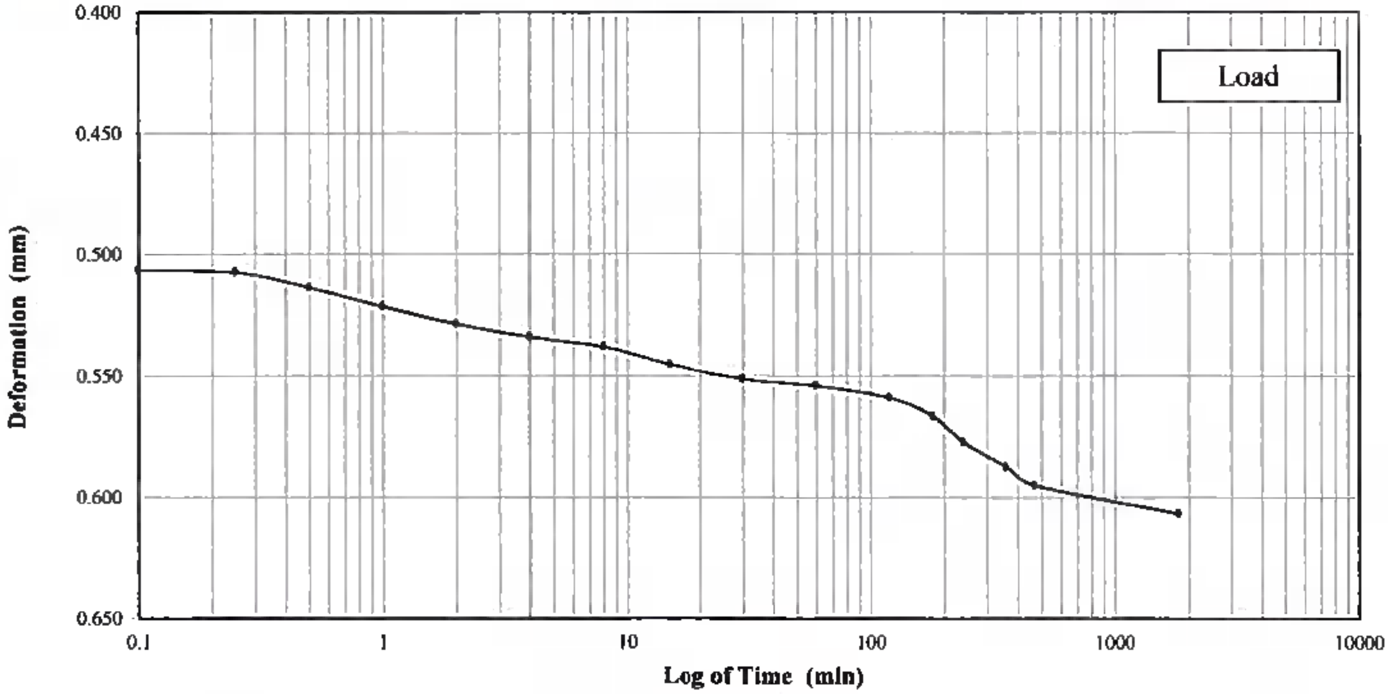
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

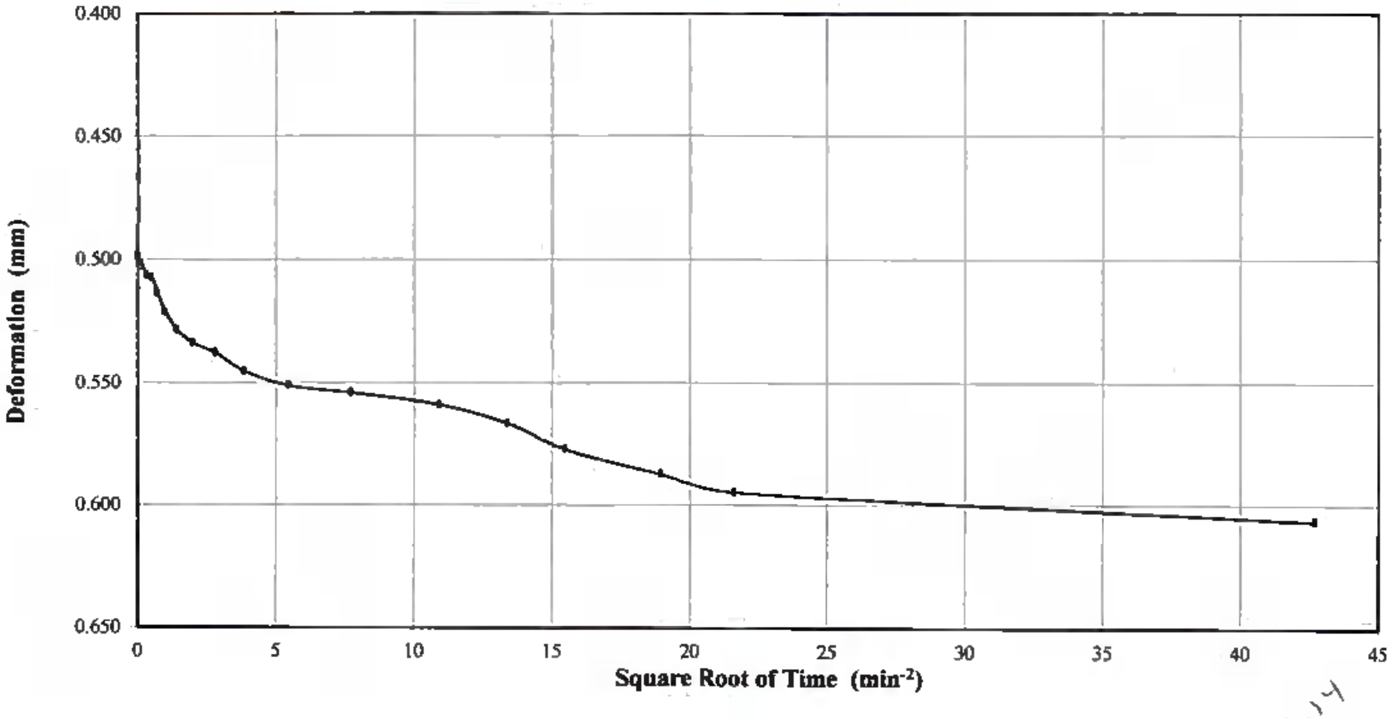
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf





1-16-14
NSK

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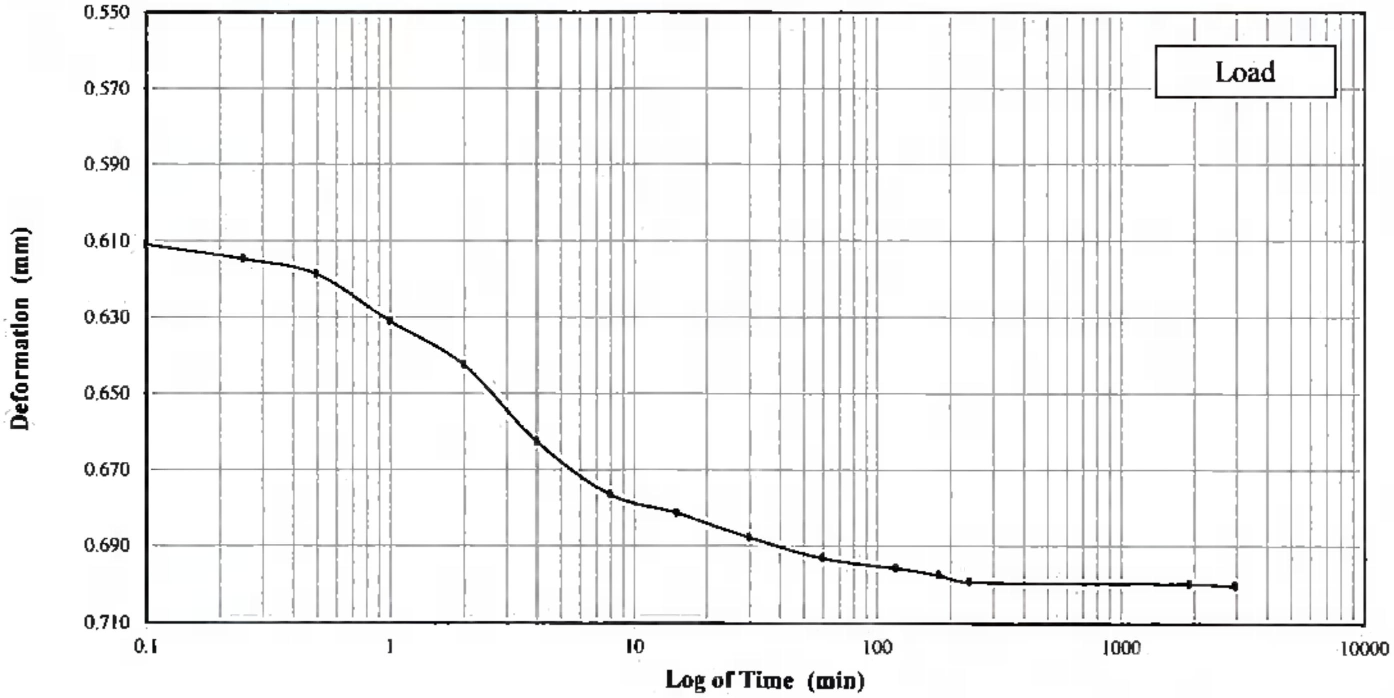
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

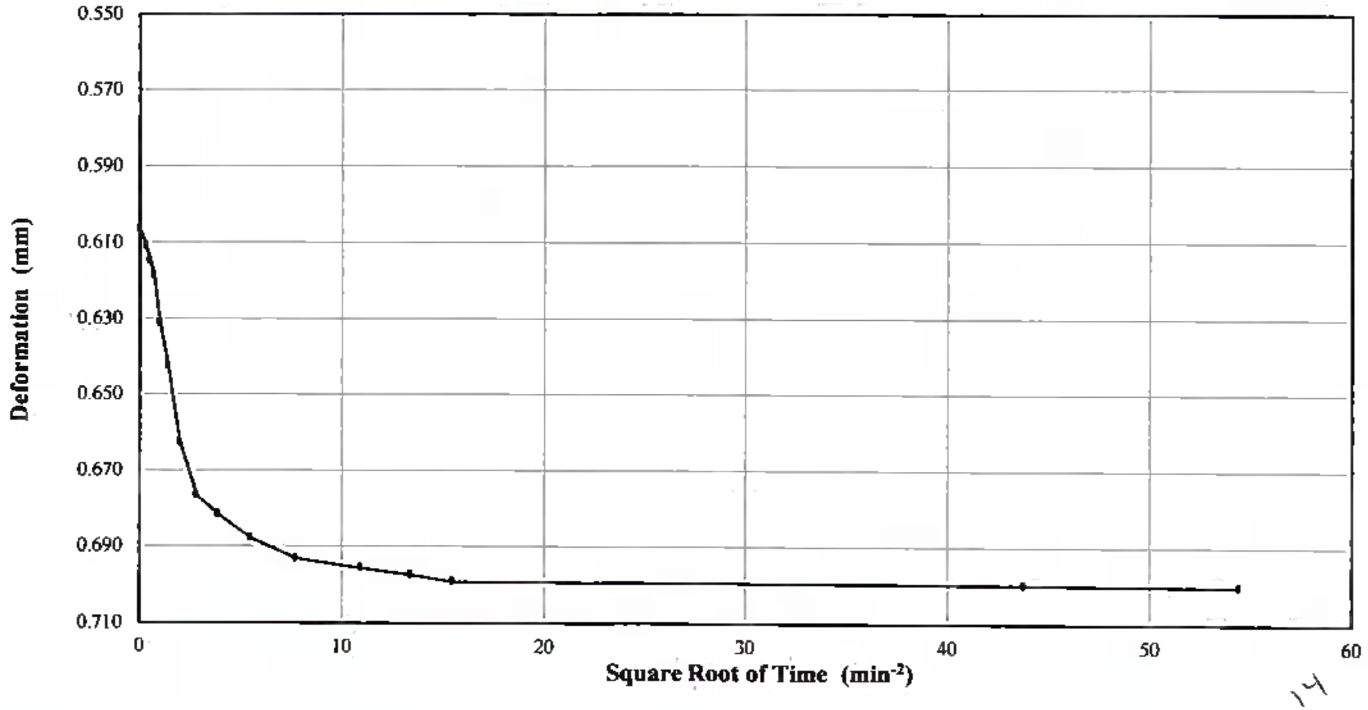
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-112, ST-03 (46.5-48.5')
Lab Sample No:	13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 3 - 1000 psf





1-16-14
NSK

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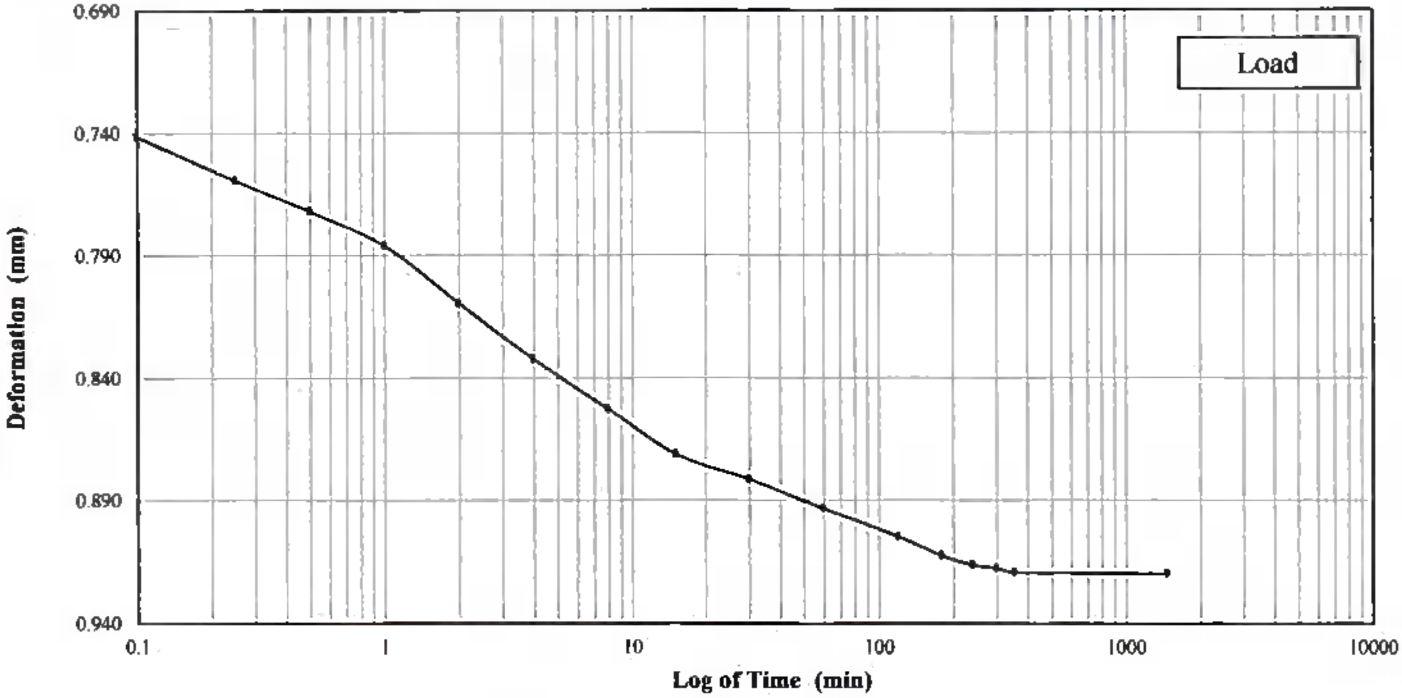
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

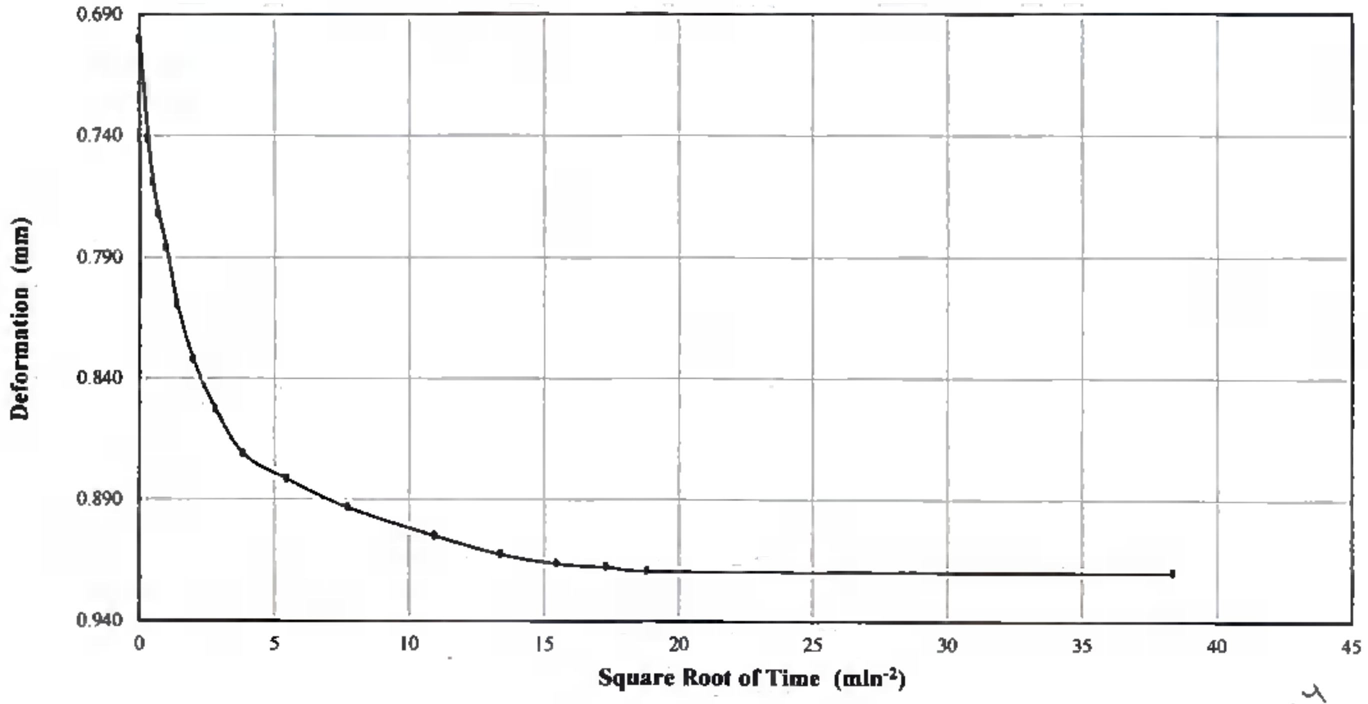
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf





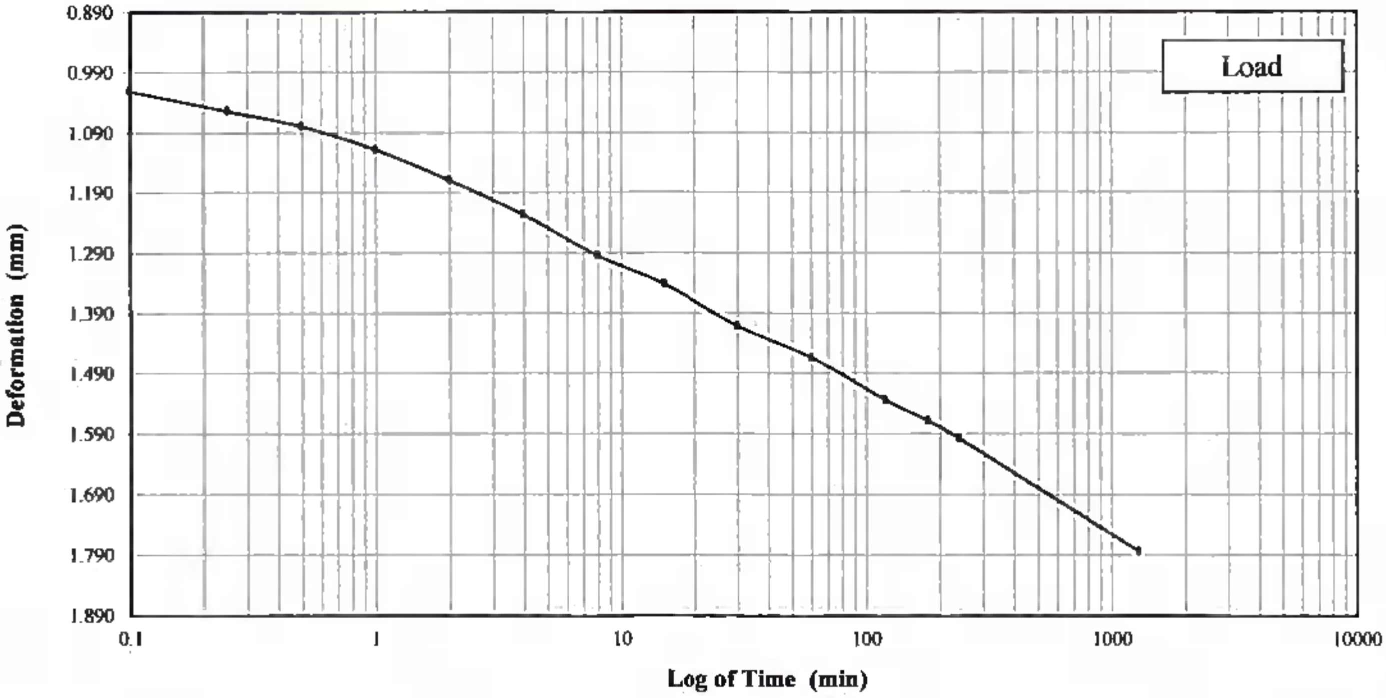
1-16-14
NSK

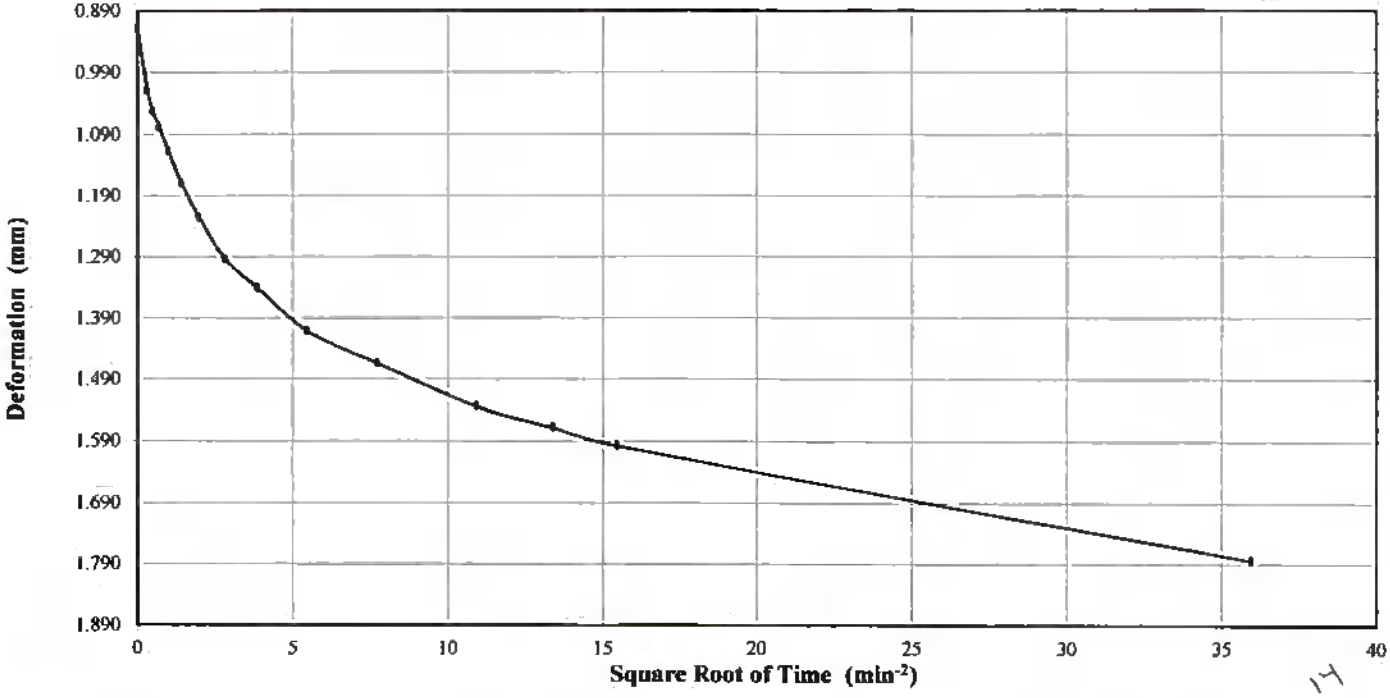
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 5 - 4000 psf





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NSK

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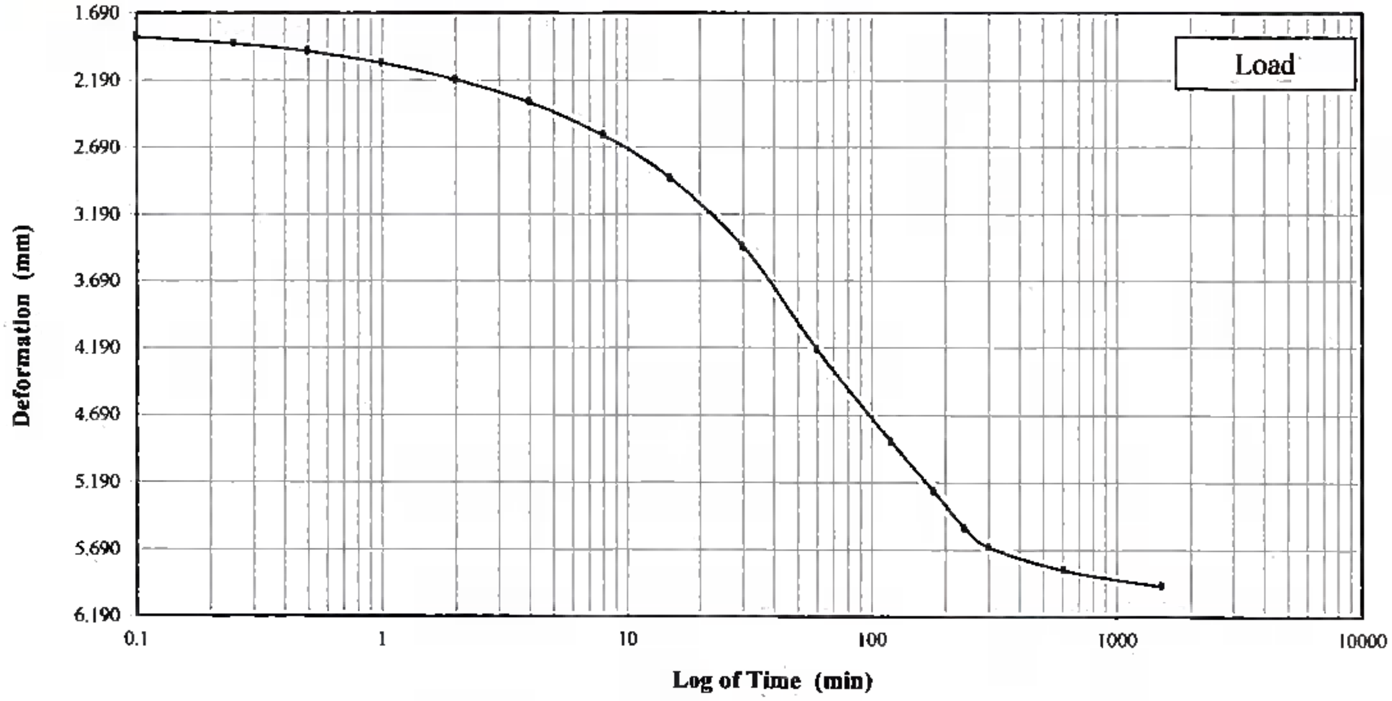
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

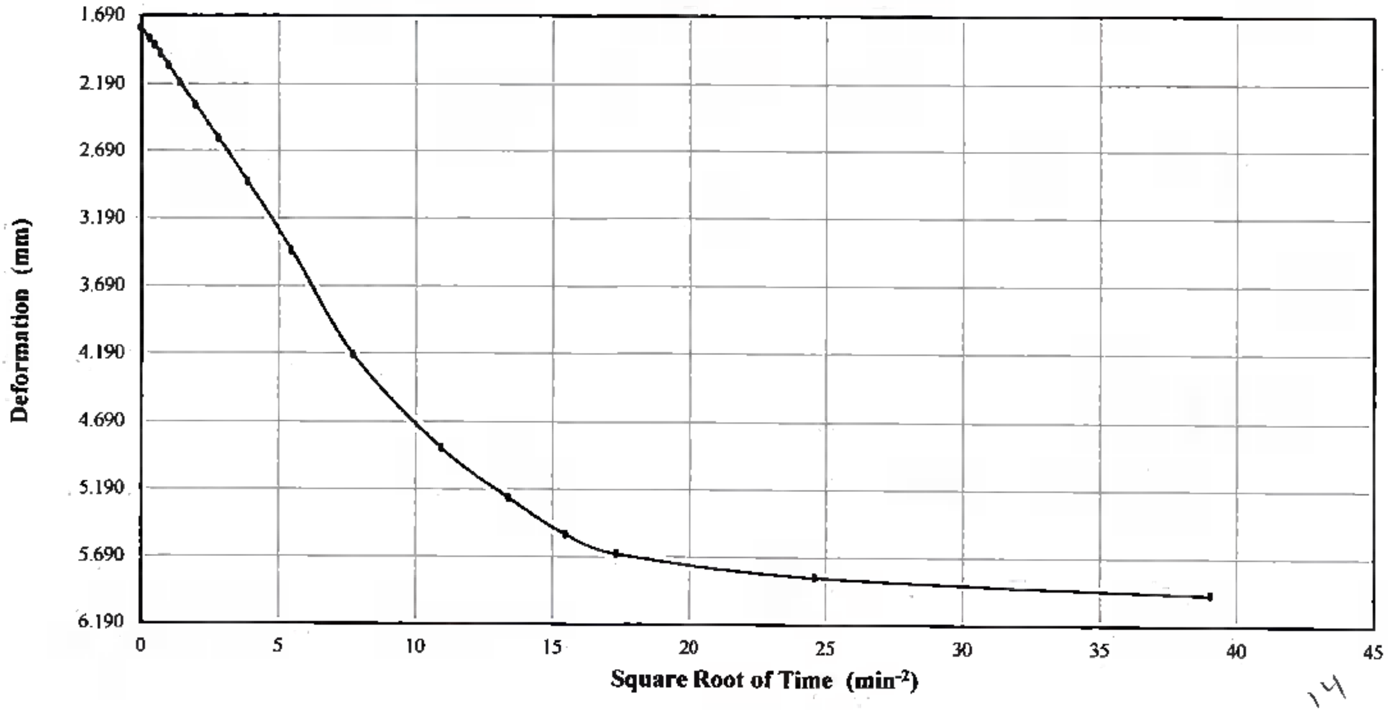
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 6 - 8000 psf





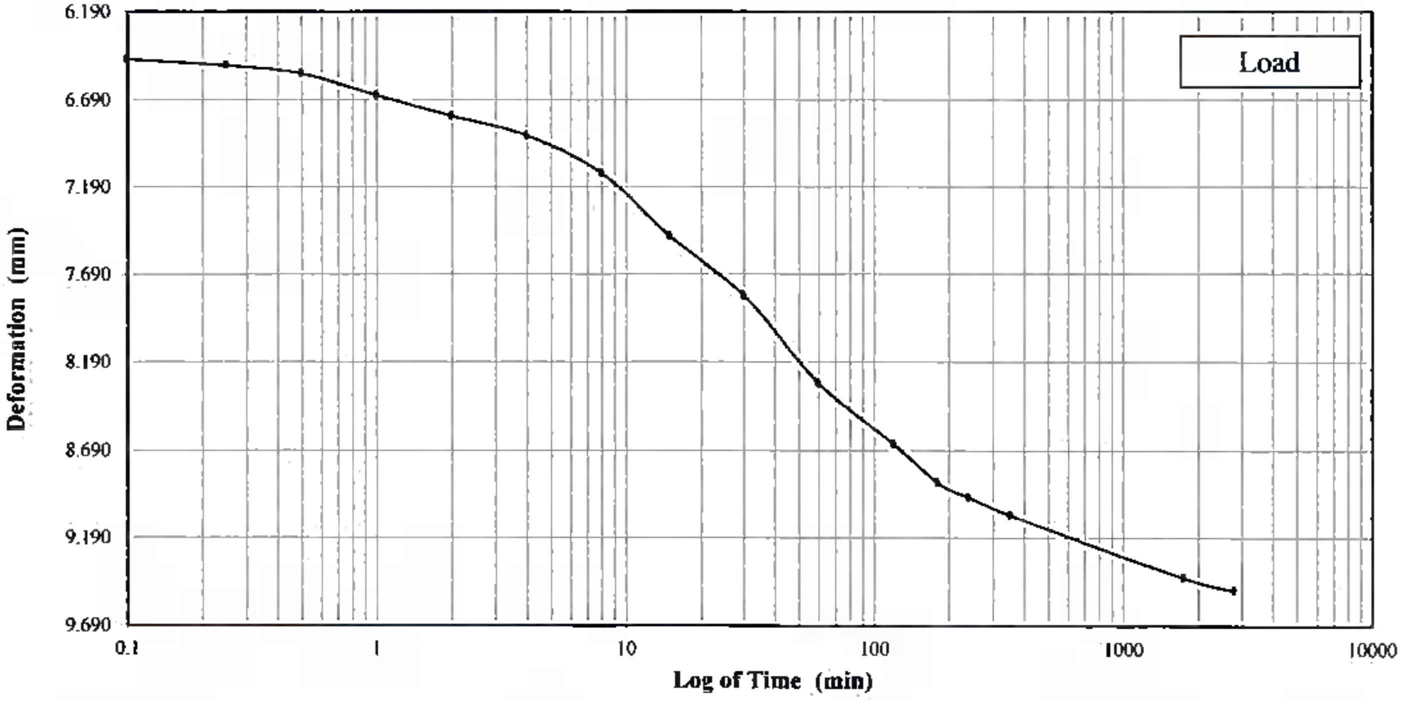
1-16-14
NSK

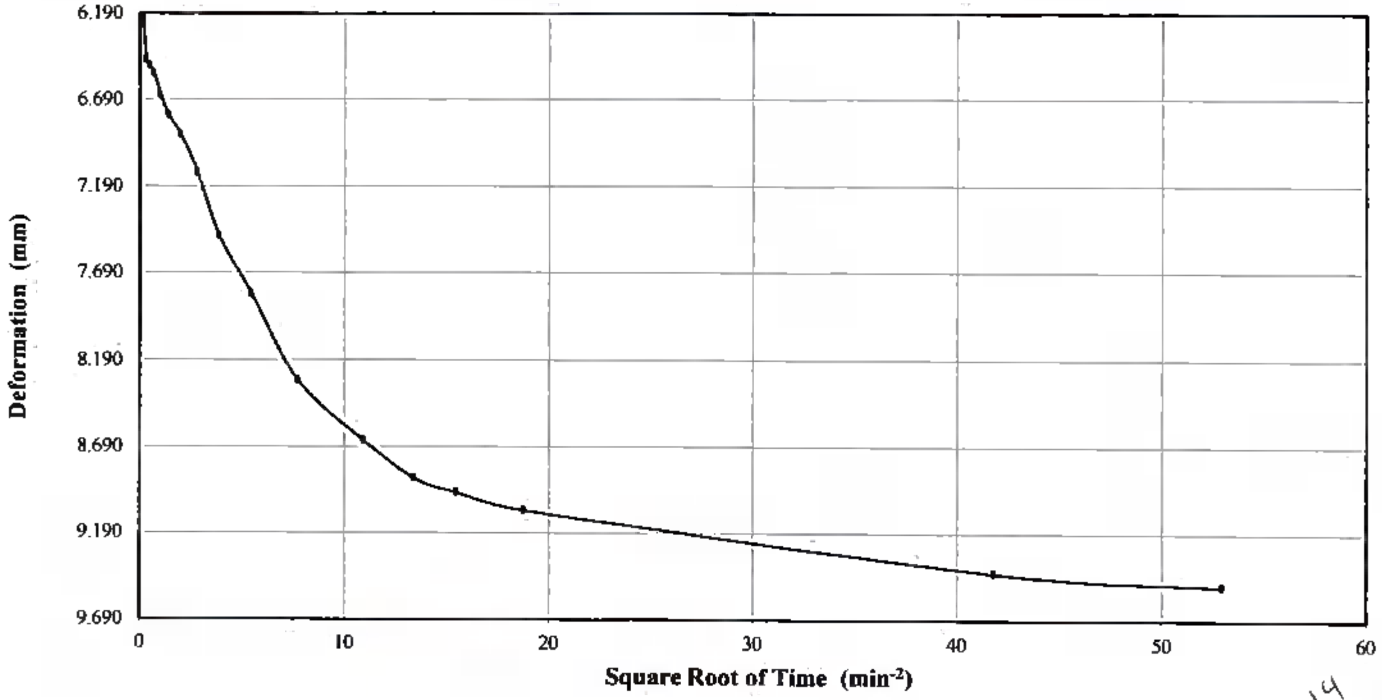
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 7 - 16000 psf





1-16-14
NSK

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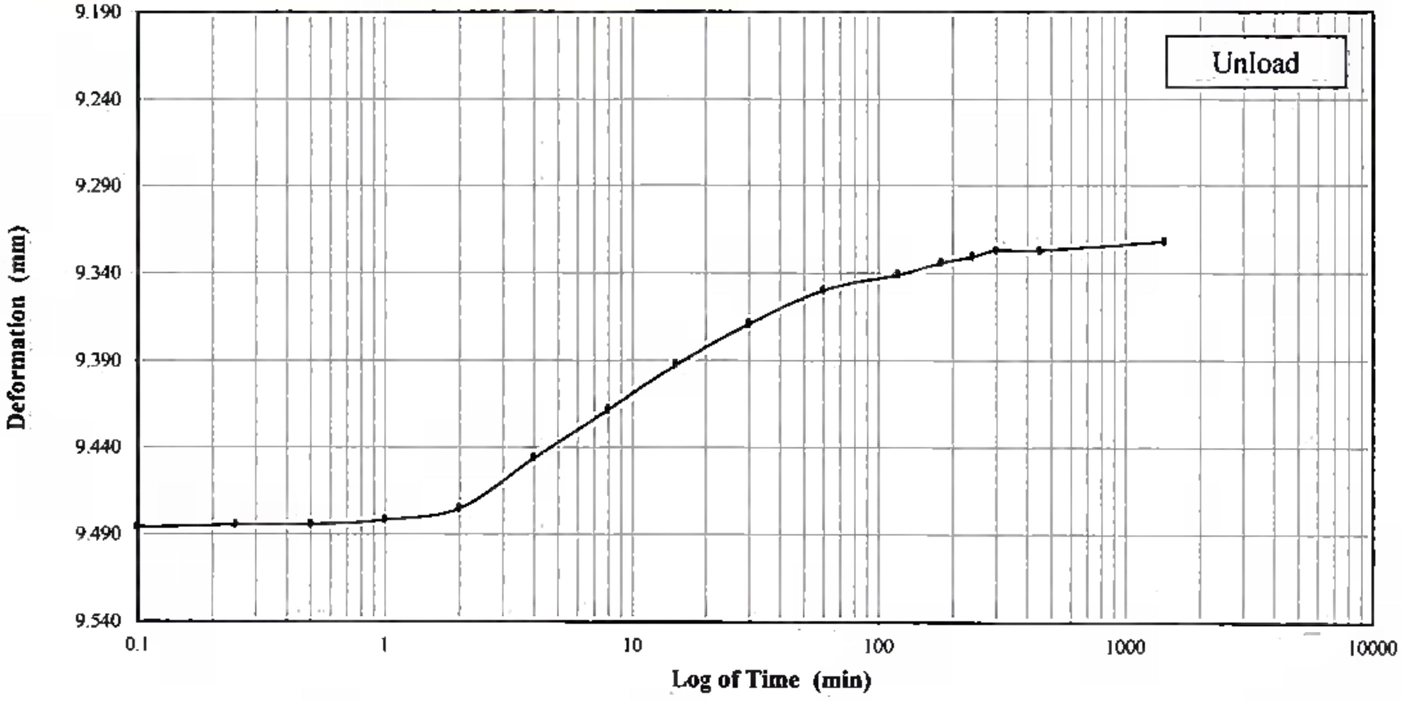
Tel: (770) 910 7537 Fax: (770) 910 7538

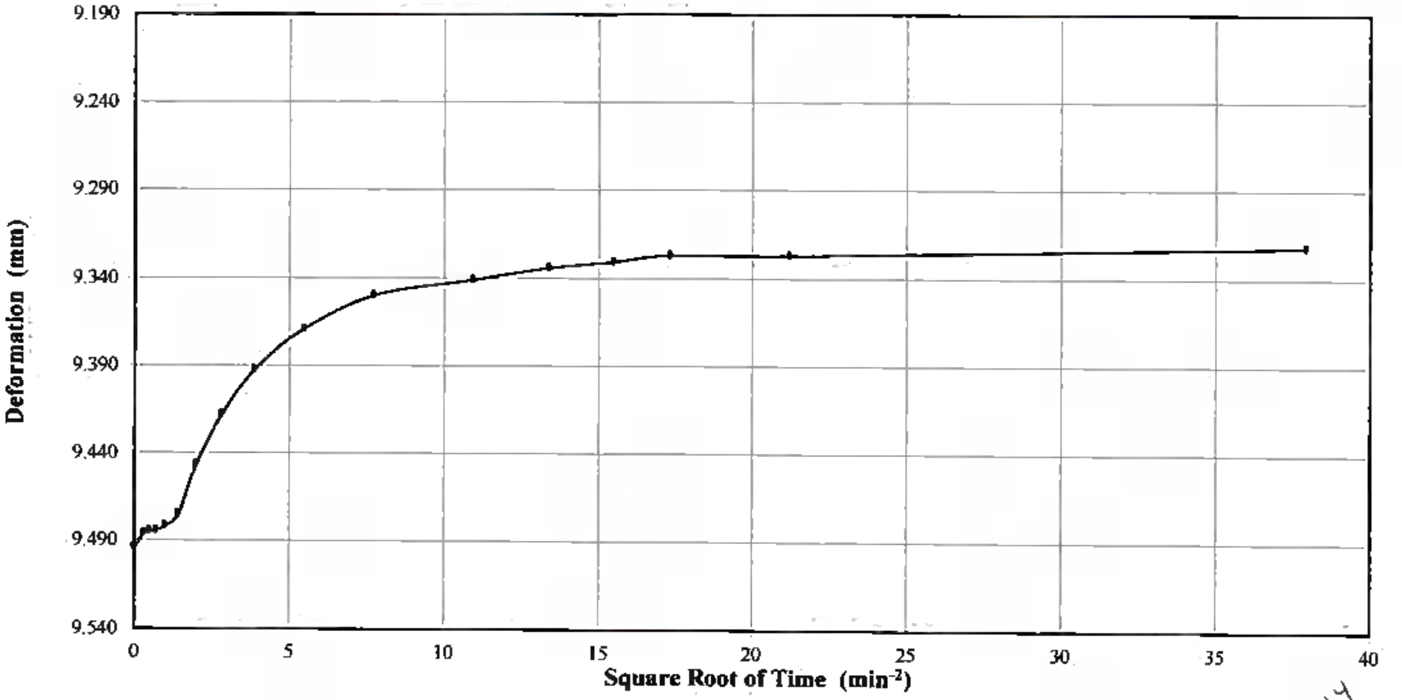
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-112, ST-03 (46.5-48.5')
Lab Sample No: 13J367

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 8 - 8000 psf





1-16-14
NSK

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FLEXIBLE WALL PERMEABILITY TEST (1)

ASTM D 5084 *

Project Name:	Winyah Generating Station
Project Number:	618
Client Name:	Geosyntec Consultants
Site Sample ID:	SPT-112, ST-04 (65.0-67.0')
Lab Sample Number:	13J368
Material Type:	Soil
Specified Value (cm/sec):	NA
Date Test Started:	12/4/2013

Specimen	Test Specimen Initial Condition					Test Conditions					Hydraulic
No.	Spec. Prep. (2) (-)	Spec. Length (cm)	Spec. Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)	Cell Press. (psi)	Back Press. (psi)	Consolid. Press. (psi)	Permeant Liquid (3) (-)	Average Gradient (-)	Conductivity (cm/s)
1	ST	5.72	7.26	73.6	42.5	101.0	70.0	31.0	DTW	7	1.8E-8

Notes:

1. Method C, "Falling-Head, Increasing-Tailwater" test procedures were followed during the testing.
2. Specimen preparation: ST = Shelby Tube, R = Remolded, B = Block Sample.
3. Type of permeant liquid: DTW = Deaired Tap Water, DDI = Deaired Deionized Water

* Deviations:

Laboratory temperature at 22±3 °C.

Test specimen final conditions are not presented.

1-04-14
NJR

Attachment 4-B

S&ME Results (1978)

Index Testing

SOIL DATA SUMMARY

SBME JOB NO. SS7735

BORING NUMBER	SAMPLE DEPTH	CLASSIFICATION	STANDARD PENETRATION RESISTANCE	NATURAL MOISTURE (%)	% FINER # 200	UNIT WEIGHT P.C.F.		PROCTOR DATA		SPECIFIC GRAVITY	VOID RATIO Eo	UNCOFINED COMP. MAX.	ATTER-BERG LIMIT		TRIAXIAL SHEAR		CONSOLIDATION Cc	OTHER
						W	D	MAX	OMC				LL	PI	C	φ		
SC55	2-4'	Gray Silty SAND		22.6	11.6									NP				
SP4	2-3'	Brown Silty SAND		21.1 14.9	8.6	109.4	95.9	105.8	13.8		.757			NP	0	32.5°		
SC19	6-8'	Black Silty SAND			4.7									NP				
SC41	20.5	Gray Slightly Clayey SAND & SHELLS				123.6	111.9				.507				250	0°	(UU)	
SC7	2-5'	Gray Brown Silty SAND		14.7		109.7	95.6				.762				0	30°		
SC77	10-12'	Gray Silty CLAY		89.2		99.1	52.4				2.2				350 (400°	11.5° 15.5°)		
SC19	16-18'	Gray Silty CLAY		69.1		101.3	59.9				1.89				600	0°		
SC15	17-19'	Gray Silty CLAY		129.2		77.1	33.6				3.64						2.44	
SC78	11-13'	Gray Silty CLAY		71.4		99.9	58.3				1.98				570	0°	(UU)	
SC76	16.5 - 18.5	Gray Silty CLAY		96.7		91.3	46.4				2.66				300	0°	(UU)	
SC68	24.5 - 26.5	Gray Silty CLAY with Sand Seams				102.4	83.5				0.65						.094	
SC17	9.5 - 11.5	Gray Silty CLAY		135.2		84.6	34.0				3.76							
SC78	16-20'	Gray Silty CLAY		87.9		85.3	45.4				2.1				400 (300	9° 20°)	2.48	
MP13	41.5'	Gray Sandy Silty CLAY		45.8		109.1	74.8				1.25				2900	4°		
SC25	3-4'	Gray Clayey SAND		17.6	27.7	124.0	105.4	112.8	16.0		.585		34	17(	2000 1110	19° 25.8°)		
SC25	6-8'	Gray Silty SAND			25.0			108.8	15.8					NP				

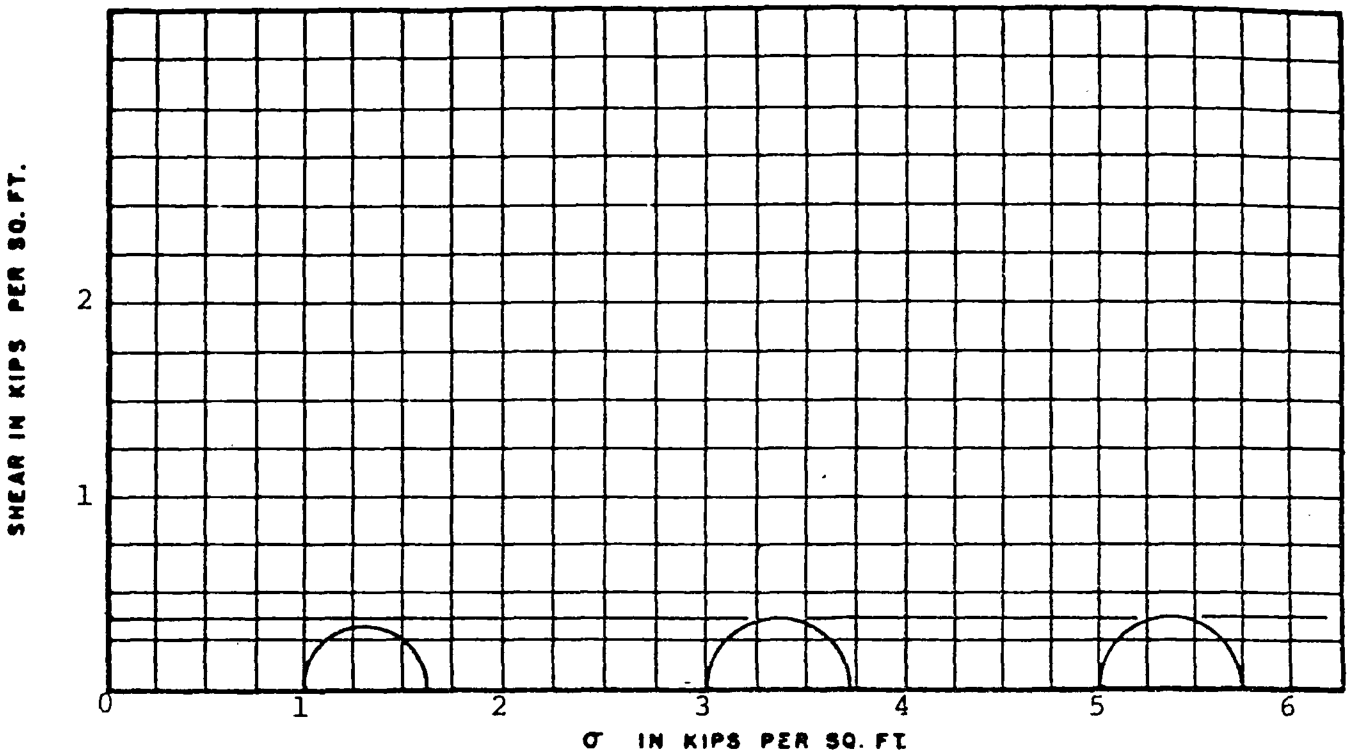
SOIL & MATERIAL ENGINEERS, INC.

Boring Location	Sample Number Or Depth *	Moisture Content %	Passing #200 Sieve - %
SC-1	1	22.8	16.1
SC-2	1	16.3	6.1
SC-3	1	9.7	4.5
SC-3	2	33.8	-
SC-4	1	17.9	8.1
SC-5	1	15.1	9.5
SC-6	1	14.0	7.7
SC-6	2	13.0	10.4
SC-9	1	8.7	17.0
SC-10	2	28.0	5.6
SC-12	1	21.8	5.6
SC-12	2	24.8	-
SC-13	2	26.9	4.5
SC-14	1	26.8	-
SC-19	2	31.0	-
SC-19	1	17.4	11.4
SC-19	2	34.0	-
SP-4	2-3'	21.1	8.6
SC-20	1	21.4	16.3
SC-20	2	15.6	23.0
SC-20	3	25.4	9.4
SC-22	1	25.4	-
SC-22	2	23.5	-
SC-25	3-4'	7.6	27.7
SC-25	6-8'	-	25.0
SC-30	2-3'	27.5	40.1
SC-31	2-4'	19.7	25.2
SC-34	1.5-3'	24.0	33.6
SC-34	7-9'	-	11.1
SC-36	2-3'	32.7	62.5
SC-36	3-4'	22.3	37.9
SC-36	6-7'	-	1.2
SC-38	2-3'	33.8	52.4
SC-39	1.5-3'	28.8	75.2
SC-39	6-7'	-	57.9
SC-45	1	22.6	16.6
SC-45	2	23.9	9.9
SC-49	1-2'	28.0	44.7
SC-50	1-2'	26.9	49.5
SC-53	1	-	10.1
SC-55	2-4'	22.6	11.6
SC-57	1	18.8	-
SC-57	2	28.9	18.0
SC-62	2	22.4	4.9
SC-65	2	25.6	9.8
SC-66	1	17.7	14.3

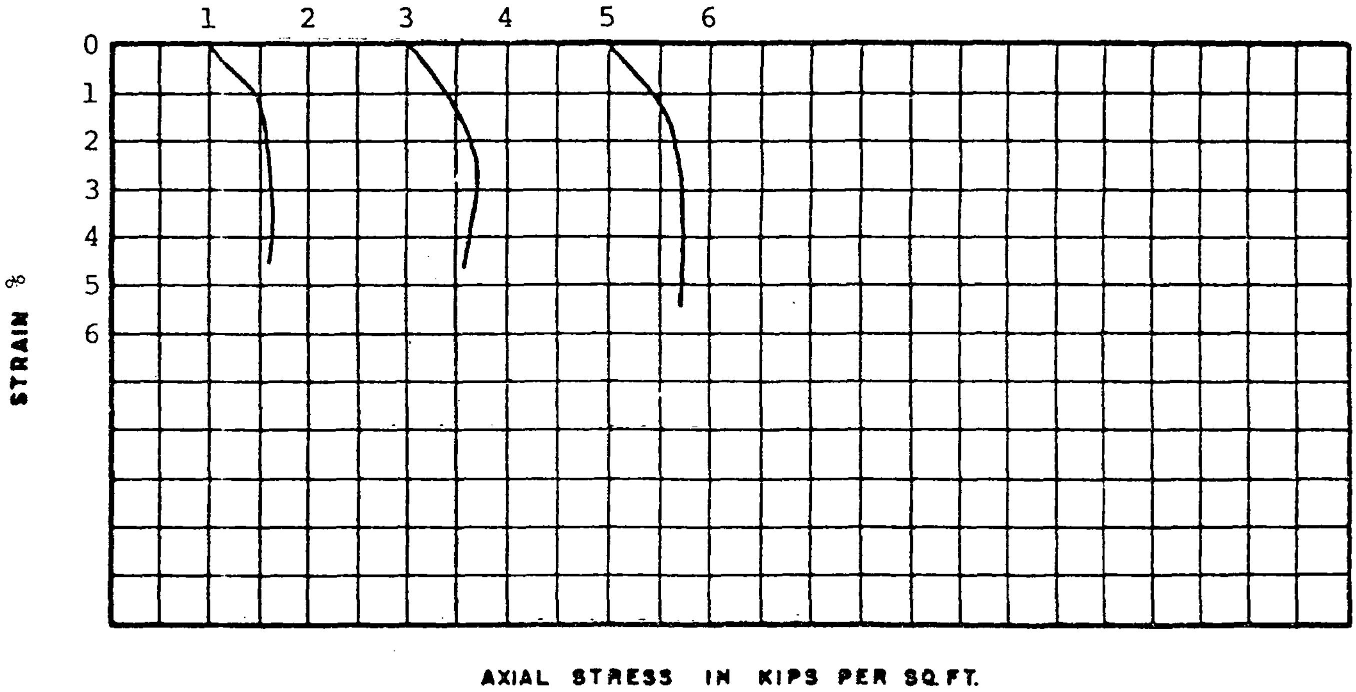
Boring Location	Sample Number Or Depth *	Moisture Content %	Passing #200 Sieve - %
SC-67	1	18.2	21.8
SC-68	1	20.1	-
SC-68	2	13.4	12.1
SC-69	1	21.6	8.8
SC-69	2	22.7	6.4
SC-70	2	21.0	4.3
SC-71	1	9.7	1.4
SC-71	2	23.0	4.9
SC-72	1	13.4	5.7
SC-72	2	25.1	2.5
SC-73	2	25.9	3.4

* Sample Number 1 taken from 1 to 2.5'; Number 2 from 4.5 to 6.0; Number 3 from 7 to 8.

Triaxial Testing



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

"COHESION", c 300 psf c'

ANGLE OF SHEAR RESISTANCE φ =0°

UNIT WEIGHT, γ_d 46.5, 47.5, 45.3 pcf

WATER CONTENT, w 95.0, 92.9, 102.4%

VOID RATIO, e 2.626, 2.549, 2.814

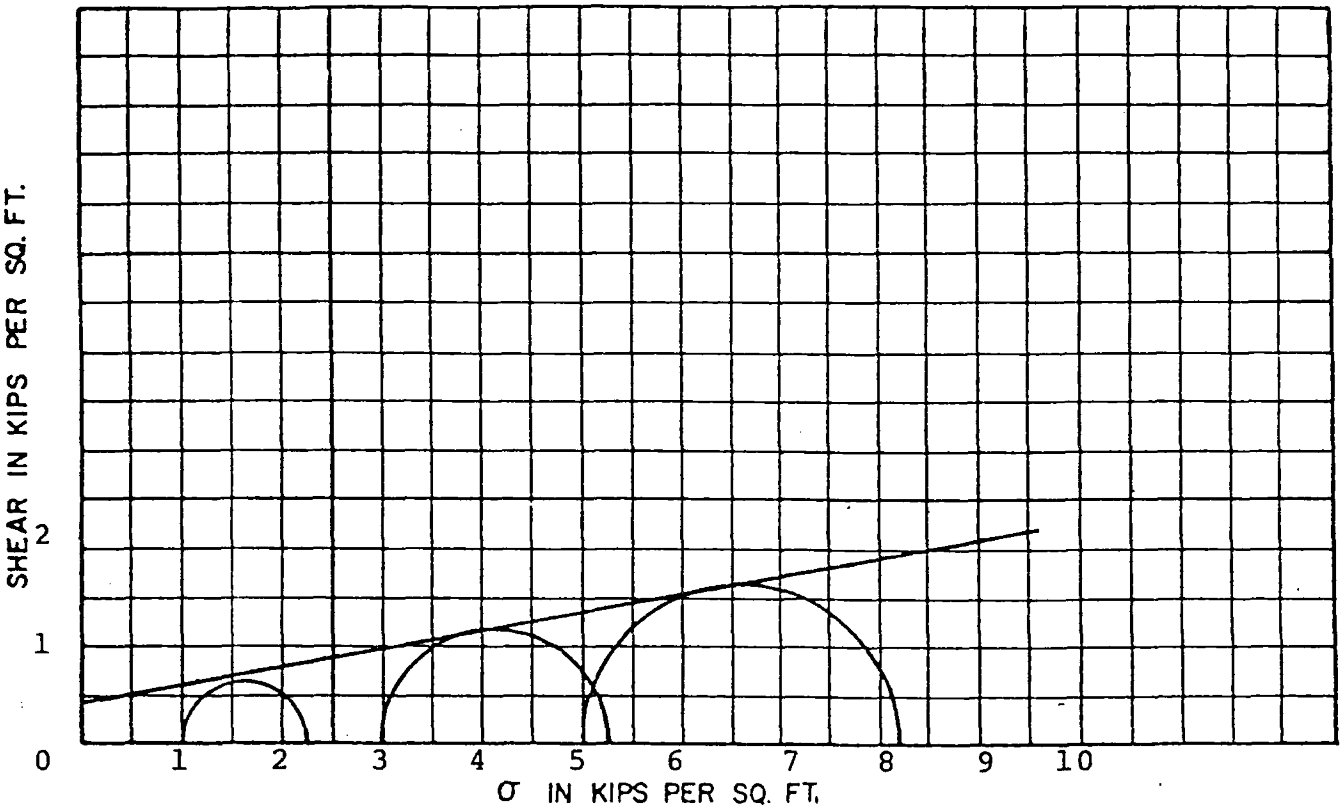
UNCONSOLIDATED UNDRAINED

TRIAXIAL SHEAR TEST

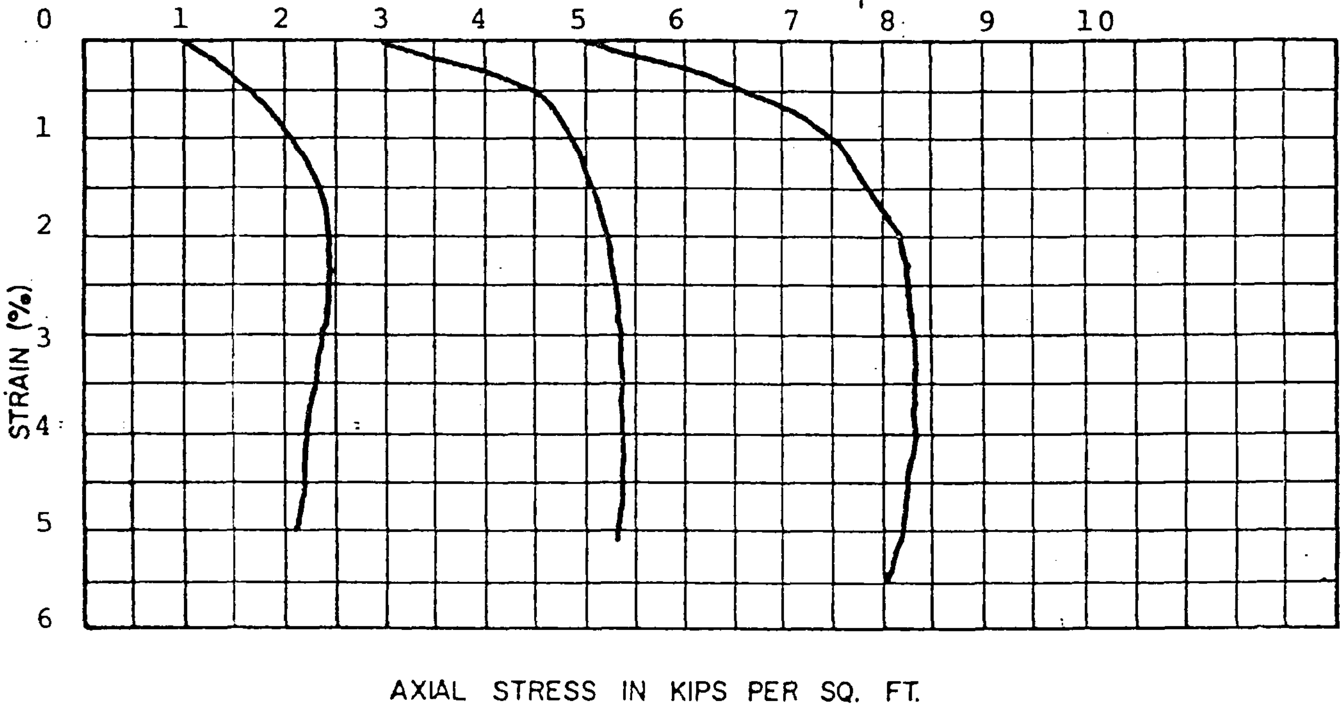
BORING NO. SC-76 SAMPLE NO. UD

ELEV. OR DEPTH 16.5-18.5 JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.



MOHR DIAGRAMS — φ



STRESS — STRAIN CURVES

TOTAL STRESS

TRIAXIAL SHEAR TEST

SATURATED, CONSOLIDATED, UNDRAINED

WITH PORE PRESSURE MONITORING

JOB NO. SS7729 BORING NO. SC-77

SAMPLE NO. ________ ELEV. OR DEPTH 10-12

SOIL & MATERIAL ENGINEERS, INC.

"COHESION", c 0.35 KSF

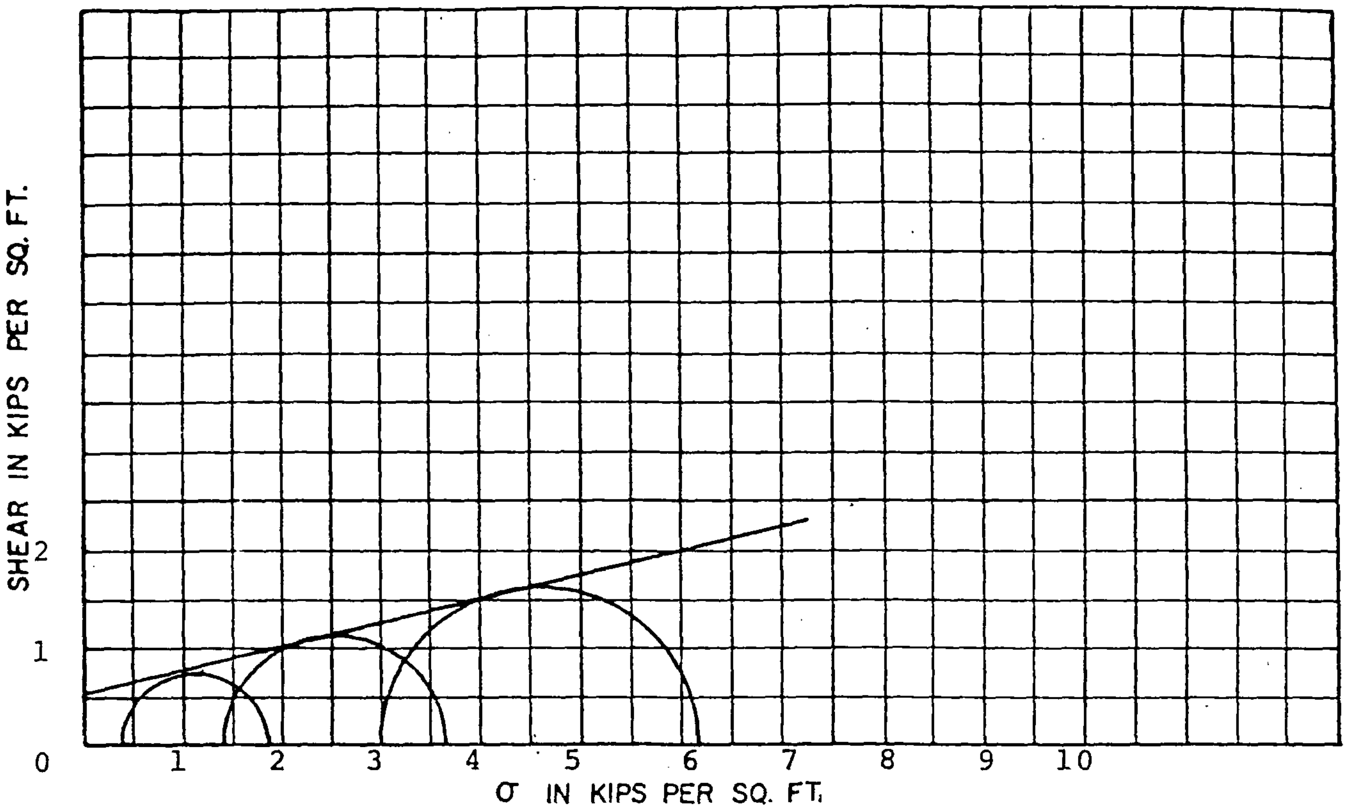
ANGLE OF SHEAR RESISTANCE φ 11.5 °

UNIT WEIGHT, γ (W) 99.1 (0) 52.4

WATER CONTENT w 89.2

VOID RATIO, e 2.215

CLASSIFICATION, GRAY SILTY CLAY
WITH FINE SANDY SEAMS
WITH SHELLS



MOHR DIAGRAMS — φ

STRAIN (%)

AXIAL STRESS IN KIPS PER SQ. FT.

STRESS — STRAIN CURVES

"COHESION", c 0.4 KSF

ANGLE OF SHEAR RESISTANCE φ 15.5°

UNIT WEIGHT, γ (W) 99.1 (o) 52.4

WATER CONTENT w 89.2

VOID RATIO, e 2.215

CLASSIFICATION, GRAY SILTY CLAY WITH FINE SANDY SEAMS WITH SHELLS

EFFECTIVE SHEAR

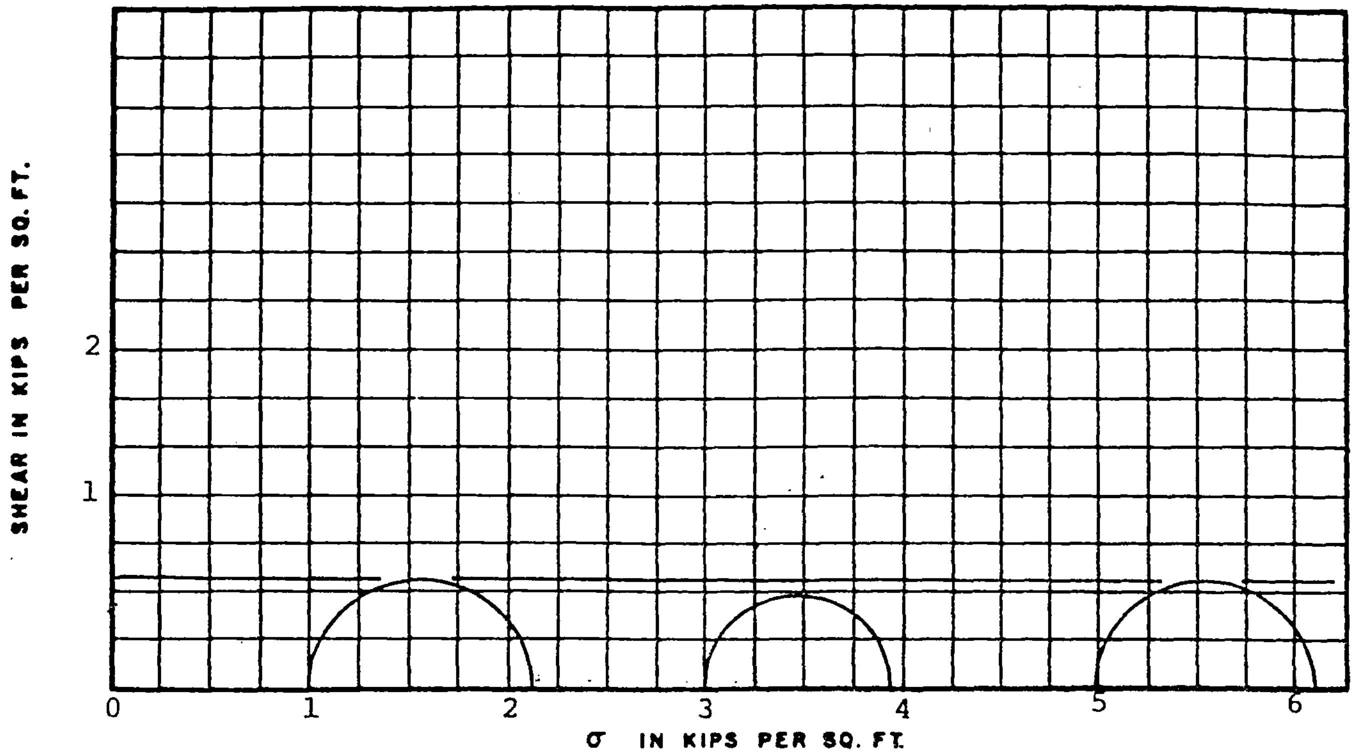
TRIAXIAL SHEAR TEST

SATURATED, CONSOLIDATED, UNDRAINED, WITH PORE PRESSURE MONITORING

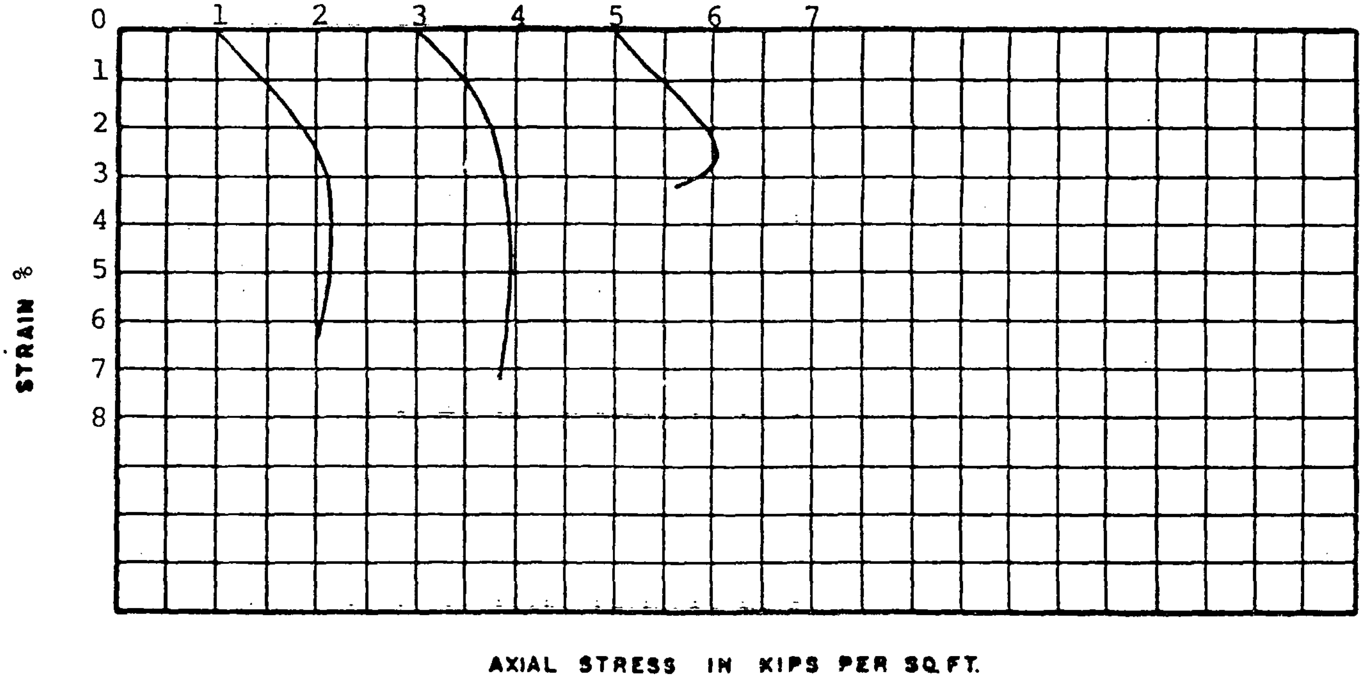
JOB NO. SS7729 BORING NO. SC-77

SAMPLE NO. ______ ELEV. OR DEPTH 10-12

SOIL & MATERIAL ENGINEERS, INC.



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

"COHESION", c 570 pcf c'

ANGLE OF SHEAR RESISTANCE. φ =0°

UNIT WEIGHT, γ_d 61.7, 68.2, 45.1 pcf

WATER CONTENT. w 63.4, 52.7, 98%

VOID RATIO. e 1.731, 1.470, 2.736

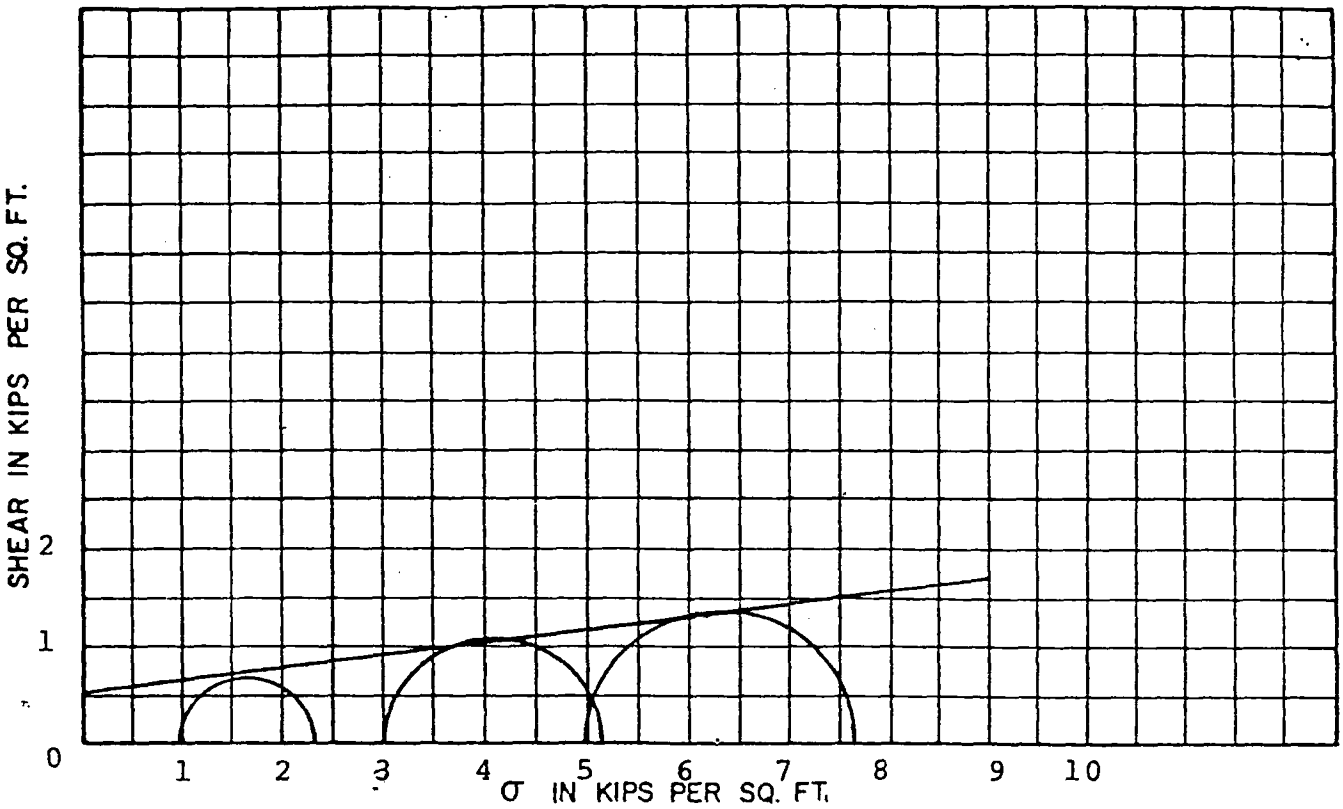
UNCONSOLIDATED UNDRAINED

TRIAXIAL SHEAR TEST

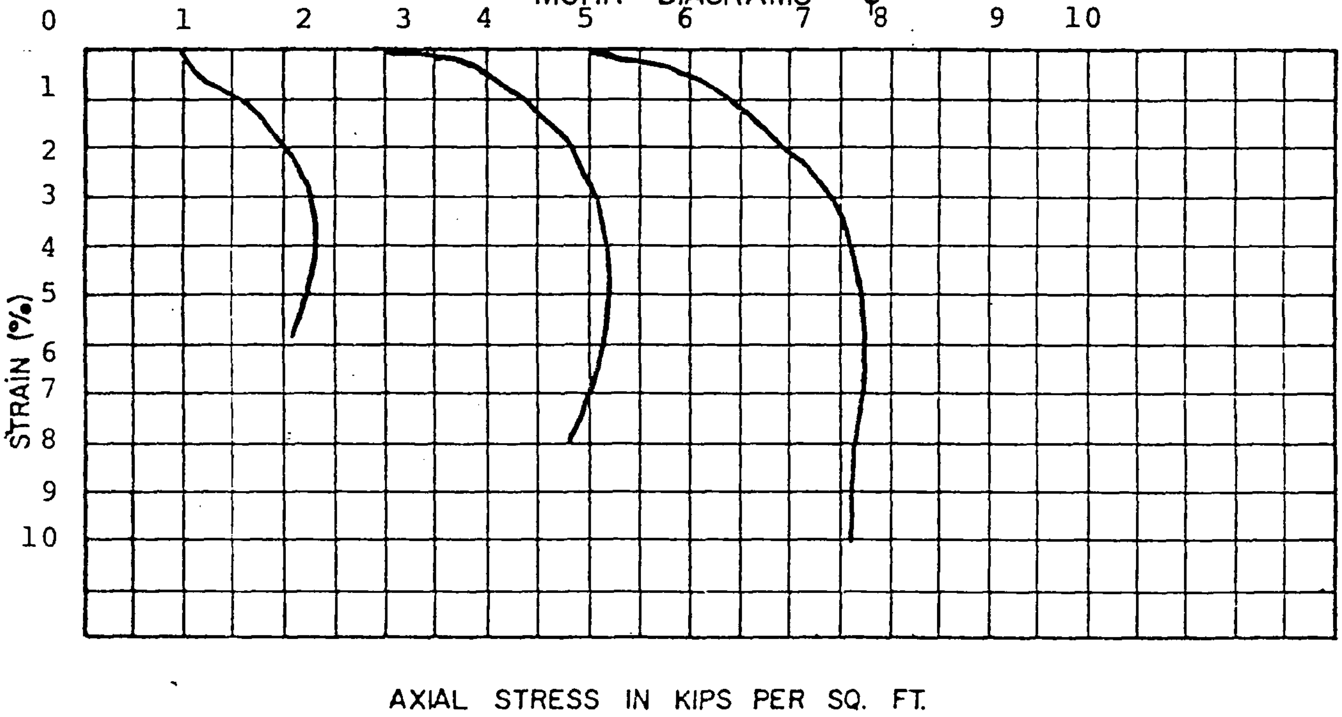
BORING NO. SC-78 SAMPLE NO. UD

ELEV. OR DEPTH 11-13 JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.



MOHR DIAGRAMS — φ



STRESS — STRAIN CURVES

TOTAL STRESS

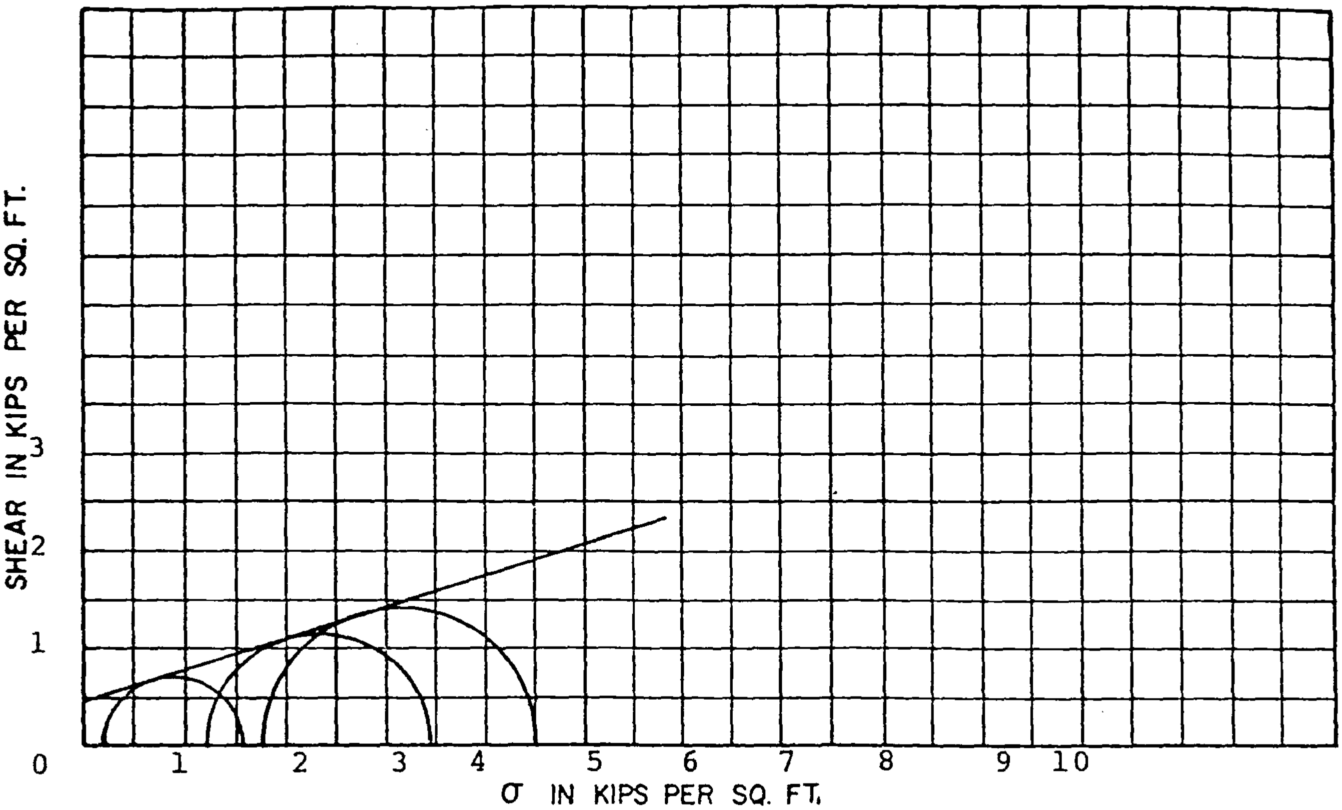
TRIAXIAL SHEAR TEST

SATURATED, CONSOLIDATED, UNDRAINED
WITH PORE PRESSURE MONITORING

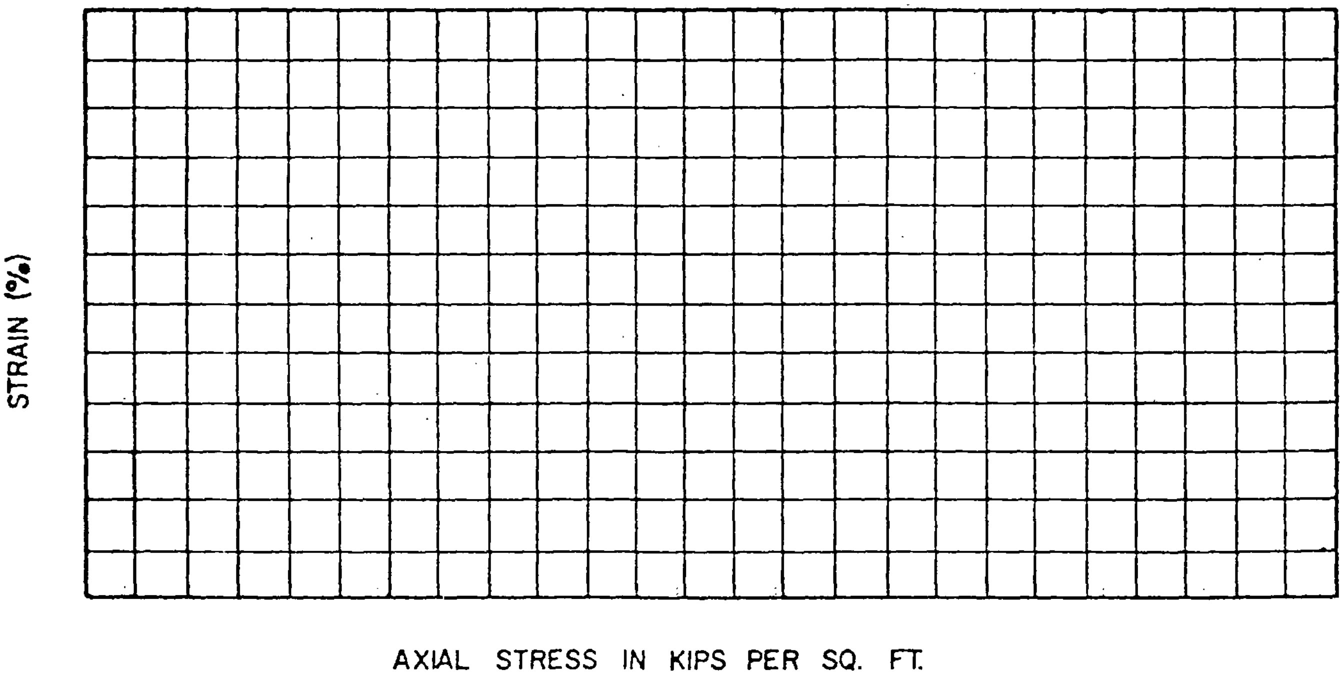
"COHESION", c 0.4 KSF
ANGLE OF SHEAR RESISTANCE φ 9°
UNIT WEIGHT, γ (W) 85.3 (D) 45.4 PCF
WATER CONTENT w 87.9
VOID RATIO, e 2.111
CLASSIFICATION, GRAY VERY SILTY CLAY WITH FINE SANDY SEAMS

JOB NO. SS7729 BORING NO. SC-78
SAMPLE NO. ______ ELEV. OR DEPTH 16-20

SOIL & MATERIAL ENGINEERS, INC.



MOHR DIAGRAMS — φ



STRESS — STRAIN CURVES

"COHESION," c 0.3 KSF

ANGLE OF SHEAR RESISTANCE φ 20°

UNIT WEIGHT, γ (W) 85.3 (D) 45.4 PCF

WATER CONTENT w 87.9

VOID RATIO, e 2.111

CLASSIFICATION, GRAY VERY SILTY CLAY WITH FINE SANDY SEAMS

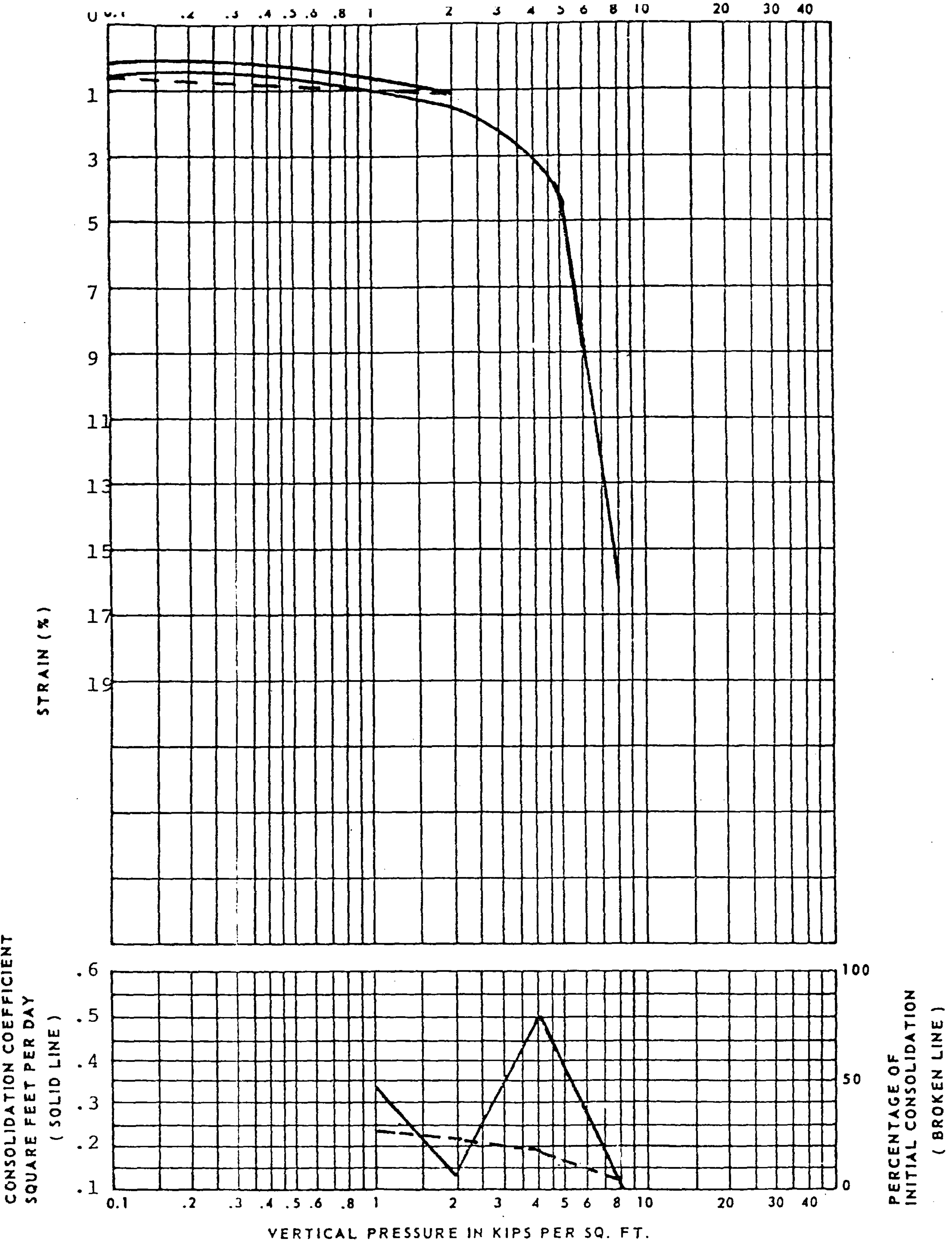
EFFECTIVE STRESS
TRIAXIAL SHEAR TEST
SATURATED, CONSOLIDATED, UNDRAINED
WITH PORE PRESSURE MONITORING

JOB NO. SS7729 BORING NO. SC-78

SAMPLE NO. ______ ELEV. OR DEPTH 16-20

SOIL & MATERIAL ENGINEERS, INC.

Consolidation/Permeability Testing



COMPRESSION INDEX .094

UNIT WEIGHT (w) 102.4 (d) 83.5

WATER CONTENT 22.7%

SATURATION 92.5%

INITIAL VOID RATIO 0.65

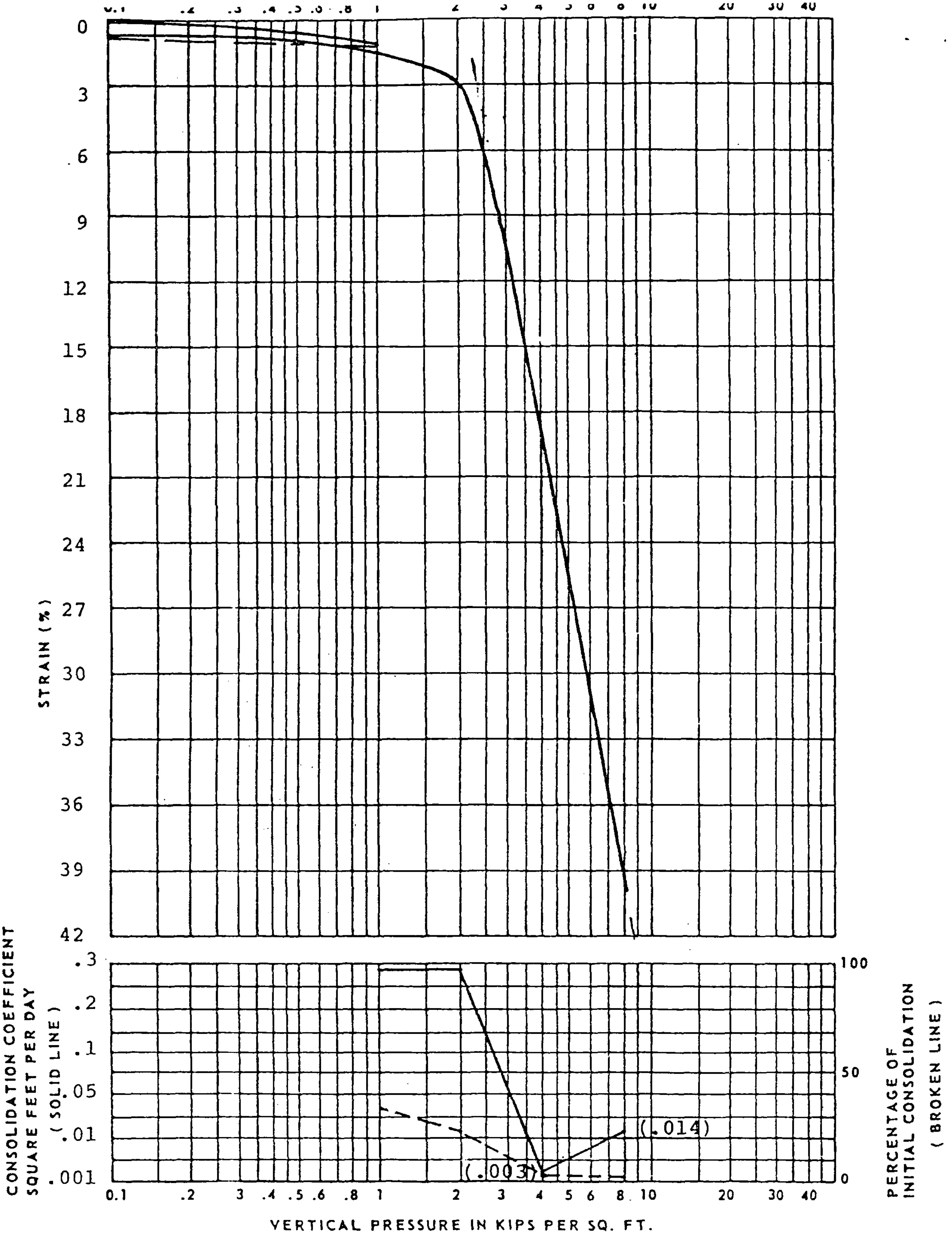
CONSOLIDATION TEST

BORING NO. SC68 SAMPLE NO. UD

ELEV. OR DEPTH 24.5-26.5 JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.

SM-32



COMPRESSION INDEX 2.48

UNIT WEIGHT (W) 85.5 D) 43.9 pcf

WATER CONTENT 96.6%

SATURATION 93.4

INITIAL VOID RATIO 2.75

CONSOLIDATION TEST

BORING NO. SC-78 SAMPLE NO. UD

ELEV. OR DEPTH 18-20' JOB NO. SS7729

SOIL & MATERIAL ENGINEERS INC.

SM-32

ATTACHMENT 5

Subsurface Stratigraphy and Material Properties

Geosyntec
consultants

CALCULATION PACKAGE COVER SHEET

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

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SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES: SOUTH ASH POND

INTRODUCTION

This calculation package was prepared to present the subsurface stratigraphy and material properties supporting the geotechnical analyses for the South Ash Pond located at Winyah Generating Station (WGS or "Site"), which is owned and operated by Santee Cooper. This calculation package is Attachment 5 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: South Ash Pond* (Safety Factor Assessment Report) prepared by Geosyntec Consultants (Geosyntec). The remainder of this calculation package presents the: (i) site investigations; (ii) subsurface stratigraphy and coal combustion residuals (CCR); (iii) interpretation of the phreatic surface and current water levels; (iv) standard penetration test (SPT) and cone penetration test (CPT) interpretations; (v) laboratory testing program; (vi) in-situ testing interpretation; and (vii) selected material properties for analysis.

SITE INVESTIGATIONS

This section summarizes the geotechnical subsurface investigations conducted in the vicinity of the South Ash Pond at WGS. In 1977 and 1978, Soil and Materials Engineers, Inc. (S&ME) performed a general subsurface investigation supporting the construction of CCR surface impoundments, including the South Ash Pond, at the WGS. In October 2013, Geosyntec conducted a subsurface investigation in the vicinity of the South Ash Pond to collect geotechnical data supporting the evaluation of closure alternatives for the surface impoundment. Geosyntec remobilized to the site in March 2016 to conduct a focused subsurface investigation to collect additional samples of the soft clay foundation layer that underlies the western perimeter dike of the surface impoundment.

Figure 1 presents the locations of soil test borings performed during the investigations and the CPT soundings conducted as part of Geosyntec's subsurface investigations. Soil test boring logs, CPT sounding data, and laboratory test results for the subsurface investigation programs are provided in Attachments 2, 3, and 4, respectively, of this Safety Factor Assessment Report. Geotechnical laboratory test data are summarized in Appendix 1 of this attachment.

Historical Investigation

The S&ME investigation (S&ME, 1978) was conducted to assess the suitability of on-site materials for construction and to design the perimeter dikes. In the vicinity of the South Ash Pond, the investigation included 18 soil test borings (SC-63, SC-64, SC-66 to SC-78, SC-80, SC-81, and SC-84) advanced before construction of the surface impoundment from 26.5 to 41 feet (ft) below ground surface (bgs) until refusal was encountered at the Chicora Member (dense cemented shell unit). SPT blow counts (i.e., N-values) were recorded at approximately 2.5 ft intervals up to 10 ft below ground surface and at 5-ft depth intervals thereafter. Representative samples were collected by a standard split spoon sampler or by thin-walled Shelby tubes, which were utilized for index, consolidation, and triaxial shear strength testing. The geotechnical laboratory program consisted of index (natural moisture content, grain size distribution and Atterberg limits), unit weight, compaction, consolidation, and shear strength testing of select samples.

Geosyntec Investigations

The October 2013 subsurface investigation conducted by Geosyntec included five soil test borings (SPT-109 to SPT-113) and twelve CPT soundings (CPT-122 to CPT-126, CPT-128 to CPT-133, and CPT-130A). One of the soil borings (SPT-113) and three of the CPT soundings (CPT-131 to CPT-133) were advanced within the interior South Ash Pond and were terminated once native foundation materials were encountered. The remaining borings and soundings were conducted in the dike materials and, except as described below for SPT-110 and SPT-112, were terminated once refusal was encountered. Refusal was defined in the field as an SPT N-value of 50 blows per ft over an advancement of 6 inches (in.) or the inability to further advance the cone; refusal occurred at the top of the Chicora Member. Soil Consultants, Inc. (SCI) of Charleston, SC was the drilling subcontractor, and Mid-Atlantic Drilling, Inc. (MAD) of Wilmington, NC conducted the CPT soundings.

The four soil test borings drilled in the dike materials were advanced to a depth of 51 to 68 ft bgs using a CME-550X drill rig. Drilling was performed using the mud rotary wash method in general accordance with recommendations of Idriss and Boulanger (2008) (Table 1). Split-spoon samples and SPT blow counts (i.e., N-values) were generally collected in 5-ft depth intervals. Several thin-walled Shelby tube samples were also collected in the vicinity of the perimeter dikes. In two soil borings (SPT-110 and SPT-112), SCI replaced the side discharge drill bit with a tri-cone drill bit once the Chicora Member was encountered in order to penetrate the unit. The Chicora Member was slowly drilled through until the underlying Williamsburg Formation Clay was encountered, and then these borings were advanced an additional 5 ft before collecting a Shelby tube sample and terminating the borings. Boreholes located on the dike centerline were left open for two to three days prior to abandonment, and depths to water levels were recorded before the borings were plugged with a cement-bentonite grout.

Of the nine CPT soundings of the perimeter dike, six were advanced through the perimeter dike centerline and three CPT soundings were advanced at the dike toe. Shear wave velocity (V_S) testing was conducted at 5-ft depth intervals for three locations along the perimeter dike centerline (CPT-123, CPT-124, and CPT-129), two locations at the dike toe (CPT-125 and CPT-130A), and two locations within the impoundment interior (CPT-132 and CPT-133). Pore pressure dissipation tests were performed as well along the dike centerline (CPT-122 and CPT-129), dike toe (CPT-130A), and within the CCR (CPT-131 and CPT-133). Results of the V_s and pore pressure dissipation tests are included in Attachment 3 of the Safety Factor Assessment Report.

In March 2016, Geosyntec remobilized to WGS to conduct supplemental soil test borings and CPT soundings on the west corner of the South Ash Pond. Three soil test borings (SPT-302, SPT-303, and SPT-303A) were advanced by the mud rotary wash drilling method, four CPT soundings (CPT-204, CPT-205, CPT-206 and CPT-208) were advanced through the perimeter dike centerline, and one CPT sounding (CPT-207) was advanced at the dike toe. Two of the CPT soundings were conducted with V_s measurements. The purpose of the subsurface investigation was to: (i) collect physical samples of foundation soils immediately underlying the dike fill for geotechnical laboratory testing; (ii) further characterize the material properties of the observed soft clay foundation soil; and (iii) evaluate the relative density of dike fill soils.

SUBSURFACE STRATIGRAPHY AND COAL COMBUSTION RESIDUALS

Subsurface Stratigraphy

Unless noted, the subsurface stratigraphy for the South Ash Pond was developed based on the results of the previously discussed subsurface investigations. The general subsurface stratigraphy is described as follows:

- Dike Fill Soils: Soils along the South Ash Pond perimeter between the dike crest elevation of 38 ft National Geodetic Vertical Datum of 1929 (NGVD29) to the dike toe at 24.0 ft NGVD29 were considered dike fill soils. These soils were observed to be medium dense to very dense, poorly graded to silty sands. Uncorrected SPT blow counts typically ranged from 15 to 60 blows per foot, and CPT tip resistances typically ranged from 100 to 500 tons per square foot (tsf).

- Foundation Soils: Foundation soils were observed to be variable across the South Ash Pond footprint, consisting primarily of poorly graded to silty sands with shells and pockets of clayey sand to high plasticity clay. Uncorrected SPT blow counts within the sandy foundation soils typically ranged from 2 to 35 blows per foot, and CPT tip resistances typically ranged from 40 to 200 tsf. A 15 to 20-ft thick layer of soft clay, with uncorrected blow counts ranging from 0 to 4 blows per foot and CPT tip resistances below 20 tsf, was observed in the west to southwest corner of the South Ash Pond.

- Chicora Member: A layer of dense to very dense, partially cemented to heavily cemented shells was encountered beneath the foundations soils during the past subsurface investigations at WGS. Blow counts in this unit exceeded 50 blows over less than 6 in. of advancement, with minimal sample recovery without rock coring. In the two South Ash Pond borings that penetrated the Chicora Member, the layer was found to be between 5 ft and 8 ft thick. 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".

- Williamsburg Formation Clay: The Williamsburg Formation Clay was encountered beneath the Chicora Member and 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 unit was found to be between 30 ft and 90 ft thick in the vicinity of WGS from a review of the regional geology. Based on two SPTs, uncorrected SPT blow counts within this stratum ranged from 10 to 19 blows per foot in the upper 10 ft of the unit. In other areas of the Site, uncorrected SPT blow counts exceeding 20 blows per foot, increasing with depth, in the upper 20 ft of the unit.

Coal Combustion Residuals

CCR, primarily in the form of fly ash but also as boiler slag and bottom ash, have been stored within the South Ash Pond since 1980. One soil boring (SPT-113) and three CPT soundings (CPT-131 to CPT-133) were advanced to the pond bottom from finger dikes or working platforms extending into the surface impoundment interior. The South Ash Pond fly ash is described as follows:

- Fly Ash: Fly ash is typically described as a very soft (weight of hammer or weight of rod during an SPT), wet, black, slightly sandy silt-sized material without plasticity or with low plasticity. Samples collected from soil boring SPT-113 appeared to consist predominantly of coarser, black material resembling bottom ash. It was assumed that the area around SPT-113 was constructed by dumping and compacting bottom ash during operations at WGS. Blow counts were found to range between 0 (weight of hammer) and 3 blows per ft. CPT tip resistances were found to range between 5 and 40 tsf, with most values below 20 tsf.

PHREATIC SURFACE INTERPRETATION AND CURRENT WATER LEVELS

The phreatic surface in the vicinity of the South Ash Pond was evaluated from borehole water levels, CPT porewater pressure signatures, CPT dissipation testing, and surface water elevations from the South Ash Pond staff gauge. Water levels from rotary wash borings located on the dike centerline were generally collected 24 hours after borehole termination, and then daily afterwards until borehole abandonment. CPT soundings were advanced with a pore pressure transducer located behind the cone which recorded pore pressure measurements during advancement. These pore pressure signatures were interpreted to locate the phreatic surface at the time of each sounding. Dissipation tests were conducted at several locations and held for 5 to 20 minutes depending on the rate of pore pressure dissipation. During a dissipation test, excess pore pressures were allowed to dissipate to equilibrium or hydrostatic conditions, which can be applied to compute the phreatic surface. Table 2 summarizes phreatic surface elevations and data for the South Ash Pond during the 2013 and 2016 subsurface investigations. Because a CPT signature is often difficult to interpret due to noise exhibited by positive and negative excess pore pressures generated during cone advancement, if the CPT signature did not demonstrate a clear phreatic level, the sounding was excluded from Table 2. This was the case for CPT-130.

Dike Phreatic Surface

The elevation of the phreatic surface through the centerline of the perimeter dike varied from 21.6 to 34.6 ft NGVD29 during the 2013 investigation. The higher phreatic levels were generally observed on the western perimeter dikes of the South Ash Pond as surface water from the West Ash Pond was being sluiced into rim ditches in the western corner of the South Ash Pond. WGS subsequently regraded the West Ash Pond to gravity drain into the Slurry Pond and no longer pumps water from the West Ash Pond into the South Ash Pond. During the 2013 subsurface investigation, WGS was reconstructing the toe drains at the base of the northwestern perimeter dikes. The remaining toe drains appeared to be functioning and discharging into the exterior perimeter drainage ditches as designed.

During the 2016 subsurface investigation, the elevation of the phreatic surface through the centerline of the perimeter dike in the western corner of the South Ash Pond varied from 28.2 to 32.4 ft NGVD29.

Free Field (Dike Toe) Phreatic Surface

Because soil test borings were not advanced at the downstream toe of the perimeter dike for the South Ash Pond during the 2013 and 2016 subsurface investigations, CPT sounding and piezometer data was utilized to estimate the phreatic surface at the dike toe; the elevation of the phreatic surface ranged from 12.6 ft to 20.8 ft NGVD29. Based on the Hydrologic and Hydraulic Analysis (Attachment 1), the hydraulic gradient was determined to be in the southeast direction.

Surface Water Levels Since 2013

The South Ash Pond surface water level has been measured by a staff gauge (W-SW-SAP) over the past several years. WGS also measures the water level using a staff gauge within the drainage sump (PSG-6) located west of the South Ash Pond and within the Coal Pile stormwater ditch (PSG-7). As described in Attachment 1 of the Safety Factor Assessment Report, the surface water level with the South Ash Pond is maintained at an elevation of 28.73 ft NGVD29 by a concrete riser structure with wooden stop logs.

The South Ash Pond drains to the east, and water collects within the eastern interior of the pond. In the vicinity of W-SW-SAP and PPZ-SAP-1, the phreatic surface has reached an elevation of approximately 31.8 ft NGVD29.

For the Safety Factor Assessment Report, the "Maximum Normal Storage Pool" of the South Ash Pond was selected as 28.73 ft NGVD29, the elevation of the pond outlet structure, and the "Maximum Surcharge Pool" was selected as 31.8 ft NGVD29, the maximum phreatic surface recorded for the pond.

SPT AND CPT INTERPRETATION

SPT N-values and CPT soundings were processed and interpreted by the methods described below.

Standard Penetration Test

During an SPT, the number of "blows" or impacts from a standard, 140-lb hammer falling 30 in. needed to advance the split-spoon sampler 6 in. is recorded over three depth intervals for a total of 18 in. The last two 6-in. intervals are summed, and this value referred to as an "N-value". Due to variations in drill rigs, hammer efficiency, and sampling methods, the field or measured value is 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, or applies 60 percent of the theoretical maximum potential energy. The corrected N-value (N_{60}) may be computed as follows:

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

where:

N_{60} = corrected N-value 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.

Of these correction factors, the correction factor for the applied energy (C_E) is by far the most influential. This correction factor can be computed as follows:

$$C_E = \frac{ER}{60} \quad (2)$$

where:

ER = energy ratio of the SPT hammer.

SCI provided calibration records for the hammer system of the CME-550X drill rig used to advance the soils borings during the October 2013 investigation based on calibration tests performed offsite on April 3rd, 2013. An Energy Ratio (ER) of 88 percent was computed for this drill rig, and the calibration is provided in Table 3. Although the boring logs for piezometers PPZW-2D and PPZW-3D are not included or discussed in the Safety Factor Assessment Report, the rig used by South Atlantic Environmental Drilling and Construction Co. Inc. (SAEDACCO) to install these piezometers in November 2013 was calibrated on July 30th, 2013 by GRL Engineers, Inc. (GRLE), resulting in an ER = 87 percent (GRLE, 2013). The CME-55 drilling rig utilized by MAD during the March 2016 investigation was calibrated on August 19th, 2015 with an ER of 77.2 percent, as shown in Table 4. The ER for the equipment utilized in the S&ME (1978) investigation was estimated as 70 percent by comparing blow counts collected by Geosyntec with blow counts reported for these historical borings (See Attachment 5 of the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond* within the operating record for details).

Other correction factors were selected based on industry standards (Idriss and Boulanger, 2008) and are provided in Table 5. N_{60} was computed with correction factors for a 4-in. (100 mm) borehole 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 while considering a 5 ft stickup from the length of the drilling rod and anvil above the ground surface.

In many correlations and for liquefaction analysis, N_{60} is normalized based on 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:

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

where:

C_N = stress normalization parameter.

The stress normalization parameter (C_N) can be computed 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 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

CPT soundings performed onsite measured the cone tip resistance (q_c), the sleeve friction (f_s), and the pore pressure (u_2) values in 0.05 m (≈ 2 in.) intervals. However, the measured 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 can be computed as follows:

$$q_t = q_c + (1 - a_n)u_2 \tag{5}$$

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 prior to processing each CPT sounding.

The Soil Behavior Type Index (I_c) (Robertson and Cabal, 2012) was calculated using the normalized cone tip resistance and normalized sleeve friction ratio. The normalized cone tip resistance (Q) is computed as follows:

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

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 depended on soil type and stress level.

The normalized sleeve friction ratio is calculated as follows:

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

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 I_c is calculated as follows:

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

The normalized cone tip resistance and normalized friction ratio may be plotted on the Normalized Soil Behavior Type (SBT_N) Chart, as presented in Figure 2. Additionally, Figure 2 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 Geosyntec's interpretation is presented in Figure 3 for CPT-122.

LABORATORY TESTING PROGRAM

In 2013, 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, foundation soils, and CCR. The geotechnical laboratory testing program included index (grain size distribution, Atterberg limits, natural water content), shear strength, one dimensional (1-D) consolidation, and unit weight testing. Appendix 1 summarizes the index, unit weight, and strength test results from Geosyntec's subsurface investigation and the historical investigation by SM&E. The raw laboratory testing data is provided in Attachment 4 of the Safety Factor Assessment Report. Results from the laboratory testing programs are discussed further below.

Index Testing

Dike Fill, Foundation Soils, and Williamsburg Formation Clay

During the 2013 and 2016 investigations, index testing programs were conducted on the dike fill, foundation soils, and Williamsburg Formation Clay strata that included a total of 14 grain size distribution tests, seven of which included hydrometer testing to evaluate the portion of the material passing the #200 sieve. Test results from Geosyntec and SM&E for grain size distribution, fines content, and natural moisture content are provided in Figures 4, 5, and 6, respectively. Grain size testing indicated that dike fill soils typically consist of 60 percent to 91 percent sand-sized material (smaller than No. 4 sieve but greater than No. 200 sieve) and 10 percent to 40 percent silt and clay-sized material (percent fines), with most samples containing 5 percent to 20 percent fines by weight.

Foundation soils were observed to be variable across the South Ash Pond with pockets of clayey sand to high plasticity clay and shell hash among layers of poorly graded to silty sands. The poorly graded and silty sands were generally composed of 60 percent to 90 percent sand-sized material with 15 percent to 25 percent fines. Some samples of foundation material were described as resembling “shell hash” and contained many shells and fine gravel which constituted between 17 percent and 35 percent of the sample by weight. Where clay was encountered the fines content was higher, and in isolated areas the foundation materials are relatively clean sands (<10 percent fines).

From the two grain size tests performed on the Williamsburg Formation Clay, the unit consists of approximately 40 percent to 50 percent clay and 40 percent silt-sized particles with approximately 10 percent to 20 percent sand-sized particles. Additional fines content testing was performed on the material to supplement grain size testing and provide additional data for liquefaction potential analyses.

Natural moisture content was typically determined for samples during both the 2013 and 2016 investigations. Natural moisture contents of the subsurface soils typically ranged between 15 percent and 30 percent within the sandy foundation soils and 70 percent to 100 percent within the clayey foundation soils. Within the dike fill, natural moisture contents typically ranged between 10 percent and 25 percent.

Finally, Geosyntec conducted a total of seven Atterberg limit tests on the clay material found in the foundation soils beneath the perimeter dikes. Liquid limits of this clay material ranged from 108 to 151, and plasticity indices ranged from 70 to 96. One Atterberg limit test was performed on a predominantly sandy material and was found to be non-plastic. The SM&E (1978) subsurface investigation did not include Atterberg limit testing of the foundation materials.

Fly Ash

Index testing of fly ash was performed on two split-spoon samples and one Shelby tube sample collected from SPT-113. Collection of additional samples was attempted, but advancement of the split spoon sampler yielded no recovery when weight of hammer (0 blows per 18 inches) material was encountered. The two samples contained 62 percent and 81 percent sand-sized material and 31 percent and 19 percent fines, respectively. Field observations indicated that this material more closely resembled bottom ash that was used to construct the working platform than sluiced fly ash. The Shelby tube sample (< 5" recovery) was fly ash material and was tested for Atterberg limits, grain size distribution and pH (ASTM D 4792). This sample was found to be non-plastic, contain 23.6 percent and 43.8 percent sand and silt-sized material, respectively, and have a pH of 5.7.

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 tubes collected within foundation soils. Since the dike fill soils were observed to consist of dense, silty to poorly graded sands, Shelby tubes were not able to be collected. However, the unit weight was estimated using V_s measurements discussed later within this calculation package, with a representative total unit weight of 120 pcf being selected for design. Thin-walled Shelby tube samples collected in soil borings SPT-109, SPT-110, and SPT-302 indicated that the sandy foundation soils have a total unit weight ranging from 119 to 133 pcf; a representative value of 123 pcf was selected for design. Shelby tube samples were collected within clay foundation soils in SPT-112 and SPT-303A and supplemented with clay foundation data from historical S&ME (1978) borings SC-68 and SC-76; the combined data had a total unit weight ranging from 86 to 105 pcf; a representative value of 94 pcf was selected for design.

Fly Ash

The dry unit weight and initial moisture content were measured as part of the shear strength testing and consolidation testing for two samples collected at the interior of Ash Pond A and were used for the parameters of the fly ash in the South Ash Pond. The total unit weight was calculated using the measured dry unit weight and initial moisture content. The results indicate that the total unit weight of the residual fly ash ranges from 100 to 111 pcf; a value of 100 pcf was selected for design (See Attachment 5 of *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* within the operating record for details).

Williamsburg Formation Clay

One unit weight value was computed from a hydraulic conductivity test performed on a Shelby tube collected within the Williamsburg Formation Clay, which yielded a computed total unit weight of 104.9 pcf; a value of 105 pcf was selected for design.

Undrained Shear Strength

Consolidated undrained (CU) triaxial tests were performed on extruded thin-walled Shelby tube samples from foundation soils, as Shelby tubes were not collected within sandy dike fill soils. CU tests were performed on two samples from the foundation soils during the 2013 investigation and two samples collected during the 2016 subsurface investigation. Additionally, two CU and two unconsolidated undrained (UU) tests were performed by S&ME (1978). A description of the CU and UU tests and their interpretation are presented herein.

Methodology

During (CU and UU) triaxial tests, a soil sample is usually trimmed into two to three specimens (depending on the Shelby tube recovery), and each specimen is tested under a different initial confining (or consolidation) 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 during collection. 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 the specimens were subjected to in the field. Therefore, the measured S_u from each CU test cannot be used directly in analysis. However, a relationship between the S_u in the field and the S_u established 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.

UU tests are not re-consolidated prior to shearing, and the confining stress in the field is the same as within the laboratory. The undrained shear strength computed by this test can be applied directly at the depth in which the sample was collected. The UU tests were also considered when evaluating strength parameters for clayey materials beneath the South Ash Pond.

Foundation Soils

The undrained shear strength ratio was calculated for each test based on the calculated shear stress from each data point. The undrained shear strength ratios for four samples on foundation materials are provided in Figure 8. An *OCR* of 1.0 was selected to apply the appropriate correction factor discussed above. Undrained shear strength ratios range from 0.30 to 2.28 for foundation soils classified as silty or clayey sands. Additionally, two CU tests and two unconsolidated undrained (UU) tests were performed by S&ME (1978) on Shelby tube samples from borings identified as SC-76 through SC-78. These resulting shear strength ratios for these tests range from 0.32 to 3.38, and were plotted on Figure 8. When considering both historical and current data, an undrained shear strength ratio of 0.3 was viewed as the lower bound of the clayey materials over the current range of in-situ stresses within the vicinity of the perimeter dikes.

Fly Ash

Fly ash in the South Ash Pond is reasonably similar to the fly ash in Ash Pond A. Two sets of 3-point CU tests were conducted on thin-walled Shelby tube samples consisting of fly ash in Ash Pond A. The undrained shear strength ratio was also calculated for each test based on the calculated peak deviator stress from each point. The test results indicate that undrained shear strength ratios range from 0.98 to 6.93 for the residual fly ash in Ash Pond A, which is also applicable to the residual fly ash in the South Ash Pond (See Attachment 5 of *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* within the operating record for details).

Post-Liquefaction Dike Fill

A 1 ft layer of liquefiable material within the Dike Fill was identified within the liquefaction analysis of this report. A residual strength effective cohesion of 30 psf was selected based on this analysis (Attachment 7).

Drained Shear Strength

Foundation Materials

The effective-stress friction angles (ϕ′) and cohesion intercepts (c′) of the clay foundation soil samples collected by Geosyntec were estimated based on the CU test results and supplemented with historical data plotted by S&ME (1978). Geosyntec and S&ME data was in general agreement, with a (c′, ϕ′) envelope of (300 psi, 15°) encompassing the combined data set. The Mohr’s circles for clay foundation soils are plotted in Figure 9 and the selected strength envelope is plotted in Figure 10.

The effective-stress friction angles (ϕ′) and cohesion intercepts (c′) of the silty sand to clayey sand foundation soils collected by Geosyntec were estimated based on the CU test results. The ϕ′ and c′ were calculated using the effective stress Mohr circle at failure for each CU test. Effective friction angles ranged from 29.2° to 41.8° with effective cohesion intercepts of 0 psi. The Mohr’s circles for silty sand to clayey sand foundation soils are plotted in Figure 11, and the selected strength envelope is plotted in Figure 12.

Fly Ash

Fly ash parameters were obtained using the analysis conducted for Ash Pond A on a similar fly ash media. Based on the CU test results, the ϕ′ and c′ for residual fly ash contained were estimated to be 34° and 0 psf, respectively (See Attachment 5 of *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* within the operating record for details).

Consolidation Test Interpretation

Foundation Soils

During Geosyntec’s 2013 investigation, one-dimensional (1-D) consolidation tests were performed on two samples of clayey sand foundation materials (SPT-109 and SPT-110) along the southern perimeter dikes of the South Ash Pond and one sample within the clay foundation materials (SPT-112) in the northern perimeter dikes of the South Ash Pond. Additionally, two historical consolidation tests were performed by S&ME (1978) on the clay foundation materials collected from SC-68, which is located beneath the northern perimeter dikes, and SC-78, which is located near the SPT-109. The computed preconsolidation pressures (σ_p') ranged from 4,000 to 5,000 psf and 1,800 to 3,200 psf for tests conducted on sandy clay and clay respectively. The strain verses applied load for each test is plotted in Figure 13. The overconsolidation ratio (OCR), which is the ratio of (σ_p') to the in-situ vertical effective stress, was calculated and ranged between 1 and 3.

The modified compression index ($C_{c\varepsilon}$) and modified recompression index ($C_{r\varepsilon}$) were calculated from each 1-D consolidation test. The $C_{c\varepsilon}$ ranged from 0.068 to 0.105 and 0.44 to 0.591 for clayey sand and clay, respectively. Meanwhile the $C_{r\varepsilon}$ ranged from 0.0065 to 0.0120 and 0.0067 to 0.0175 for sandy clay and clay, respectively. Additionally, 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 stress ratio (σ_V'/σ_P'). Figures 14 and 15 display C_V parameters for the clayey sand and clay foundation soils, respectively; Figure 16 shows the $C_{\alpha\varepsilon}$ parameters for both foundation media. Furthermore, the C_V and $C_{\alpha\varepsilon}$ values were selected for stress ratios less than 1.0 and stress ratios greater than 1.0. Representative values of C_V were selected for sandy clay as 8.4 ft^2/day (9.0 mm^2/s) and 4.7 ft^2/day (5.0 mm^2/s) for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Likewise for $C_{\alpha\varepsilon}$, representative values for clayey sand of 0.075 percent and 0.20 percent were selected for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Representative values of C_V were selected for clay as 0.70 ft^2/day (0.75 mm^2/s) and 0.23 ft^2/day (0.25 mm^2/s) for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Likewise for $C_{\alpha\varepsilon}$, representative values for clay of 0.075 percent and 1.0 percent were selected for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively.

Fly Ash

A 1-D consolidation test was conducted on a thin-walled Shelby tube sample collected from the interior of Ash Pond A and used for the parameters of the fly ash in the South Ash Pond. The σ_P' estimated during this test was 11,000 psf. The strain versus applied load variation is plotted 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 (See Attachment 5 of *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A* within the operating record for details).

Hydraulic Conductivity

One hydraulic conductivity test (ASTM D 5084) was performed from a Shelby Tube advanced into the Williamsburg Formation Clay. This test yielded a hydraulic conductivity (k) of 1.8×10^{-8} cm/s.

IN-SITU TESTING INTERPRETATION

Correlations were applied to in-situ testing data (q_t, f_s, N_{60}, etc.) and compared with laboratory test data during the selection of material parameters. Furthermore, in-situ measurements of the shear wave velocity (V_S) and pore pressure dissipation were performed at several locations along the perimeter dike centerline and dike toe. The following section describes the methodology and correlations applied to interpret in-situ testing performed at the site. The correlations are applied later in this calculation package.

Shear Wave Velocity

Shear wave velocity (V_S) 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 correlations based on CPT sounding data from adjacent soundings. Mayne (2006) provides a correlation to shear wave velocity for saturated sands, clays, and silts, as follows:

$$V_s = 118.8 \log(f_s) + 18.5 \tag{9}$$

where:

V_S = shear wave velocity (meters per second (m/s)); and

f_S = sleeve friction (kilopascal (kPa)).

Pore Pressure Dissipation Tests

The CPT cone measures the porewater pressure within the soil as the cone is pushed through the subsurface. A dissipation test is conducted by measuring the pore pressure over time once advancement of the cone has ceased. The excess pore pressures generated during advancement of the cone dissipate to hydrostatic conditions over time. The rate of excess pore pressure dissipation can be interpreted to estimate the lateral coefficient of consolidation of the soil (c_H).

Hydrostatic conditions were estimated from the recorded pore pressures toward the end of the dissipation. The measured pore pressure were converted to a height of water column above the cone and added to the depth of cone to estimate the phreatic surface.

Drained Friction Angle

SPT N-values and CPT soundings were utilized to estimate the drained peak effective-stress friction angle of subsurface soils. The correlation presented by Hatanaka and Uchida (1996) was applied to estimate the peak friction angle of natural sands layers that are relatively clean, as follows:

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

ϕ' = effective stress friction angle (°); 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. Undrained shear strength for cohesive materials (i.e., silts and clays) may be estimated from CPT tests 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 ratio;
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.

Depending on material type, shear mode, and other factors, N_{kt} typically varies between 10 and 20 (FHWA, 2002), although N_{kt} values as low as 6 have been recorded in sensitive fine-grained soils (Robertson and Cabal, 2012). For projects which have laboratory shear strength measurements available, a site-specific value of N_{kt} may be developed based on laboratory measurements of S_u (Robertson and Cabal, 2012). Based on Table 33 in FHWA (2002), and correlation of CPT sounding results with the laboratory shear strength testing performed on site soils (CU and UU triaxial tests), a site-specific value of 10 was selected for N_{kt}. The estimated values of undrained shear strengths developed from CPT data using the site-specific value of N_{kt} were generally found to be lower than measurements of S_u from triaxial compression tests conducted on soil samples from nearby geotechnical borings.

N_{60} from CPT Soundings

CPT sounding data has been correlated to SPT N_{60} value in order to compute an "equivalent N_{60}" profile. This correlation was applied in calibrating current borings and CPT soundings with historic borings from the S&ME investigation. The correlation from Robertson and Cabal (2012) is as follows:

$$\frac{(q_t/p_a)}{N_{60}} = 10^{(1.1268-0.2817I_c)} \tag{12}$$

where:

N_{60} = corrected N-value to 60 percent efficiency (blows/ft)
q_t = corrected tip resistance (tsf);
p_a = atmospheric pressure (tsf); and
I_c = soil behavior type index.

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:

$$\tan\phi' = \frac{1}{2.68}\left[\log\frac{q_c}{\sigma'_{vo}} + 0.29\right] \tag{13}$$

where:

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

RECOMMENDED MATERIAL PROPERTIES

The following paragraphs describe the selected parameters for analysis of the perimeter dikes surrounding the South Ash Pond. Table 6 summarizes the general parameters 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 selected γ_t envelopes. A γ_t of 94 pcf was selected for the clayey foundation soils, generally encountered below 5 ft NGVD29. A γ_t of 123, 120, and 105 pcf was recommended for the sandy foundation, dike fill, and Williamsburg formation materials, respectively. Following the data interpretation performed in Ash Pond A, a unit weight of 100 pcf was recommended for fly ash within Ash Pond A.

Undrained Shear Strength

Based on undrained shear strength ratios estimated based on CU testing, a typical S_u/σ_c' value of 0.3 was 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 varies throughout the South Ash Pond. For the safety factor assessment (Attachment 8), the closest CPT and laboratory data at each evaluated cross section were used to select the undrained shear strength.

A residual strength with effective cohesion of 30 psf was selected for the identified potentially liquefiable sand layer and applied to the relevant cross-section for the post-liquefaction safety factor case (Attachment 8).

Drained Shear Strength

In general, estimated drained strength parameters exhibited significant variability across the South Ash Pond dike fill and foundation soils, as shown in Figures 18 through 20. The effective friction angle ranges from 23.8° to 60.0° within the dike fill soils and 26.7° to 45.0° within the sandy foundation soils. Drained shear strength parameters were selected on a cross section by cross section basis during the safety factor assessment (Attachment 8). An effective friction angle of 15° with an effective cohesion intercept of 300 psf was selected for the clay foundation soils.

Consolidation Parameters

The following parameters were selected for clayey sand and clay materials based on the lab testing provided. The both materials are assumed to be normally consolidated (OCR = 1.0). The average modified compression index of 0.087 and 0.50 were selected for clayey sand and clay, respectively. The average modified recompression index of 0.0095 and 0.012 were selected for sandy clay and clay, respectively. For the clayey sand, representative C_V values of 8.4 ft^2/day (9.0 mm^2/s) and 4.7 ft^2/day (5.0 mm^2/s) were selected for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Representative value of 0.70 ft^2/day (0.75 mm^2/s) and 0.23 ft^2/day (0.25 mm^2/s) were selected for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively, for C_V of clay based on two tests from S&ME (1978) and a test from SPT-112. Likewise for $C_{\alpha\varepsilon}$, representative values of 0.075 and 0.20 percent were selected for sandy clay for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. Meanwhile for $C_{\alpha\varepsilon}$ of clay, representative values of 0.075 and 1.0 percent were selected for sandy clay for stress ratios less than 1.0 and stress ratios greater than 1.0, respectively. A representative C_V value of 0.74 ft^2/day (8.00 mm^2/s) was selected for the fly ash based on the conclusions shown in Attachment 5 of *2016*

*Surface Impoundment Periodic Safety Factor Assessment Report: Ash Pond A (*see the operating record for details).

Representative Subsurface Profiles for Site Response Analysis

Shear wave velocities profiles, soil plasticity, and unit weight are input parameters for site response analyses presented in Attachment 6 of this Safety Factor Assessment Report. Therefore, two representative profiles were developed for sections of the perimeter dike structures based on the height of the perimeter dikes and underlying soils. One representative profile was developed considering a thick zone of highly plastic clay overlying the Chicora stratum; while, the second considers sandy non-plastic soils in this zone. Shear wave velocity profiles were developed from seismic in-situ CPT performed in 5-ft depth intervals. Additionally, shear wave profiles were developed via correlation to CPT sleeve friction (by Equation 9). The raw data and interpretation of these tests is provided in Sub-Attachment 3B to the Safety Factor Assessment Report. The developed V_S profiles (by elevation) are provided in Figures 20 and 21, and summarized within Table 7 (note that shear wave profiles developed from seismic in-situ CPT profiles are designated as “sCPT” data points). Furthermore, selection of the shear wave velocity of the Chicora and Williamsburg Formation Clay strata is discussed in Attachment 6 of this Safety Factor Assessment Report.

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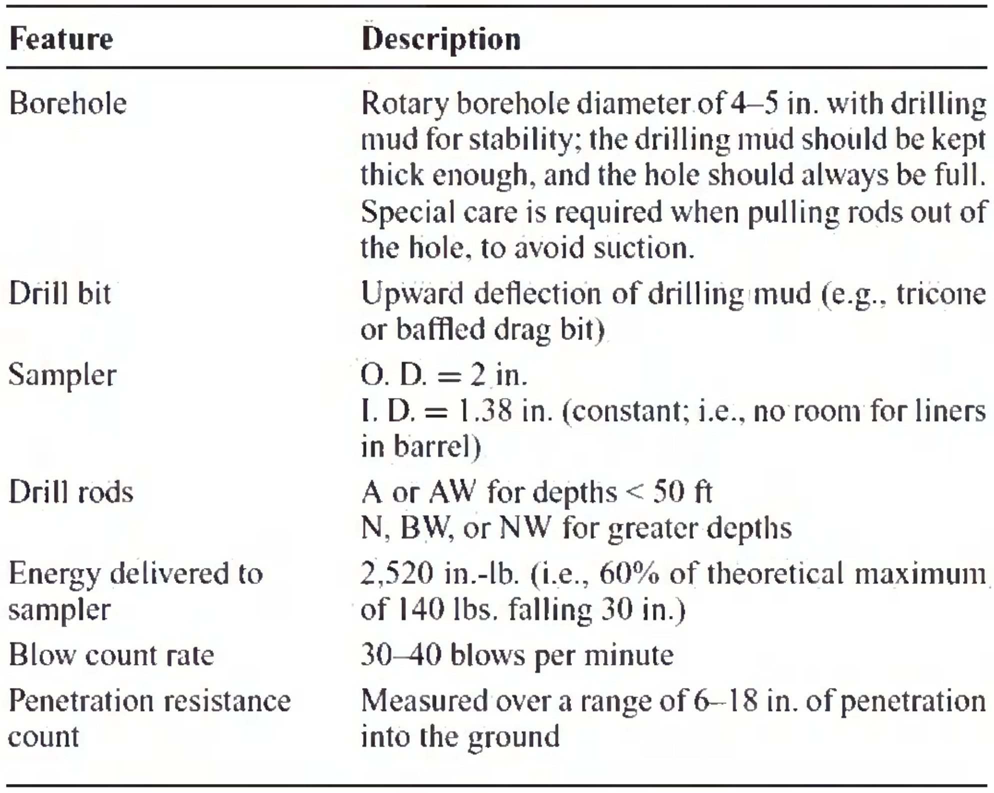
TABLES

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

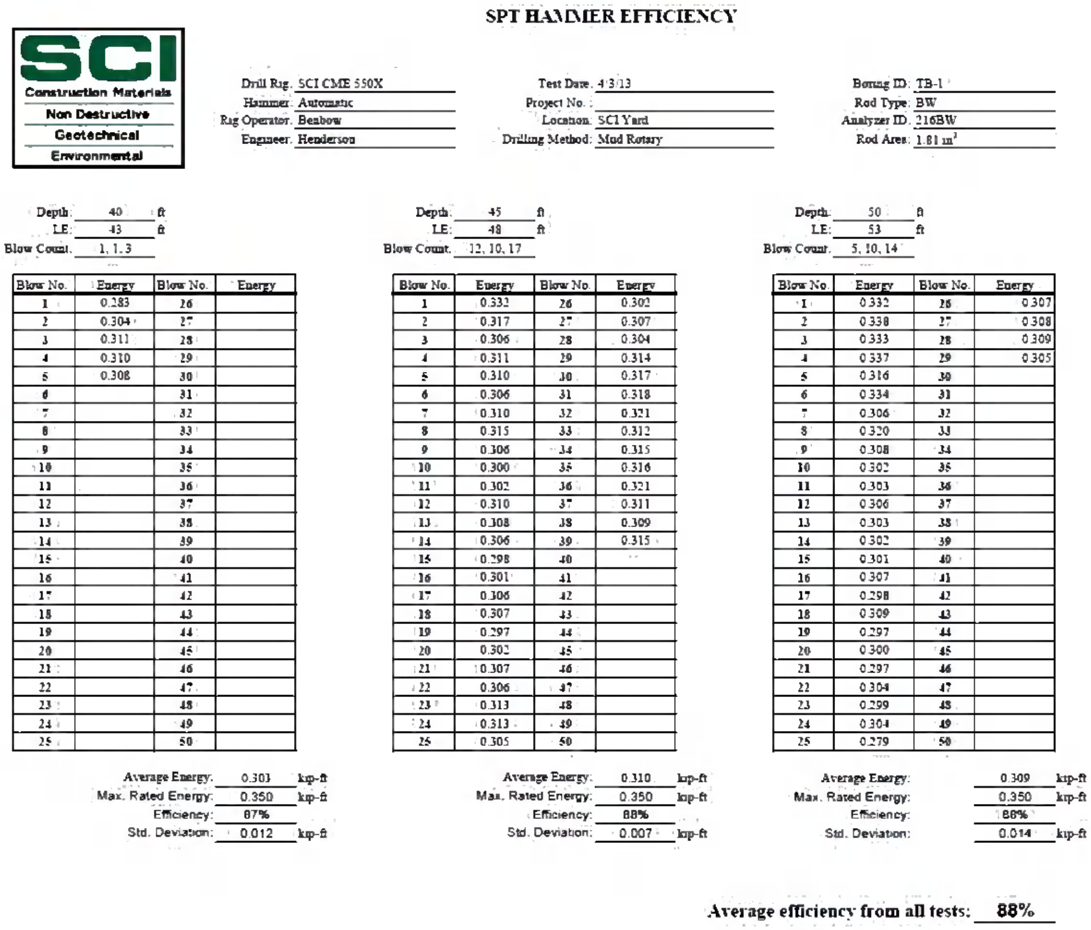
Table 2. Summary of Water Level Measurements

Boring ID	Method	Location	Ground Surface El. [3]	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-122	Diss. Test	Centerline	38.82	-	30.05	21.59	30.36
CPT-123	u_2 Signature	Centerline	37.84	8.00	-	-	29.84
CPT-124	u_2 Signature	Centerline	38.39	13.50	-	-	24.89
CPT-125	u_2 Signature	Dike Toe	23.51	3.00	-	-	20.51
CPT-126	u_2 Signature	Centerline	38.63	17.00			21.63
CPT-129	Diss. Test	Centerline	38.88	-	30.51	17.39	25.76
CPT-130A	Diss. Test	Dike Toe	23.08	-	15.91	10.68	17.86
CPT-131	Diss. Test	Pond Interior	42.25	-	14.93	7.30	34.62
CPT-131	Diss. Test	Pond Interior	42.25	-	24.93	17.53	34.85
CPT-133	Diss. Test	Pond Interior	38.58	-	15.42	8.14	31.30
CPT-133	Diss. Test	Pond Interior	38.58	-	25.43	17.88	31.03
CPT-204	u_2 Signature	Centerline	36.62	8.00	-	-	28.62
CPT-205	u_2 Signature	Centerline	37.99	9.64	-	-	28.35
CPT-206	u_2 Signature	Centerline	38.09	9.64	-	-	28.45
CPT-207	u_2 Signature	Dike Toe	23.06	5.40	-	-	17.66
CPT-208	u_2 Signature	Centerline	38.23	10.00	-	-	28.23
SPT-109	Borehole	Centerline	37.39	5.10	-	-	32.29
SPT-110	Borehole	Centerline	38.72	11.00	-	-	27.72
SPT-111	Borehole	Centerline	39.41	10.10	-	-	29.31
SPT-112	Borehole	Centerline	37.66	5.30	-	-	32.36
SPT-113	Borehole	Pond Interior	42.27	8.40	-	-	33.87
SPT-302	Borehole	Centerline	38.01	5.66	-	-	32.35
SPT-303A	Borehole	Centerline	37.48	5.43	-	-	32.05
PPZ-SAPS-1	Piezometer	Pond Interior	41.16	-	-	-	32.09
WAP-2	MW	Dike Toe	24.57	-	-	-	21.27
PPZW-2D	MW	Dike Toe	21.75	-	-	-	22.17
WAP-3	MW	Dike Toe	20.30	-	-	-	14.69
PPZW-3D	MW	Dike Toe	21.10	-	-	-	14.41
PPZ-14	MW	Dike Toe	27.27	-	-	-	22.34

Notes:

1. Depth to water levels in mud rotary boreholes may not be representative of existing conditions due to borehole collapse or the influence on drilling mud on measured depth to water levels.
2. Piezometer and monitoring well (MW) water levels were measured between February 2015 and May 2015.
3. For the piezometer and monitoring wells (MW), top of casing (TOC) elevation is provided.

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



SPT HAMMER EFFICIENCY

SCI
Construction Materials
Non Destructive
Geotechnical
Environmental

Drill Rig: SCI CME 550X
Hammer: Automatic
Rig Operator: Beabow
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 in^2

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 (provided Mid Atlantic Drilling, Inc.) for March 2016 Investigation

Rig 5 CME 45C Track Rig (Serial #273964)										
				Blows						
	LE	LP	Increment 1 (0-6")	Increment 2 (6-12")	Increment 3 (12-18")	N-Value	BPM	AET (k-ft)	ETR	Soil Classification
Test 1	35.5	30.0	7	16	15	31	47.4	0.261	74.6	SC
Test 2	37.5	32.0	10	13	13	26	49.8	0.275	78.6	SP
Test 3	39.5	34.0	4	6	17	23	49.1	0.275	78.4	SP

LE	Length Below Gages	BPM	Blows Per Minute	AET	Average Energy Transfer
LP	Length of Penetration	ETR	Energy Transfer Ratio		

Note:

1. Average values of the three tests result in an ER = 77.2 percent.

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 $C_E = 0.5–1.0$ 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): $C_S = 1.1 \quad \text{for} \quad (N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100} \quad \text{for} \quad 10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3 \quad \text{for} \quad (N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Table 6. Selected Material Parameters for Analysis

Material	γ_t (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 (mm^2/s)	OCR
Dike Fill	120	Varies[2]	-	-	-	-	-	-	-	-
Dike Fill (Post-Liquefaction)	120	0	30							
Foundation Soils (Clayey)	94	15	300	Varies[2]	300	0.52	0.012	0.010	0.25	1.0
Foundation Soils (Clayey Sands)	123	Varies[2]	0	-	-	0.087	0.0095	0.002	5.0	1.0
Chicora	130	50	0	-	-	-	-	-	-	-
Williamsburg Formation Clay [3]	105	50	0	-	-	-	-	-	-	-
Fly Ash	100	34	0							
Riprap Buttress	150	45	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 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 are not anticipated to pass through this zone. 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.

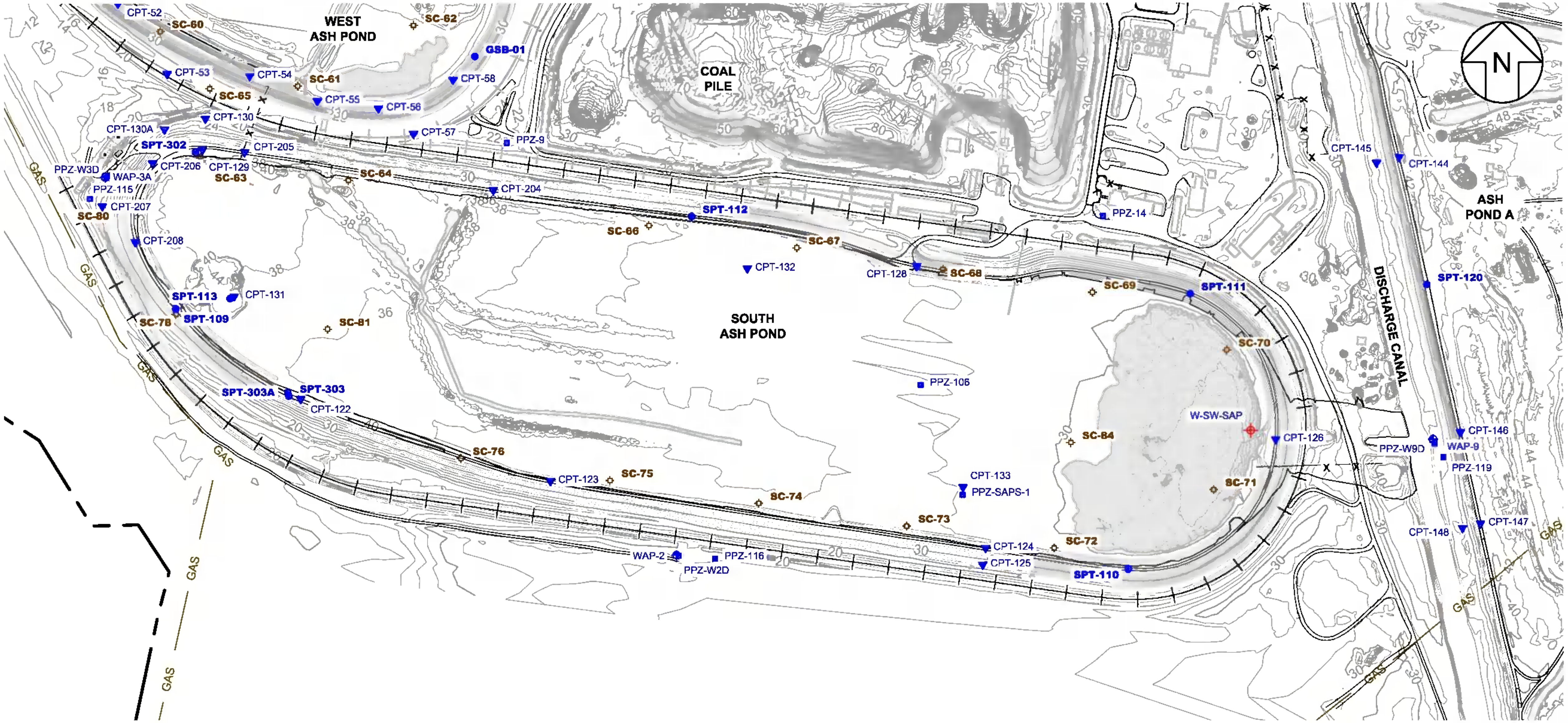
Table 7. Summary of Representative Shear Wave Profiles

Profile 1 (Dike Centerline)		Profile 2 (Dike Centerline)	
Elev. (ft)	V_s (ft/s)	Elev. (ft)	V_s (ft/s)
-8 to 3	400	-10 to 21	600
3 to 21	600		
21 to 35	800	21 to 35	1000
35 to 40	500	35 to 40	700

Notes:

1. Elevations are provided in NGVD29.
2. Shear wave velocities at elevations below -10 ft NGVD29 are discussed in Attachment 6.
3. Representative Profile 1 corresponds to a perimeter dike with foundation that includes a thick zone of highly plastic clay overlying the Chicora stratum, and Representative Profile 2 corresponds to a perimeter dike with foundation excludes this clay layer.

FIGURES



LEGEND

GAS ——— GAS ——— EXISTING GAS LINE

——— 10 ——— EXISTING MAJOR GRADE CONTOUR

—+—+—+— EXISTING RAILROAD

▨ EXISTING PONDED WATER (28.73 NVGD) (2016)

✛ W-SW-SAP EXISTING STAFF GAUGE

▼ CPT-52 GEOSYNTEC CONE PENETRATION TEST

● **SPT-110** GEOSYNTEC SOIL BORING

✧ **SC-60** HISTORICAL BORING

◆ WAP-9 MONITORING WELL

■ PPZ-9, PPZ-SAPS-1, PPZ-W3D 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.
4. HISTORICAL BORING SC-77 IS NOT SHOWN ON THIS MAP AS ITS LOCATION IS NOT KNOWN. IT IS ASSUMED TO BE A SOUTH ASH POND BORING BASED ON ITS NUMERICAL SEQUENCE RELATED TO OTHER SOUTH ASH POND BORINGS.

0 400' 800'

SCALE IN FEET

SOUTH ASH POND
BORING LOCATION MAP

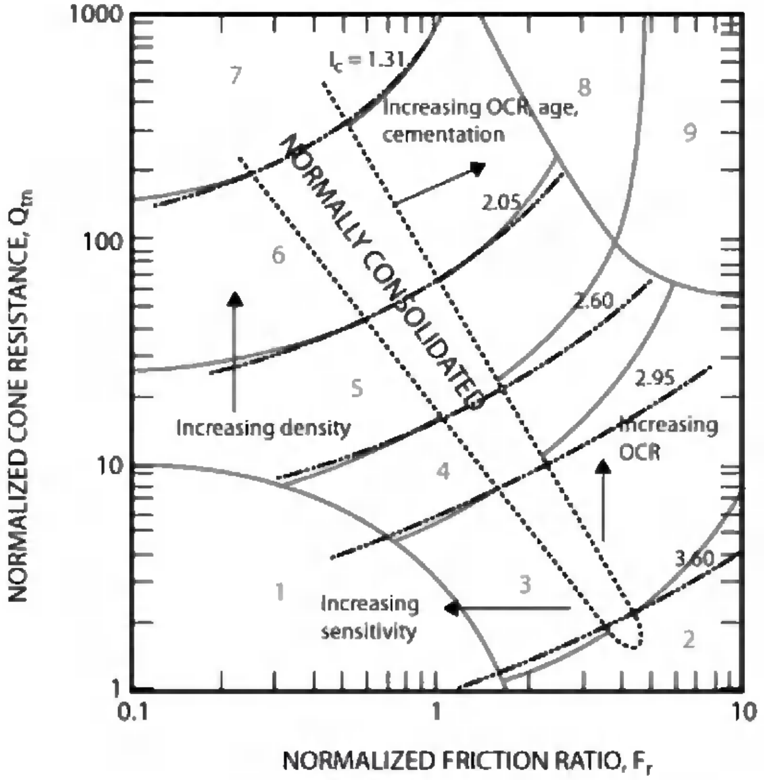
Geosyntec consultants

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-F0028-035



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)

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

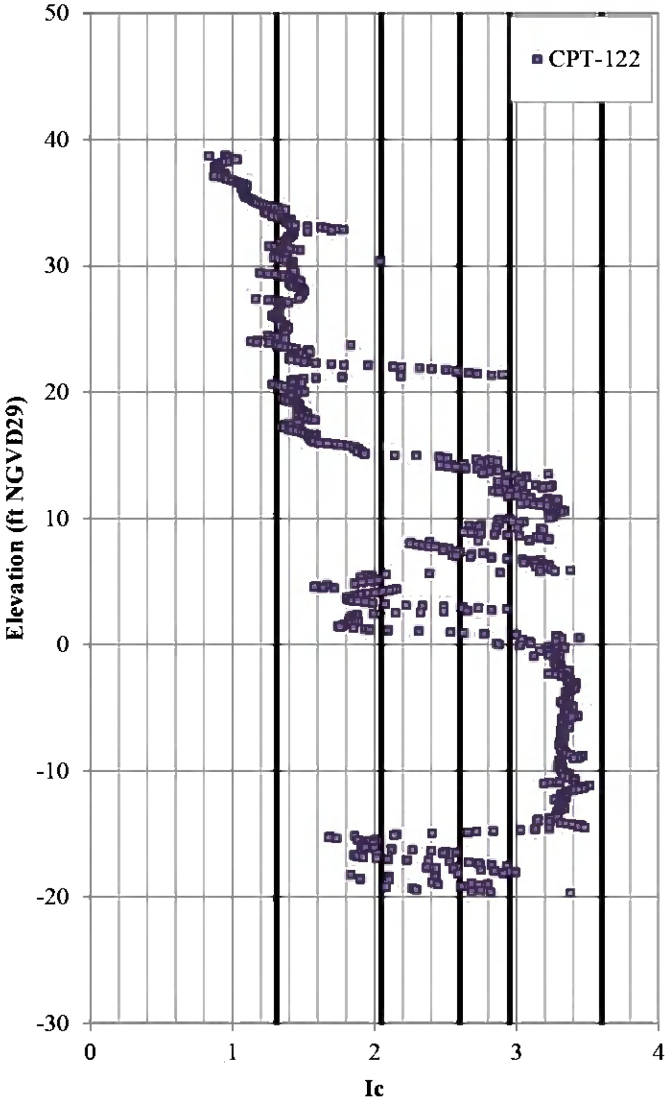

Figure 3. Example I_c classification profile for CPT-122

Note:

1. I_c – Soil Behavior Index by Robertson and Cabal (2012).

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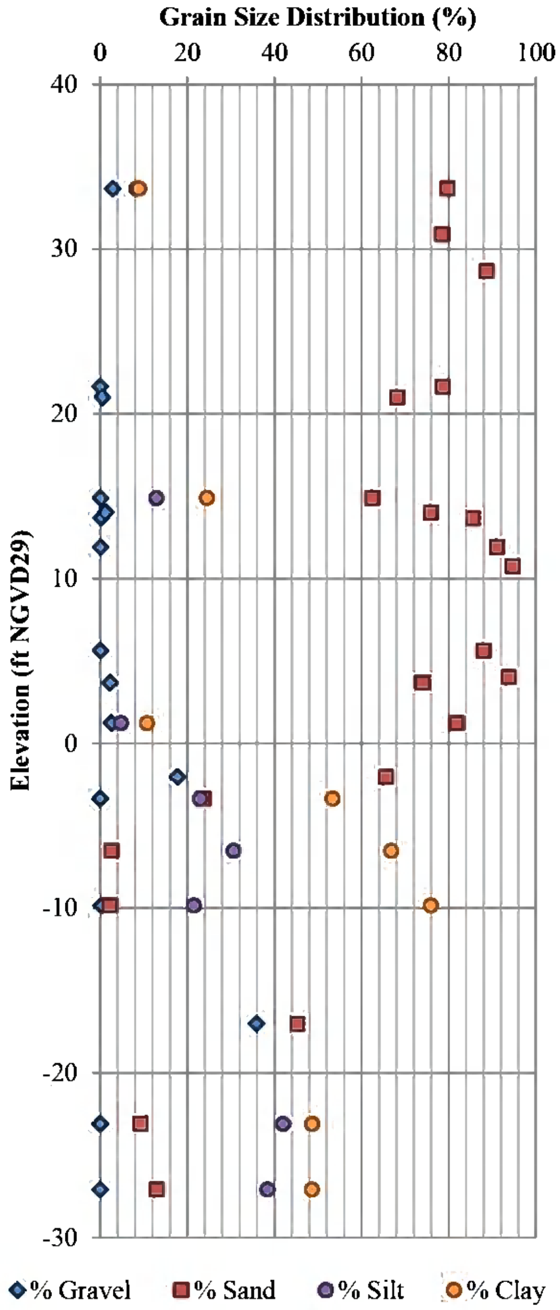

Figure 4. Geosyntec Grain Size Distribution Test Results

Note:

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

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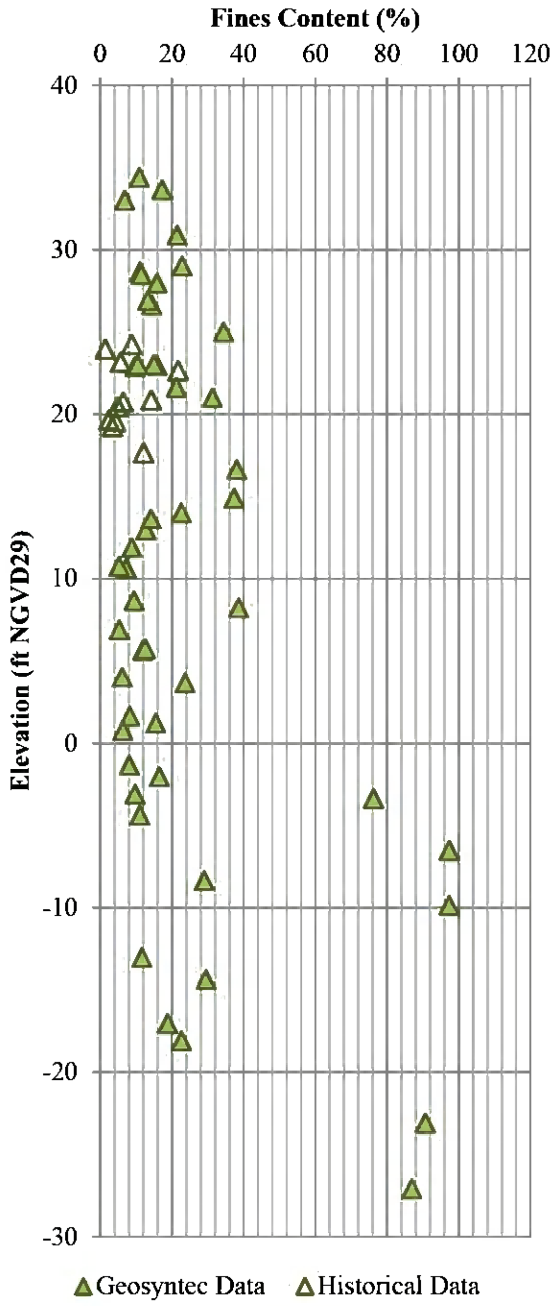

Figure 5. Geosyntec and Historical Fines Content Test Results

Note:

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

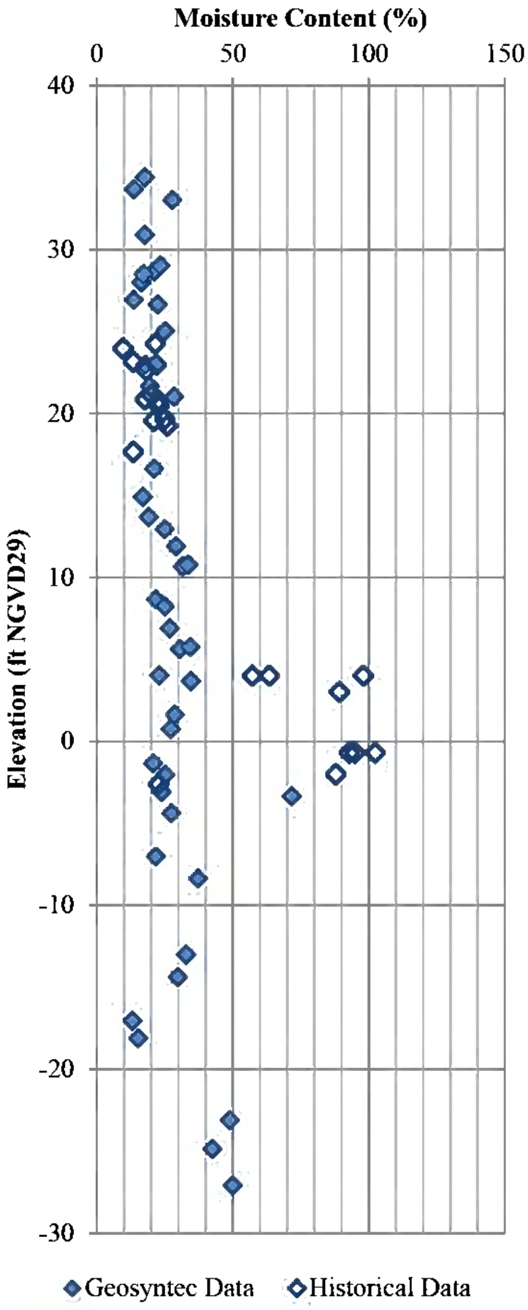

Figure 6. Geosyntec and Historical Natural Moisture Content Test Results

Note:

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

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

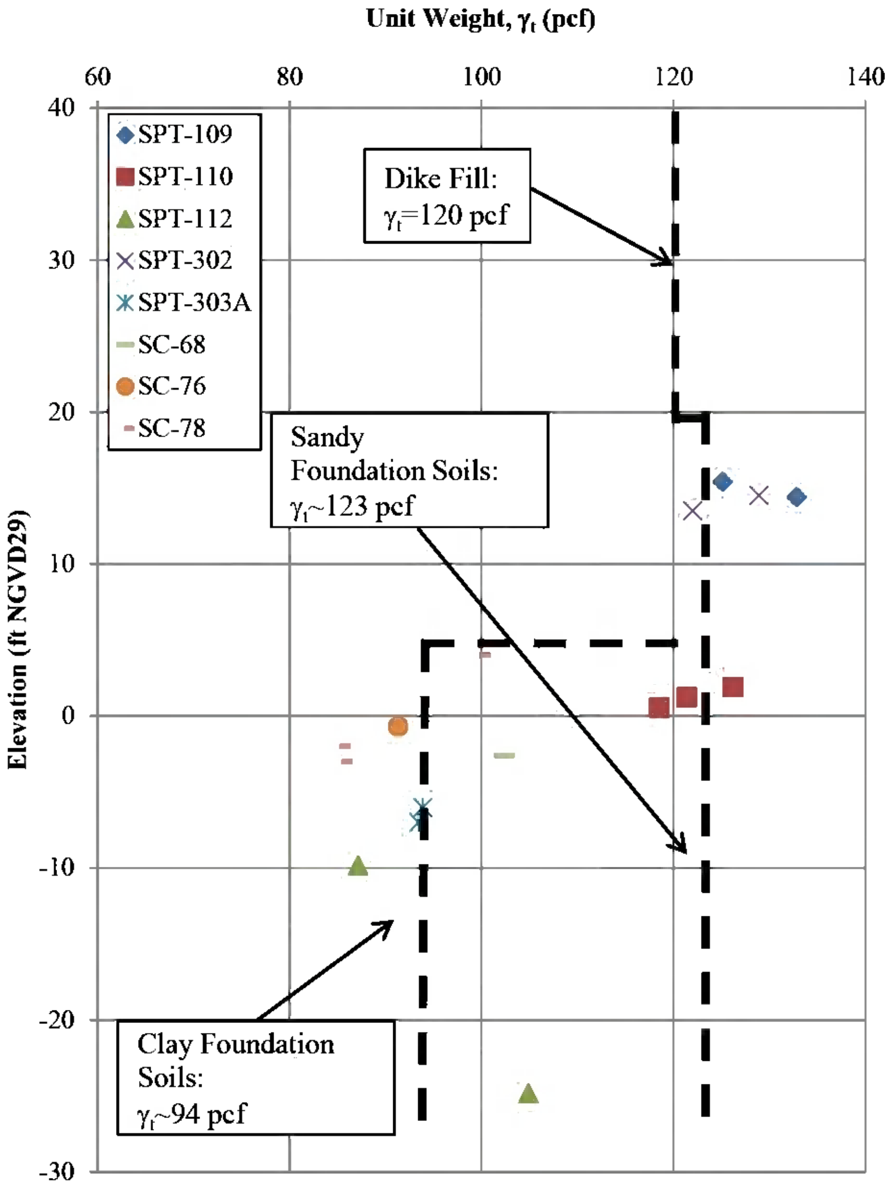

Figure 7. Total Unit Weight from CU Tests

Notes:

1. The total unit weight measurements from triaxial tests are plotted for tests performed on Geosyntec-collected SPT samples and are supplemented with historic S&ME (1978) data (SC-68, SC-76, and SC-78) from undisturbed laboratory specimens.
2. Dike Fill total unit weight was selected as γ_t= 120 pcf using the analysis from the geotechnical investigation performed at Ash Pond A.
3. Fly ash total unit weight was selected with γ_t= 100 pcf using the analysis from the geotechnical investigation performed at Ash Pond A.

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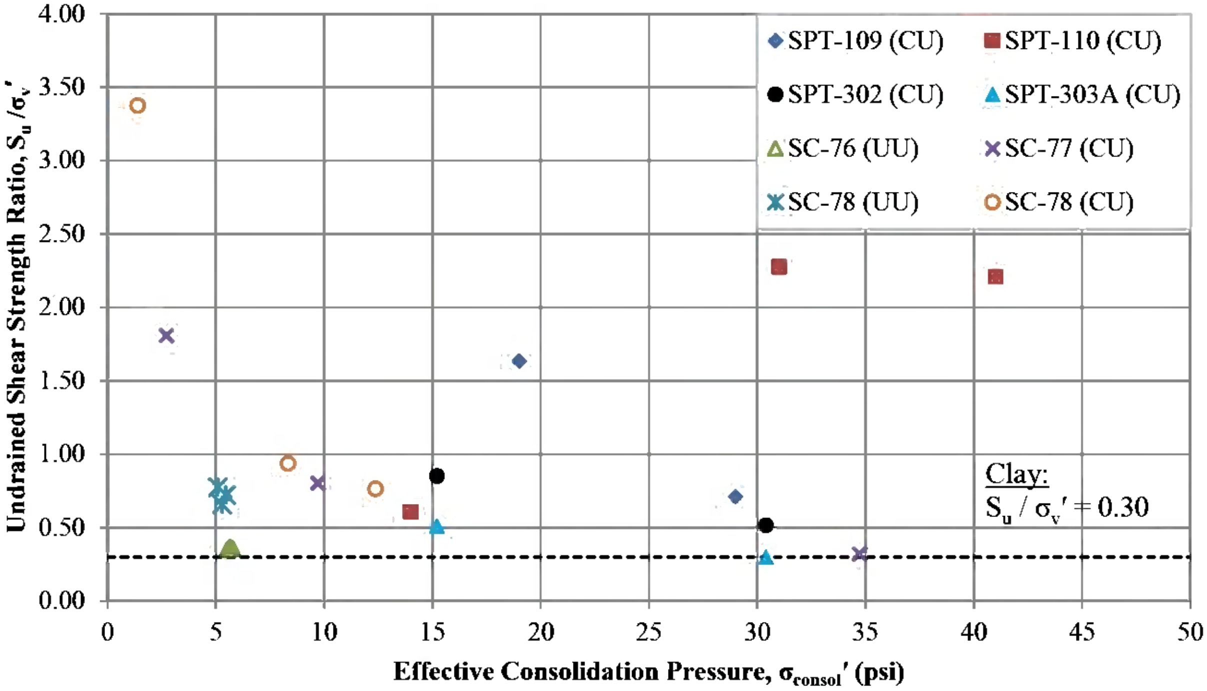

Figure 8. Undrained Shear Strength Ratio from CU tests (Approach 1)

Notes:

1. Foundation Soil samples collected from SPT-109, SPT-110, and SPT-302 generally consisted of sandy clay.
2. Foundation Soil samples collected from SPT-303A and historic specimens SC-76, SC-77, and SC-78 in S&ME (1978) generally consisted of a soft clay
3. Samples from SPT-123 were collected as part of the Ash Pond A investigation and consisted of fly ash.

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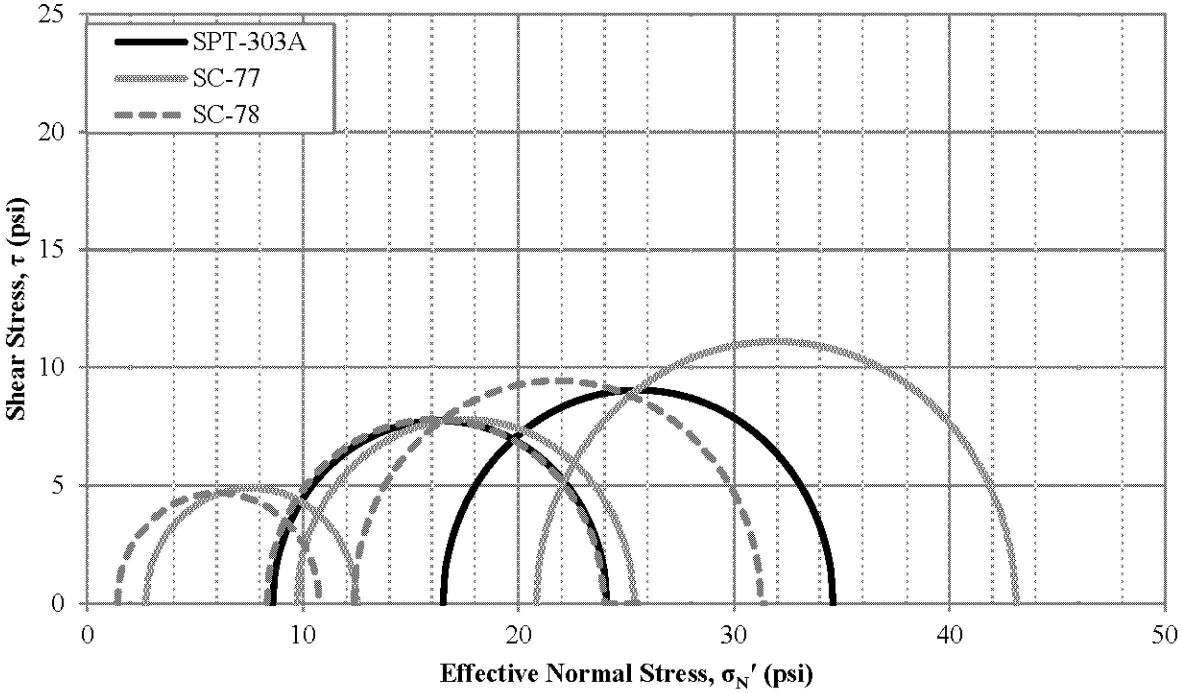

Figure 9. Mohr's Circles for Foundation Clay Soils

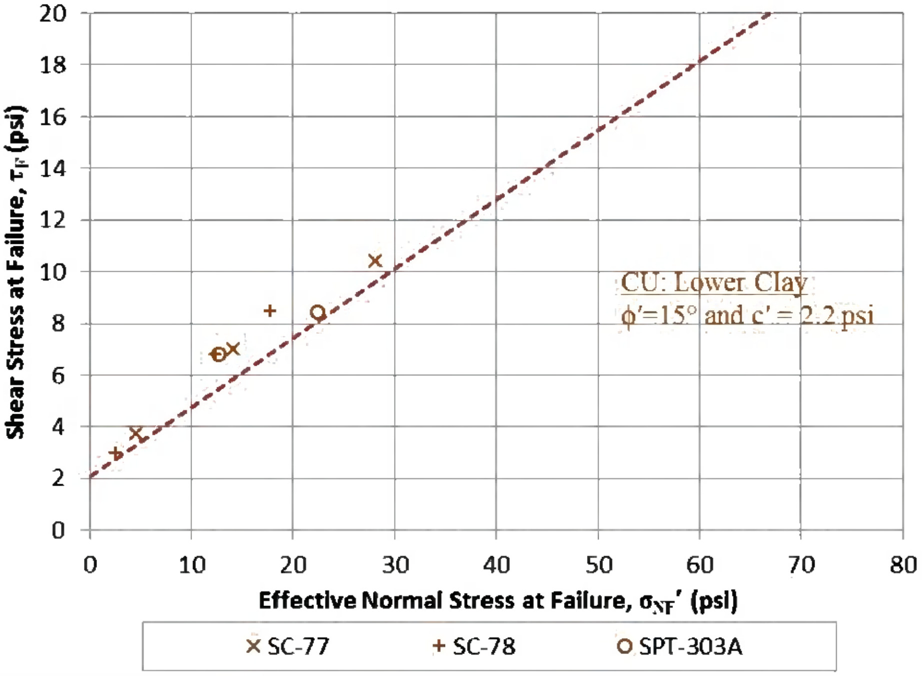

Figure 10. Drained Triaxial Test Failure Envelopes for Foundation Clay Soils

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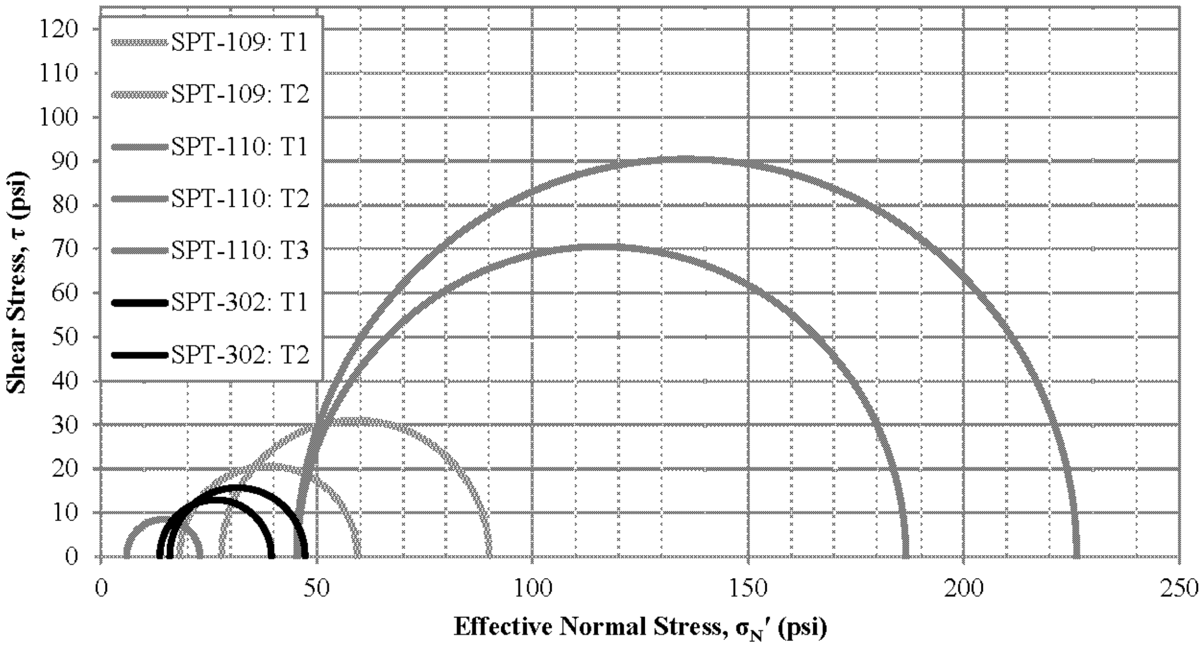

Figure 11. Drained Triaxial Test Failure Envelopes for Foundation Clayey Sand Soils

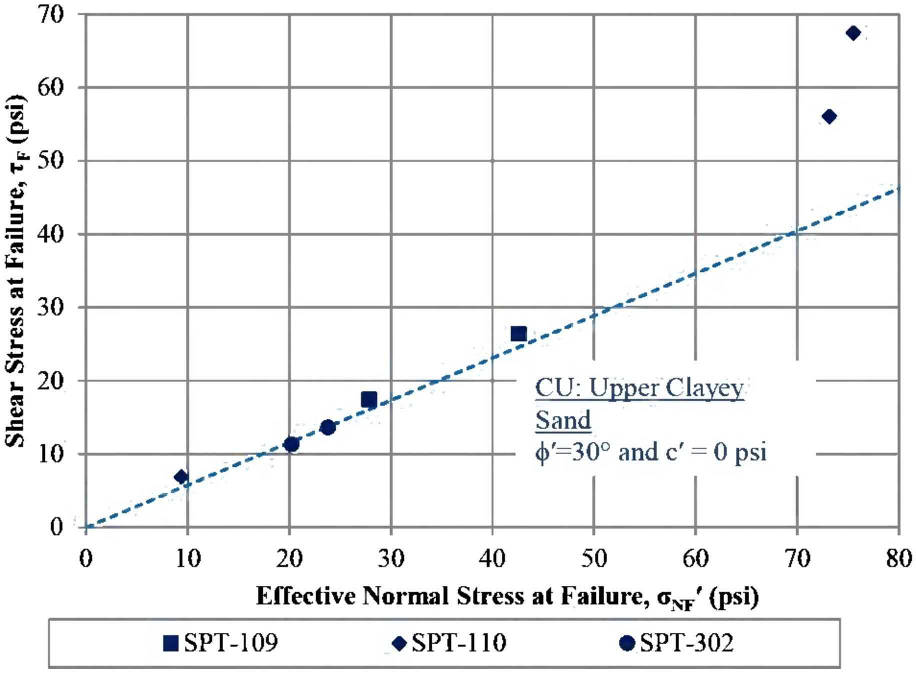

Figure 12. Drained Triaxial Test Failure Envelopes for Foundation Clayey Sand Soils

Written by: **A. Brewster** Date: **10/12/2016** Reviewed by: **A. Brown/B. Gross** Date: **10/12/2016**

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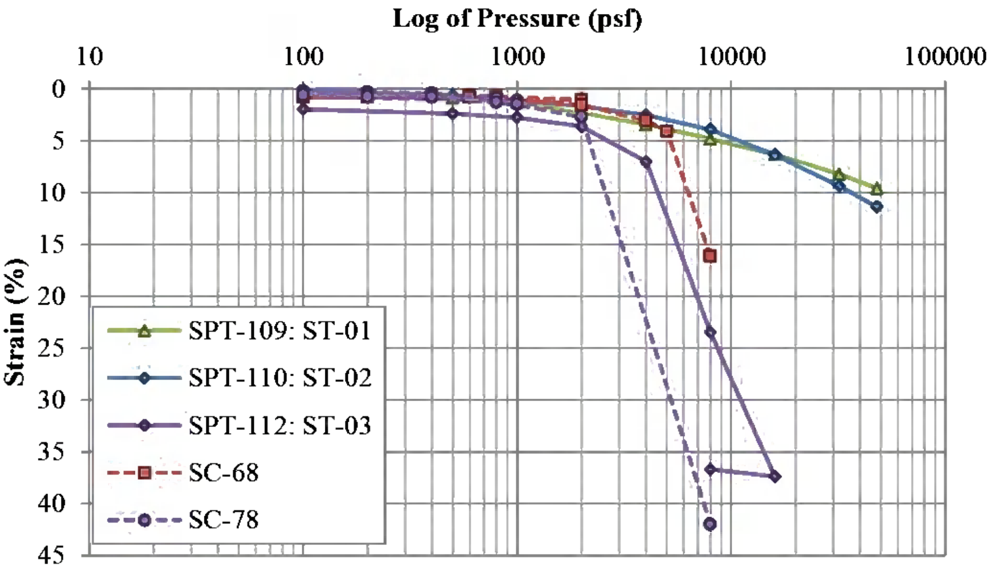

Figure 13. Load-Strain Curves for Foundation Soils

Notes:

1. SPT-109 and SPT-110 tests were performed on samples of silty/clayey sand. The test from soil boring SPT-112 was performed on a sample of high plasticity clay.
2. SC-68 and SC-78 tests were provided by S&ME (1978) on samples of clay soils. SC-68 and SC-78 load-strain curves were digitized by Geosyntec.

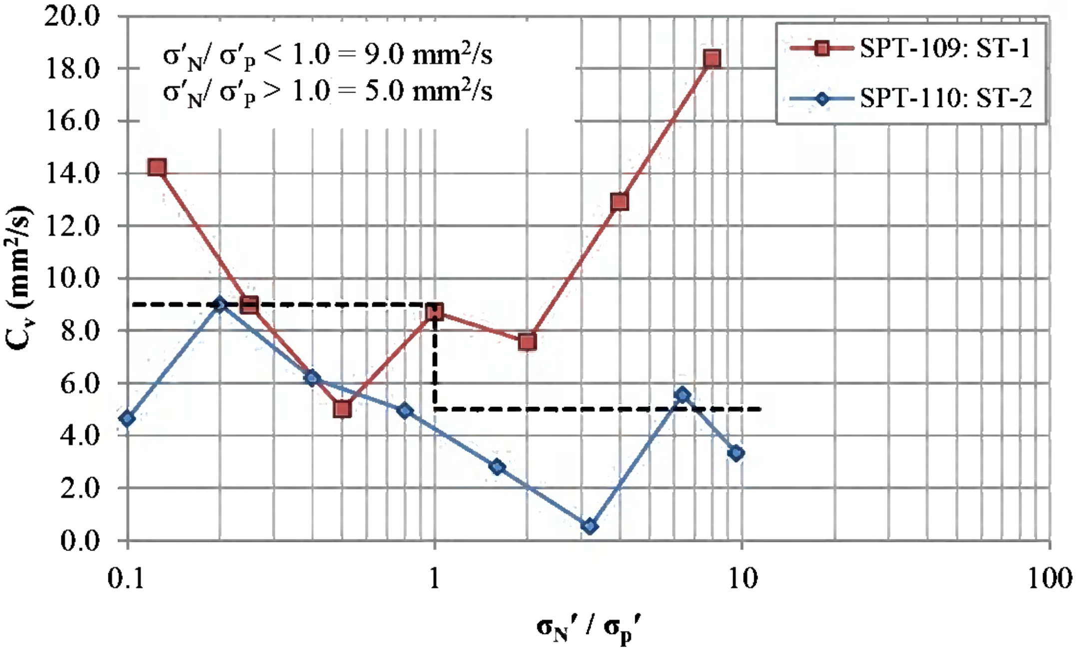

Figure 14. Evaluation of the Coefficient of Consolidation (C_V) for Silty and Clayey Sands

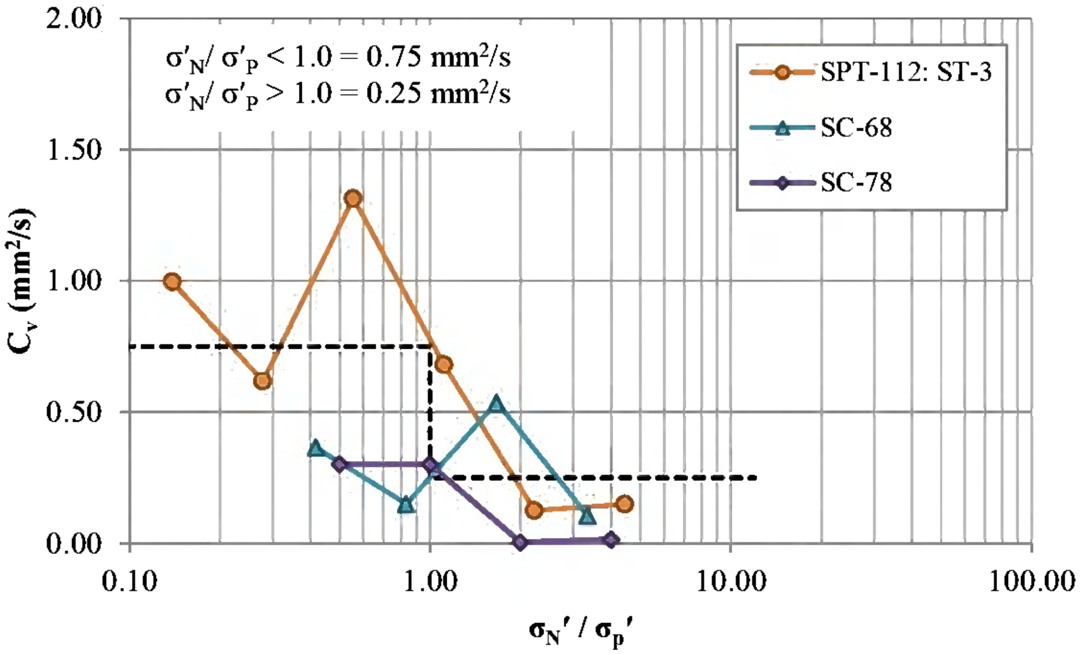

Figure 15. Evaluation of the Coefficient of Consolidation (C_v) for Clays

Note:

1. C_v for SPT-112 was evaluated by Geosyntec from laboratory data. C_v for SC-68 and SC-78 was reported by S&ME (1978) for the stress ratios shown.

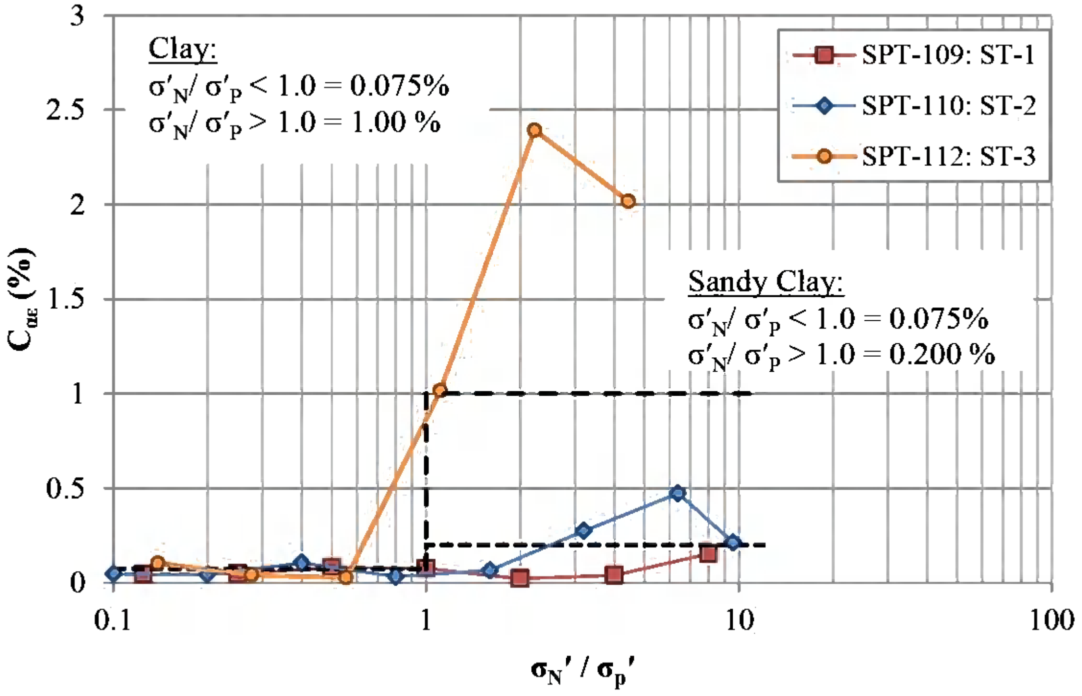

Figure 16. Evaluation of the Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$) for Silty and Clayey Sands and Clays

Note:

1. $C_{\alpha\varepsilon}$ for samples collected from SPT-109, SPT-110, and SPT-112 was evaluated by Geosyntec from laboratory data. $C_{\alpha\varepsilon}$ for SC-68 and SC-78 was not reported by S&ME (1978) for the stress ratios shown.

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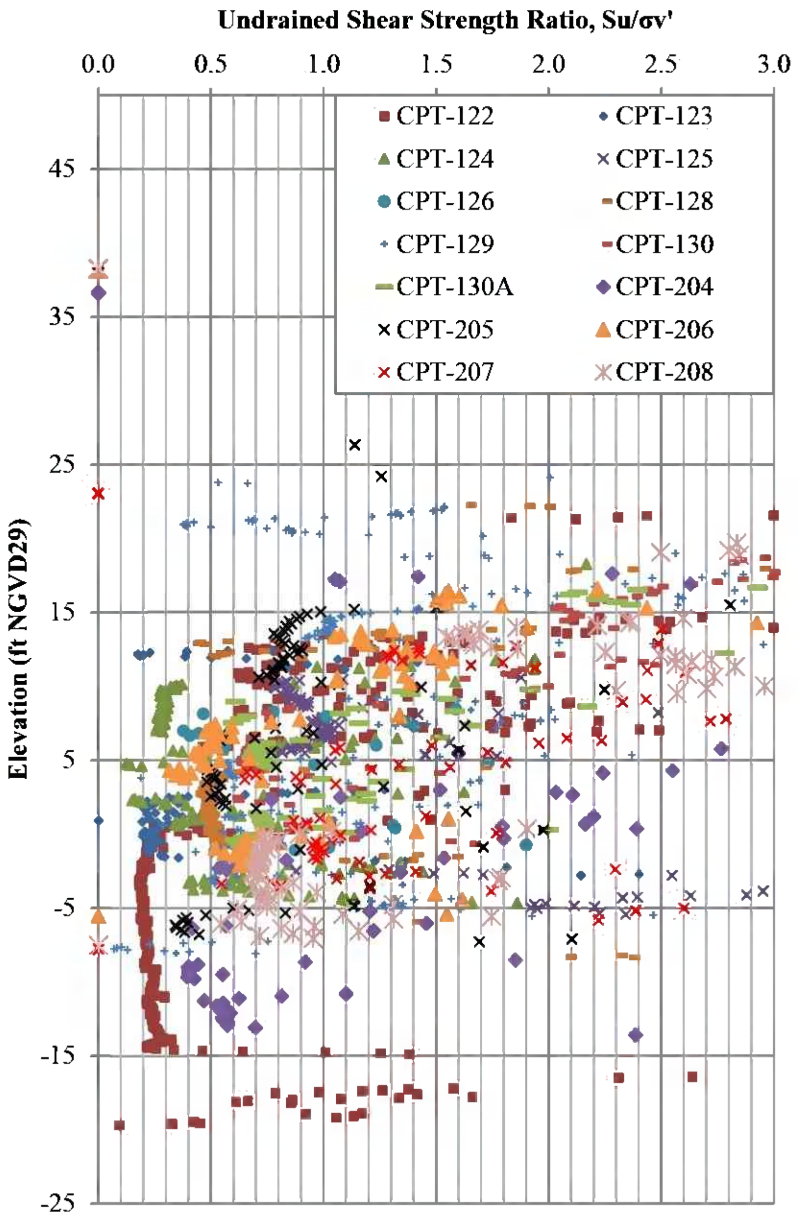

Figure 17. Undrained Shear Strength Ratio for Foundation Soils by CPT Sounding Correlation

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. Selected parameters for foundation soils (i.e., sands) was selected on a cross section by cross section basis.

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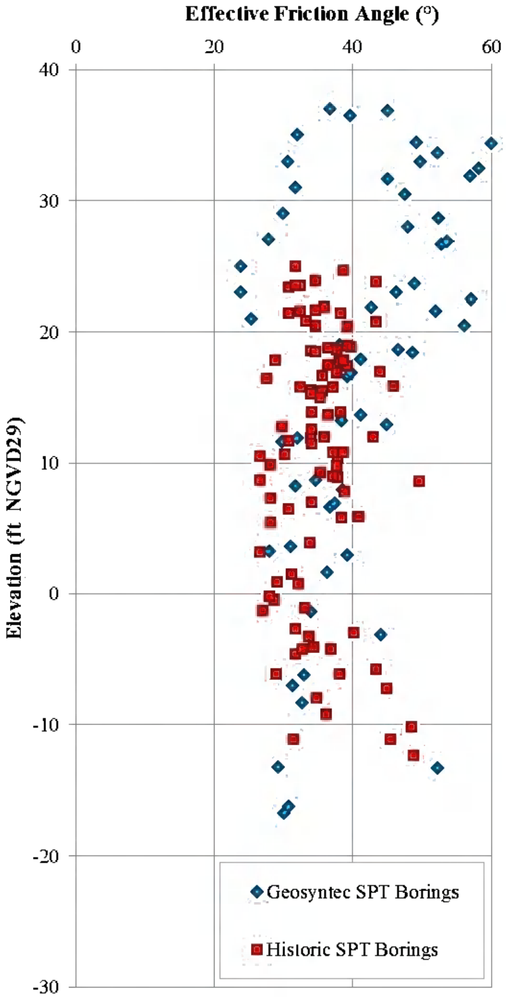

Figure 18. Effective Stress Friction Angle by SPT Correlation

Notes:

1. Effective stress friction angle is computed by Hatanaka and Uchida (1996) for sands.
2. Effective stress friction angle was plotted for materials classified as sands only.

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

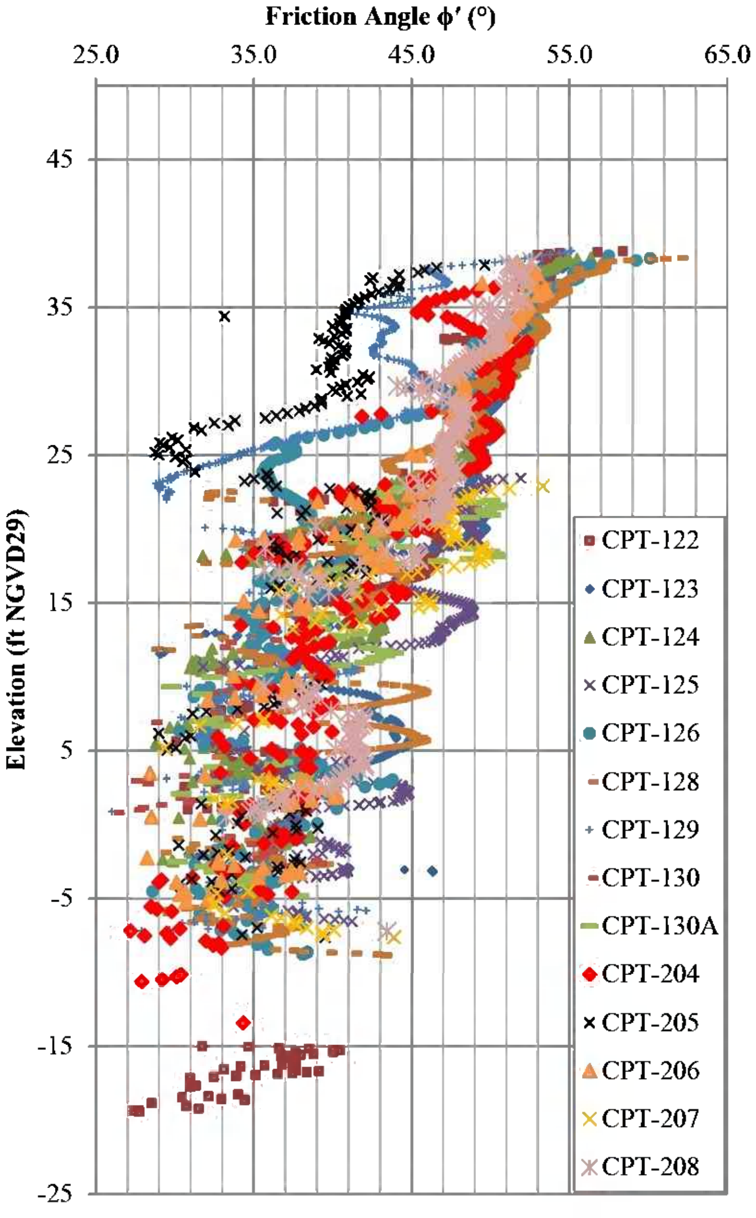

Figure 19. Effective Stress Friction Angle by CPT Correlation

Notes:

1. Effective stress friction angle is computed by Robertson and Campanella (1983) for sands (I_c <2.60).
2. Selected parameters for foundation soils (i.e., sands) was selected on a cross section by cross section basis.

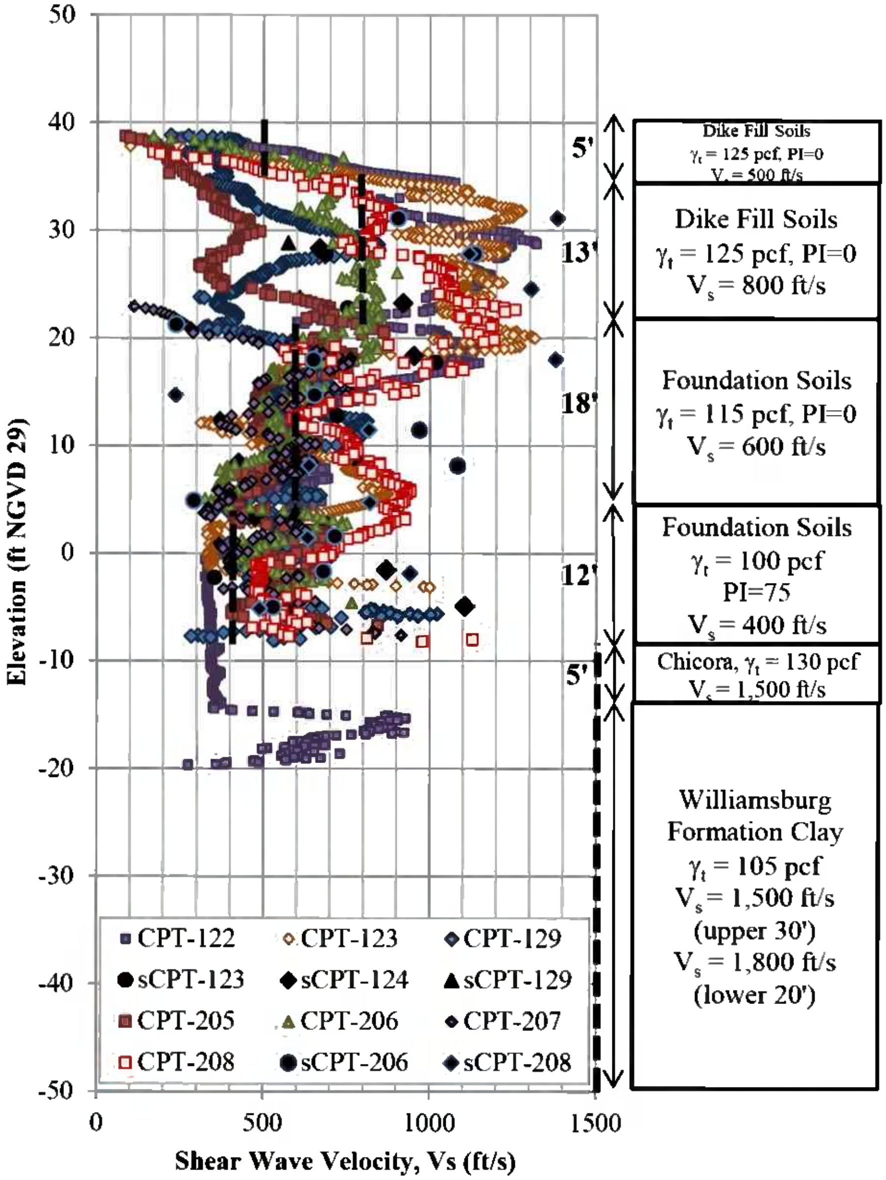

Figure 20. Representative Site Response Profile 1 (Dike Centerline Profile with Clay Foundation Zone)

Notes:

1. sCPT refers to a seismic CPT where V_s measurements are collected in 5 ft depth intervals.
2. The upper 5 ft of Dike Fill Soils were modeled with a V_s = 500 ft/s.
3. A V_s value within the upper range of data for the foundation soils layer overlying the Chicora stratum was selected to conservatively minimize the effects of attenuation during site response analyses.

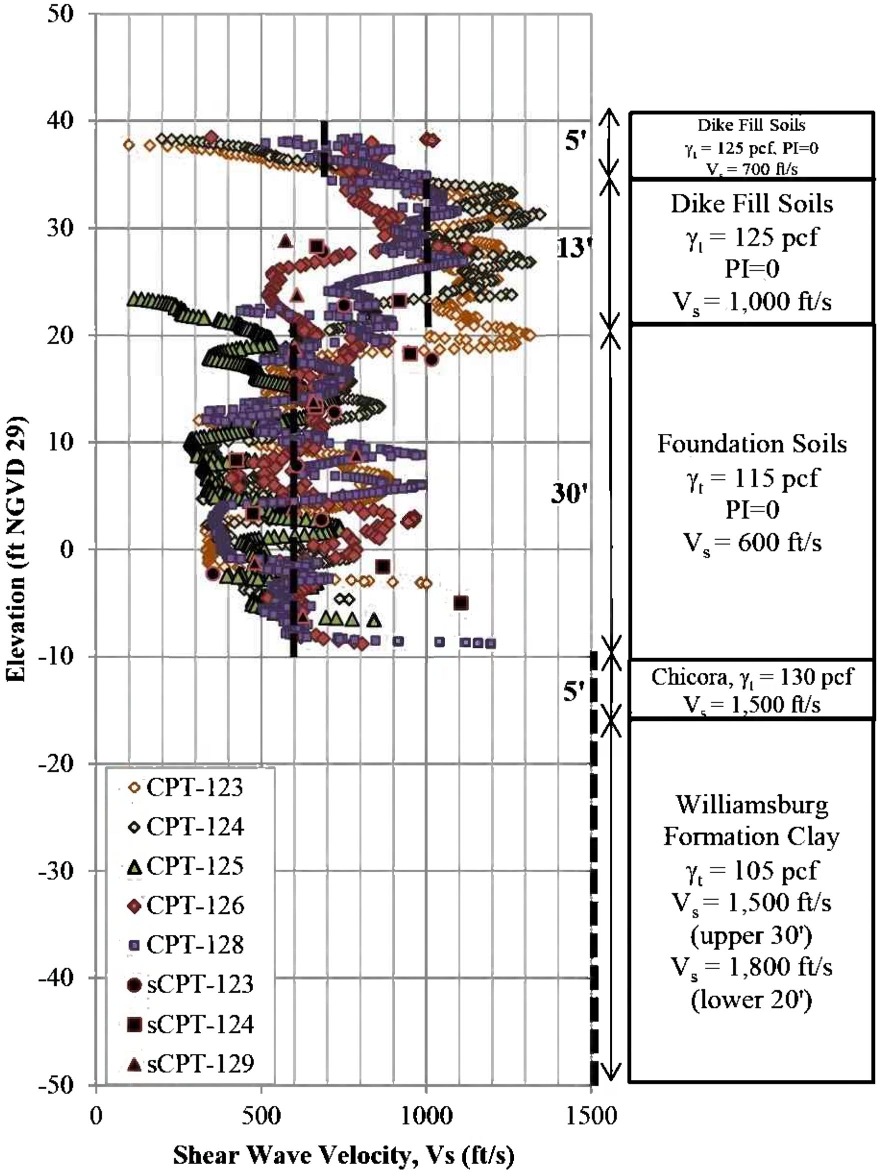

Figure 21. Representative Site Response Profile (Dike Centerline Profile without Clay Foundation Zone)

Notes:

1. sCPT refers to a seismic CPT where V_s measurements are collected in 5 ft depth intervals.
2. The upper 5 ft of Dike Fill Soils were modeled with a V_s = 700 ft/s.
3. A V_s value within the upper range of data for the foundation soils layer overlying the Chicora stratum was selected to conservatively minimize the effects of attenuation during site response analyses.

Appendix 1
Summary of Laboratory Testing Results

Table 1-1. Summary of Geosyntec Index Test Results

Boring ID	Sample ID	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	Hydraulic Conductivity	pH
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%	cm/s	-
SPT-109	SS-2	10.75	26.64	22.4	-	-	-	-	-	-	-	14.5	-	-
SPT-109	SS-3	15.75	21.64	19.6	-	-	-	0.0	78.7	-	-	21.3	-	-
SPT-109	SS-4	20.75	16.64	21.2	-	-	-	-	-	-	-	38.1	-	-
SPT-109	ST-1	22.50	14.89	37.4	48	19	29	0.1	62.5	12.9	24.5	37.4	-	-
SPT-109	SS-7	35.75	1.64	28.7	-	-	-	-	-	-	-	8.3	-	-
SPT-109	SS-8	40.75	-3.36	71.7	110	33	77	0.0	23.7	22.9	53.4	76.3	-	-
SPT-109	SS-9	45.75	-8.36	37.2	-	-	-	-	-	-	-	29.1	-	-
SPT-110	SS-2	10.75	27.97	16.3	-	-	-	-	-	-	-	15.9	-	-
SPT-110	SS-3	15.75	22.97	18.0	-	-	-	-	-	-	-	15.9	-	-
SPT-110	SS-5	25.75	12.97	24.9	-	-	-	-	-	-	-	12.8	-	-
SPT-110	ST-2	37.50	1.22	-	NP	NP	NP	2.6	81.9	4.8	10.7	15.5	-	-
SPT-110	SS-8	40.75	-2.03	25.2	-	-	-	17.8	65.7	-	-	16.5	-	-
SPT-110	SS-9	45.75	-7.03	21.7	-	-	-	-	-	-	-	-	-	-
SPT-110	SS-11	55.75	-17.03	13.1	-	-	-	35.9	45.3	-	-	18.8	-	-
SPT-110	SS-13	65.80	-27.08	50.0	-	-	-	0.0	13.0	38.4	48.6	87.0	-	-
SPT-111	SS-1	5.75	33.66	13.7	-	-	-	2.9	79.8	8.3	9.0	17.3	-	-
SPT-111	SS-5	25.75	13.66	19.1	-	-	-	0.2	85.6	-	-	14.2	-	-
SPT-111	SS-6	30.75	8.66	21.7	-	-	-	-	-	-	-	9.5	-	-
SPT-111	SS-7	35.75	3.66	34.6	-	-	-	2.3	74.0	-	-	23.7	-	-
SPT-111	SS-8	40.75	-1.34	20.8	-	-	-	-	-	-	-	8.1	-	-
SPT-112	SS-2	3.25	34.41	17.6	-	-	-	-	-	-	-	10.9	-	-
SPT-112	ST-1	9.00	28.66	21.1	-	-	-	0.0	78.7	-	-	21.3	-	-
SPT-112	SS-4	10.75	26.91	13.6	-	-	-	-	-	-	-	13.3	-	-
SPT-112	SS-7	25.75	11.91	29.2	-	-	-	0.1	91.1	-	-	8.8	-	-

Table 1-1. Summary of Geosyntec Index Test Results (Continued)

Boring ID	Sample ID	Depth	Elev.(1)	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	Hydraulic Conductivity	pH
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%	cm/s	-
SPT-112	SS-8	30.75	6.91	26.8	-	-	-	-	-	-	-	5.4	-	-
SPT-112	SS-9	40.75	-3.09	23.8	-	-	-	-	-	-	-	9.7	-	-
SPT-112	ST-3	47.50	-9.84	-	151	55	96	0.3	2.3	21.5	75.9	97.4	-	-
SPT-112	SS-12	55.75	-18.09	15.3	-	-	-	-	-	-	-	22.7	-	-
SPT-112	SS-13	60.75	-23.10	48.9	-	-	-	0.0	9.3	42.0	48.7	90.7	-	-
SPT-112	ST-04	63.00	-24.84	42.5	-	-	-	-	-	-	-	-	1.8×10^{-8}	-
SPT-113	SS-2	10.75	31.52	31.8	-	-	-	7.7	61.8	-	-	30.5	-	-
SPT-113	SS-3	15.75	26.52	43.8	-	-	-	0.0	81.1	-	-	18.9	-	-
SPT-113[2]	ST-2	27.50	14.77	40.5	NP	NP	NP	10.4	23.6	43.8	22.2	66.0	-	5.7
SPT-302	S-3	38.01	33.01	27.9	-	-	-	-	-	-	-	6.9	-	-
SPT-302	S-7	38.01	25.01	25.1	-	-	-	-	-	-	-	34.4	-	-
SPT-302	S-8	38.01	23.01	22.1	-	-	-	-	-	-	-	14.9	-	-
SPT-302	S-9	38.01	21.01	28.4	-	-	-	0.5	68.2	-	-	31.3	-	-
SPT-302	S-11	38.01	14.01	19.80	39	18	21	1.3	76	-	-	22.7	-	-
SPT-302	S-13	38.01	4.01	22.9	-	-	-	-	93.8	-	-	6.2	-	-
SPT-303	S-5	37.48	28.48	17.10	-	-	-	-	-	-	-	11.5	-	-
SPT-303	S-12	37.48	8.23	25.00	-	-	-	-	-	-	-	38.6	-	-
SPT-303	S-19	37.48	-13.02	32.8	-	-	-	-	-	-	-	11.7	-	-
SPT-303A	S-15	37.48	-6.52	84.2	108	38	70	-	2.6	30.6	66.8	97.4	-	-

Notes:

1. Sample elevation is provided in ft NGVD29.
2. Sample was tested by ASTM D4373 to measure the carbonate content; the resulting carbonate content was 0%.

Table 1-2. Summary of S&ME (1978) Index Test Results

Boring ID	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content
Units	-	ft bgs	ft	%	%	%	%	%	%	%	%	%
SC-66	SS	1.75	20.85	17.7	-	-	-	-	-	-	-	14.3
SC-67	SS	1.75	22.65	18.2	-	-	-	-	-	-	-	21.8
SC-68	SS	1.75	21.15	20.1	-	-	-	-	-	-	-	-
SC-68	SS	5.25	17.65	13.4	-	-	-	-	-	-	-	12.1
SC-68	ST	25.5	-2.6	22.7	-	-	-	-	-	-	-	-
SC-69	SS	1.75	225	21.6	-	-	-	-	-	-	-	8.8
SC-69	SS	5.25	20.75	22.7	-	-	-	-	-	-	-	6.4
SC-70	SS	5.25	19.55	21.0	-	-	-	-	-	-	-	4.3
SC-71	SS	1.75	23.95	9.7	-	-	-	-	-	-	-	1.4
SC-71	SS	5.25	20.45	23.0	-	-	-	-	-	-	-	4.9
SC-72	SS	1.75	23.15	13.4	-	-	-	-	-	-	-	5.7
SC-72	SS	5.25	19.65	25.1	-	-	-	-	-	-	-	2.5
SC-73	SS	5.25	19.25	25.9	-	-	-	-	-	-	-	3.4
SC-76	ST	17.5	-0.70	95.0	-	-	-	-	-	-	-	-
SC-76	ST	17.5	-0.70	92.9	-	-	-	-	-	-	-	-
SC-76	ST	17.5	-0.70	102.4	-	-	-	-	-	-	-	-
SC-77	ST	11	3.00	89.2	-	-	-	-	-	-	-	-
SC-78	ST	12	4.00	63.4	-	-	-	-	-	-	-	-
SC-78	ST	12	4.00	57.2	-	-	-	-	-	-	-	-
SC-78	ST	12	4.00	98.0	-	-	-	-	-	-	-	-
SC-78	ST	18	-2.00	87.9	-	-	-	-	-	-	-	-
SC-78	ST	19	-3.00	96.6	-	-	-	-	-	-	-	-

Notes:

1. Sample type “ST” refers to a Shelby tube sample, and sample type “SS” refers to split spoon sample collected during an SPT.
2. Sample elevation is provided in ft NGVD29.

Table 1-3. Summary of Geosyntec Triaxial Test Results

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'	ϕ'	c'
Units	ft bgs	ft MSL	%	pcf	pcf	psi	psi	psi	psi	-	°	psi
SPT-109	22.5	14.89	14.7	115.8	133.6	19.0	90	27.9	31.05	1.63	31.78	0.00
SPT-109	22.5	14.89	19.4	104.8	125.1	29.0	59.4	18.2	20.6	0.71	32.07	0.00
SPT-110	37.5	1.22	36.1	87.1	118.5	14.0	22.9	5.9	8.5	0.61	36.18	0.00
SPT-110	37.5	1.22	27.2	95.4	121.3	31.0	186.6	45.5	70.55	2.28	37.44	0.00
SPT-110	37.5	1.22	16.2	108.6	126.2	40.7	226.2	45.3	90.45	2.21	41.78	0.00
SPT-302	24.0	14.01	18.8	108.5	128.9	15.2	39.5	13.6	12.95	0.85	29.19	0.00
SPT-302	24.0	14.01	20.8	101.0	122.0	30.4	47.3	15.9	15.7	0.52	29.79	0.00
SPT-303A	44.0	-6.52	82.4	51.2	93.4	21.0	24.1	8.6	7.75	0.51	28.29	0.00
SPT-303A	44.0	-6.52	85.9	50.5	93.9	42.0	34.6	16.5	9.05	0.30	20.74	0.00

Notes:

1. Undrained shear strength ratio, friction angle, and cohesion intercepts were computed by Geosyntec.
2. Moisture content and unit weight data for SPT-109 through SPT-112 are listed from bulk unit weight test, and are similar to the densities at which the samples were tested for triaxial or consolidation testing.
3. Sample elevation is provided in ft NGVD29.

Table 1-4. Summary of S&ME (1978) Triaxial Testing Results

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'	ϕ'	c'
Units	ft bgs	ft MSL	%	pcf	pcf	psi	psi	psi	psi	-	°	psi
SC-76	17.5	-0.70	95.0	46.5	90.68	5.65	-	-	2.08	0.37	0.0	2.08
SC-76	17.5	-0.70	92.9	47.5	91.63	5.78	-	-	2.08	0.36		
SC-76	17.5	-0.70	102.4	45.3	91.69	2.71	12.50	2.71	4.90	1.81		
SC-77	11.0	3.00	89.2	52.4	99.14	9.72	25.35	9.72	7.81	0.80	15.5	2.78
SC-77	11.0	3.00				34.72	43.06	20.83	11.11	0.32		
SC-77	11.0	3.00				5.10	-	-	3.96	0.78		
SC-78	12.0	4.00	63.4	61.7	100.82	5.28	-	-	3.47	0.66	0.0	3.96
SC-78	12.0	4.00	57.2	68.2	107.21	5.47	-	-	3.96	0.72		
SC-78	12.0	4.00	98.0	45.1	89.30	1.39	10.76	1.39	4.69	3.38		
SC-78	18.0	-2.00	87.9	45.4	85.3	8.33	23.96	8.33	7.81	0.94	20.0	2.08
SC-78	18.0	-2.00				12.36	31.25	12.36	9.44	0.76		
SC-78	18.0	-2.00				15.20	39.5	13.6	12.95	0.85		

Notes:

1. Sample elevation is provided in ft NGVD29.

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: SOUTH ASH POND**

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SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS: SOUTH ASH POND

PURPOSE

The purpose of this calculation package is to present the results of the seismic hazard evaluation and site response analyses performed for the South Ash Pond 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 South Ash Pond 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 target 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: South Ash Pond*" (Data Package). Information that is specific to the site response analysis is presented herein. To develop representative soil profiles, the South Ash Pond perimeter dike was divided into two sections comprised of dike fills and foundation soils. The soil stratigraphy is similar around the South Ash Pond. However, the profile on the west end of the South Ash Pond (Profile 1 shown in Figure 5) contains a layer of highly plastic foundation soils (Plasticity Index = 75) at depths between 36 and 48 ft below ground surface. Two representative profiles to 100 ft below ground surface (bgs) were developed for the perimeter dike: (i) one for the west end of the South Ash Pond (Profile 1); and (ii) one for the east end of the South Ash Pond (Profile 2). For both profiles, the water table was assumed to be at a depth of 10 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.

Table 2. Calculated Maximum Shear Stress Envelopes

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	41	2.5	36
7.5	96	7.5	80
12.5	124	12.5	110
16.5	146	16.5	135
19.5	171	20.5	160
23.5	192	25.5	182
28.5	204	30.5	195
33.5	213	35.5	205
38.0	272	40.5	214
42.0	306	45.5	223
46.0	331	50.5	283
50.5	364	58.0	383
58.0	457	68.0	488
68.0	572	78.0	555
78.0	682	88.0	679
88.0	789	98.0	758
98.0	929	108.0	892
108.0	1064		

Figures

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**

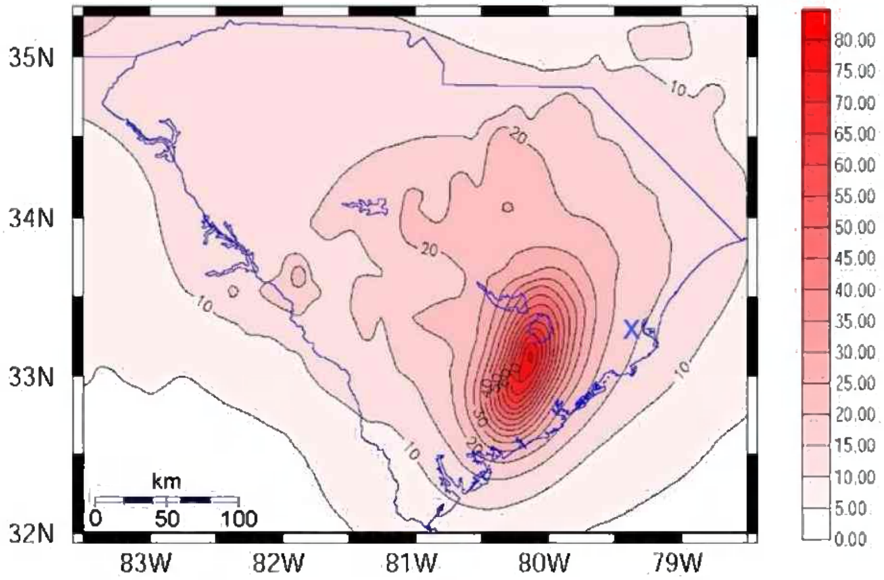

Figure 1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Chapman and Talwani, 2006)

Note:

1. PGA for WGS was selected as 0.16g.

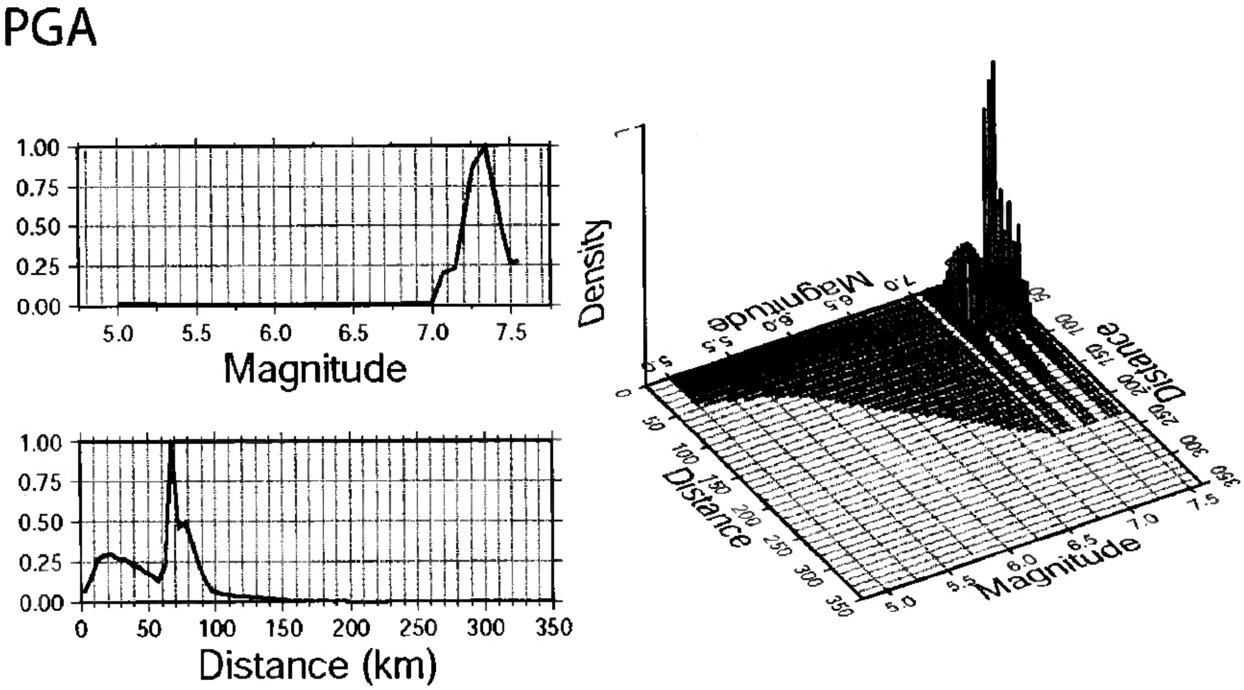

Figure 2. Deaggregation of 2 Percent Probability of Exceedance in 50 Years for PGA near Georgetown, South Carolina

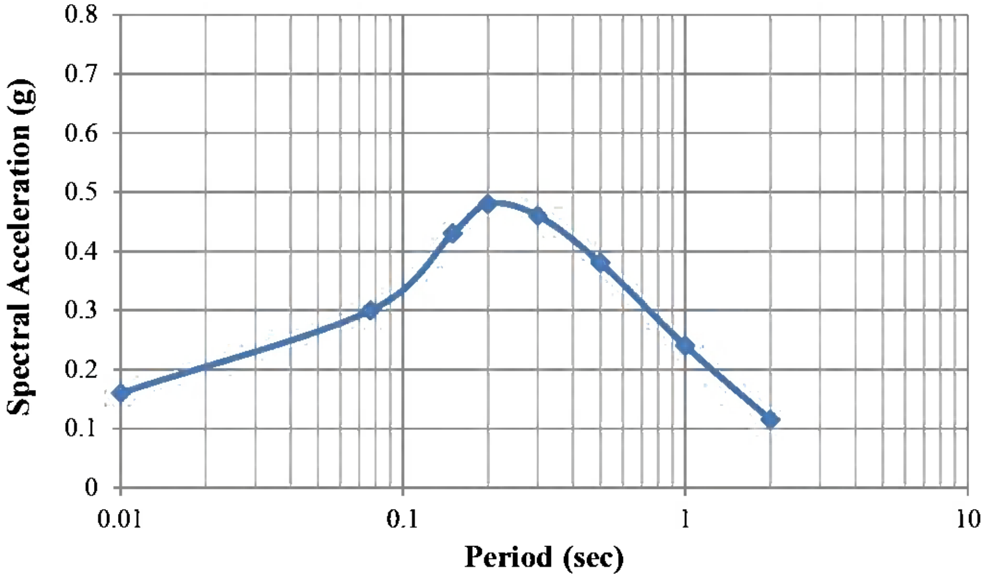

Figure 3. "Geologically Realistic" Target Response Spectrum for WGS

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.

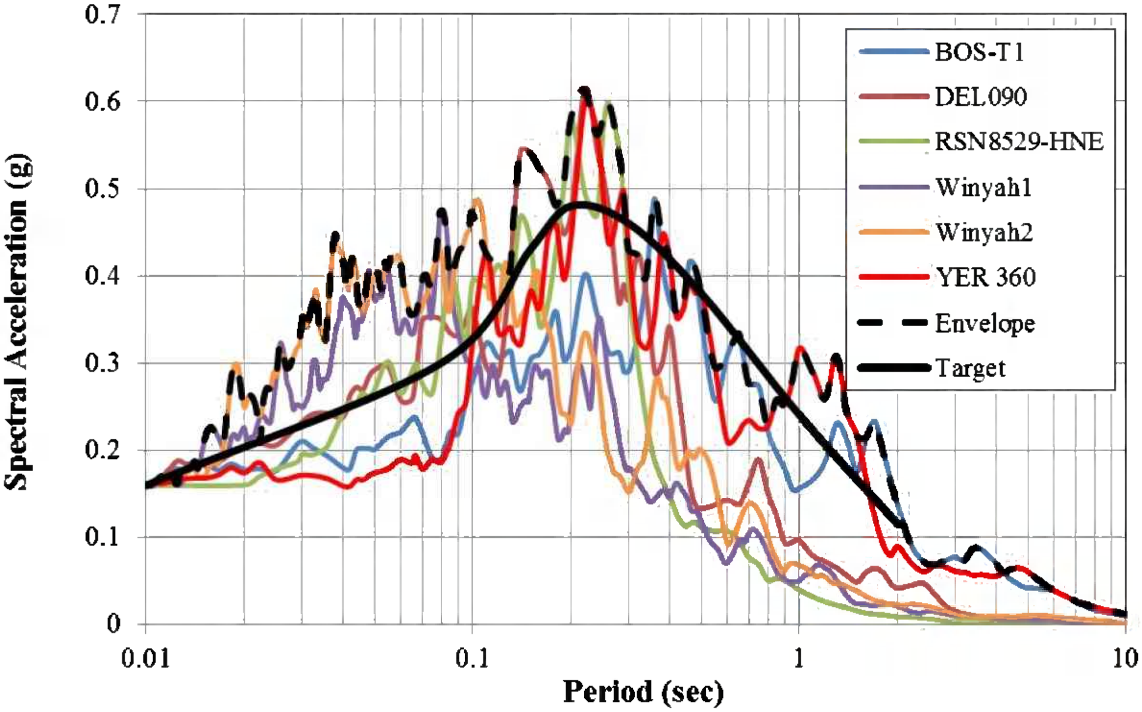

Figure 4. Response Spectra of Scaled Time Histories Selected for Seismic Evaluations

Note:

1. Time histories were scaled to match the target PGA = 0.16g (represented by a period of 0.01 seconds).

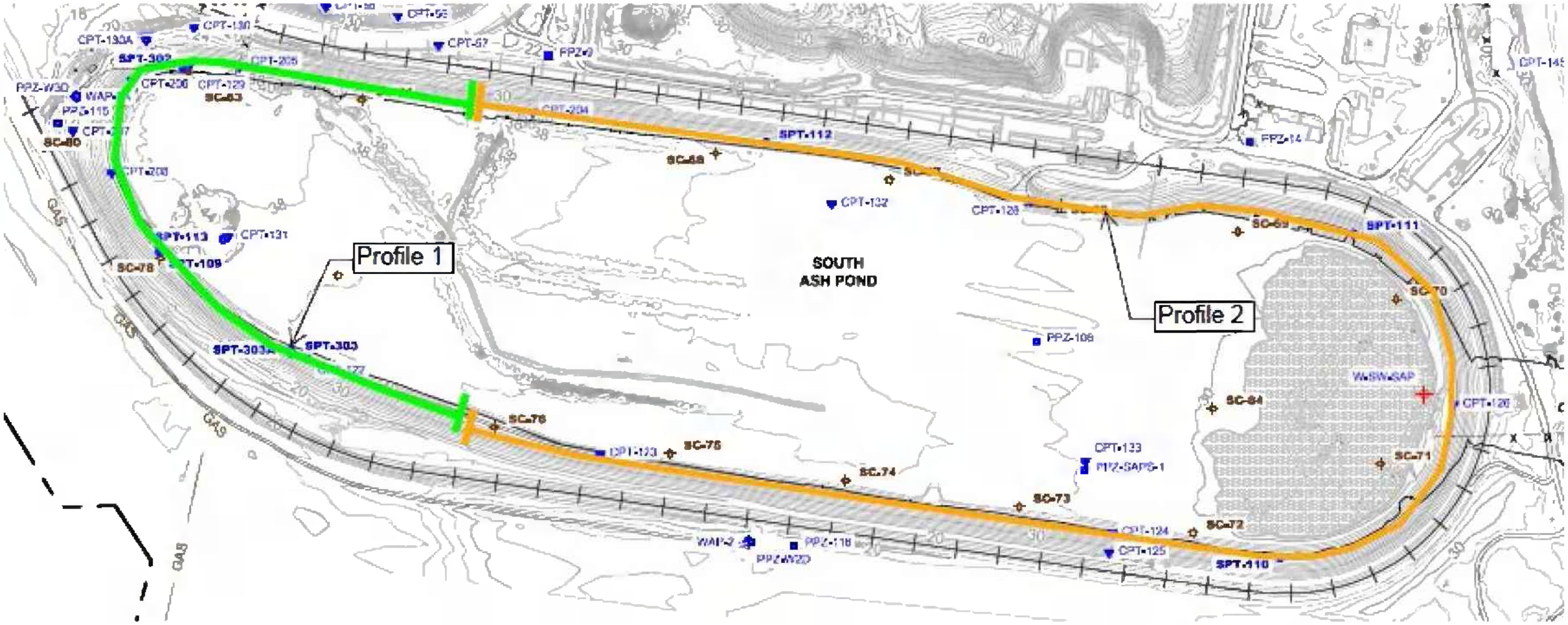

Figure 5. Locations of Representative Soil Profiles

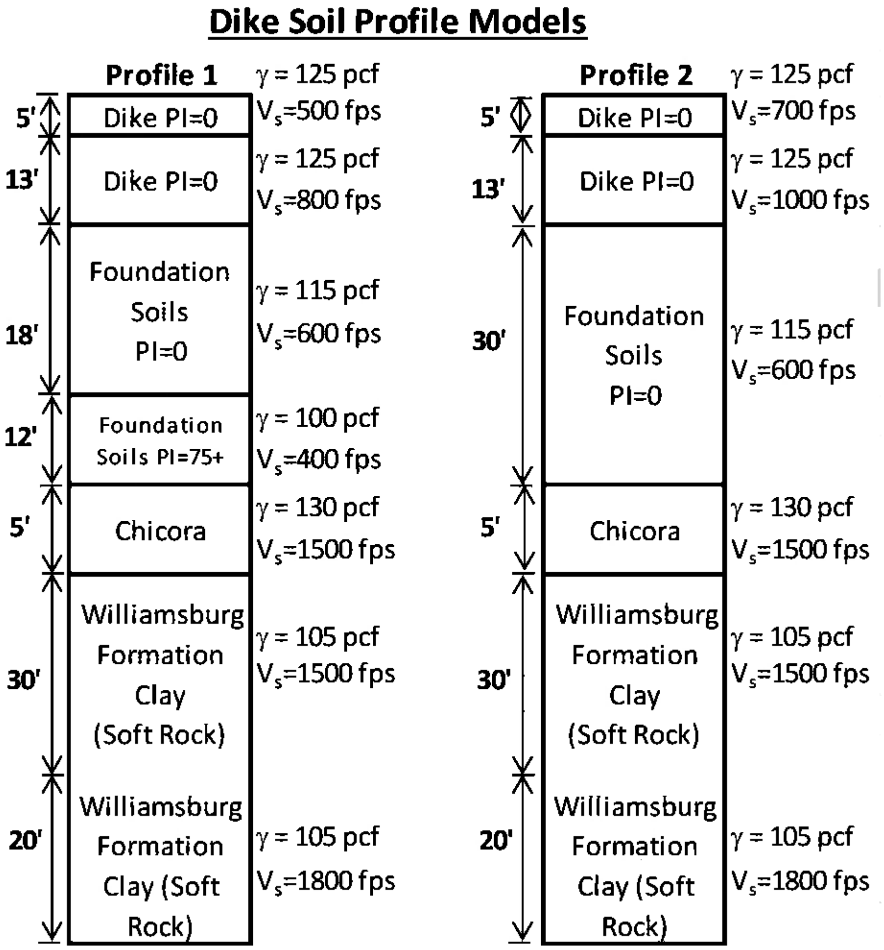

Figure 6. DEEPSOIL® Soil Profile Models for the Two Representative Profiles

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

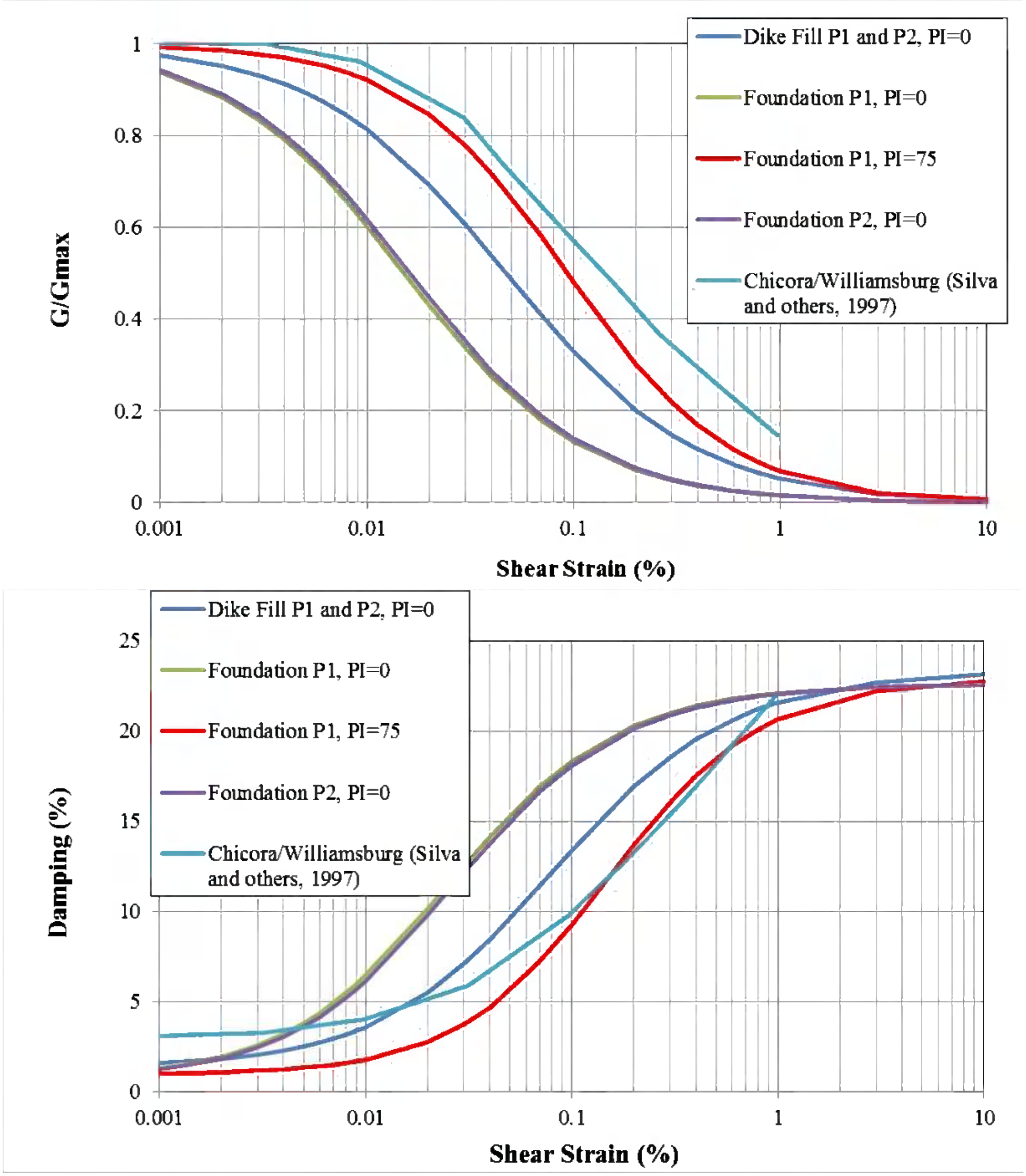


Figure 7. Shear Modulus Reduction (G/G_{max}) and Damping Curves for Soils Used in Site Response Analyses

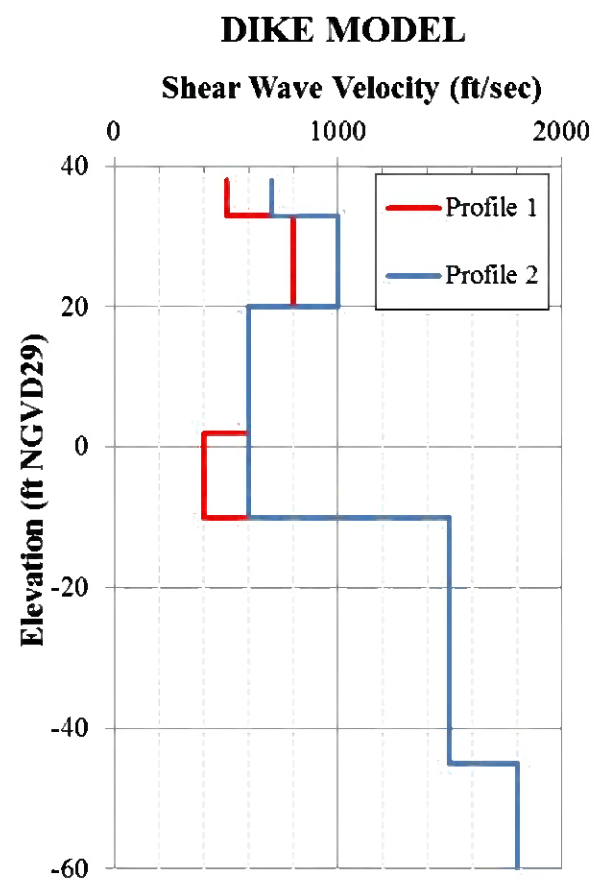


Figure 8. Selected Shear Wave Velocity (V_s) Profiles for Site Response Analyses

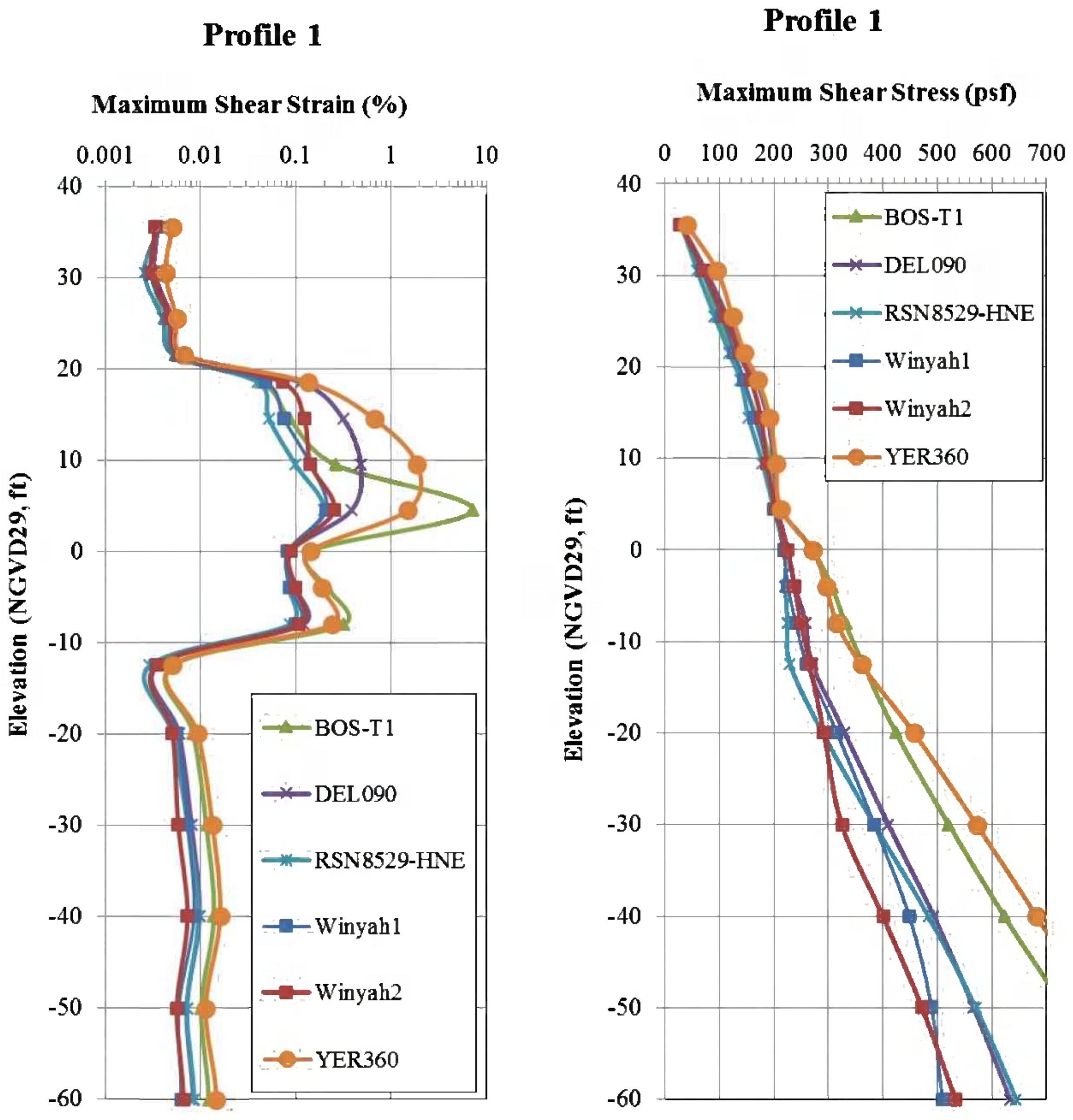


Figure 9a. Site Response Analysis Results for Profile 1

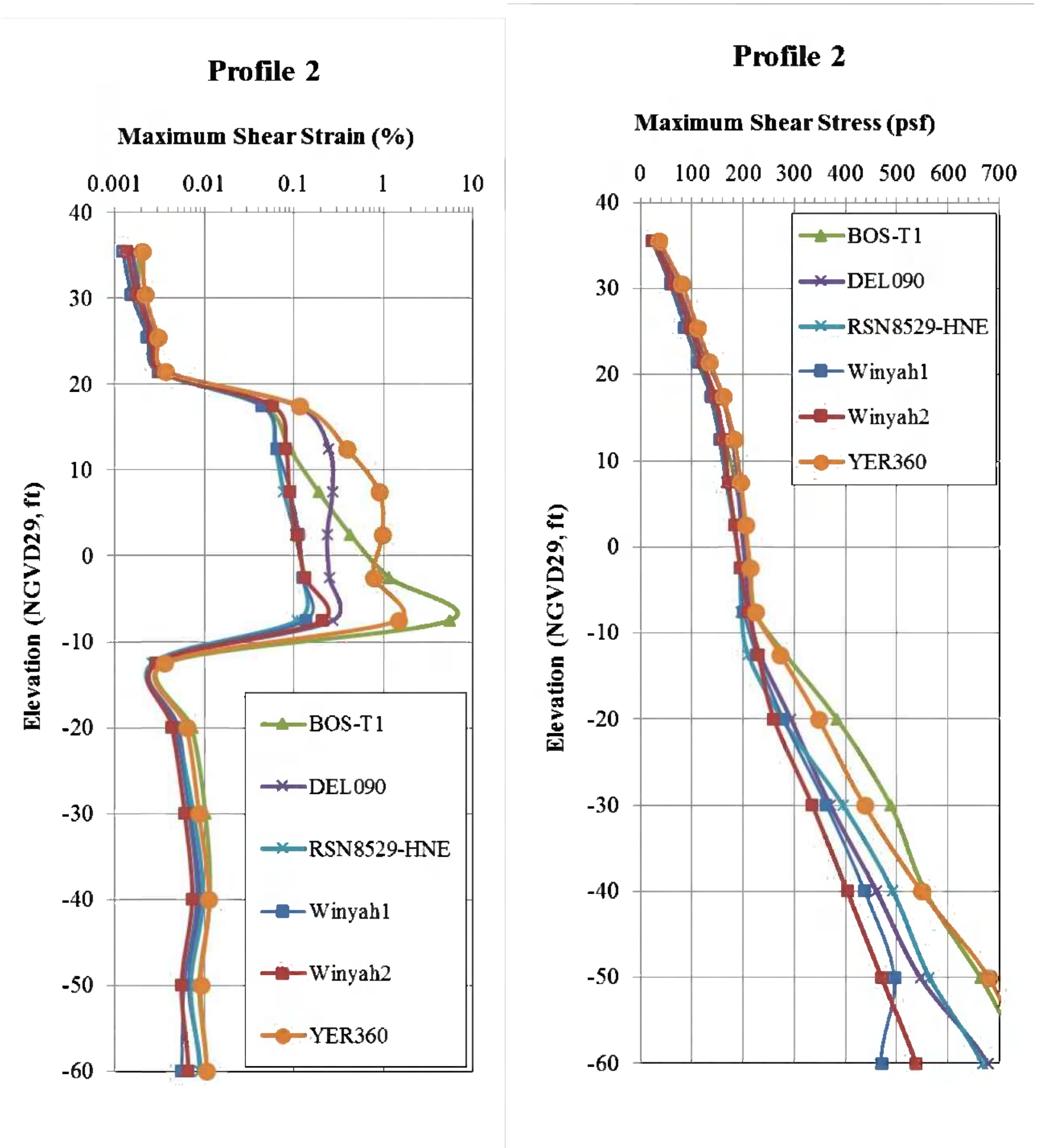


Figure 9b. Site Response Analysis Results for Profile 2

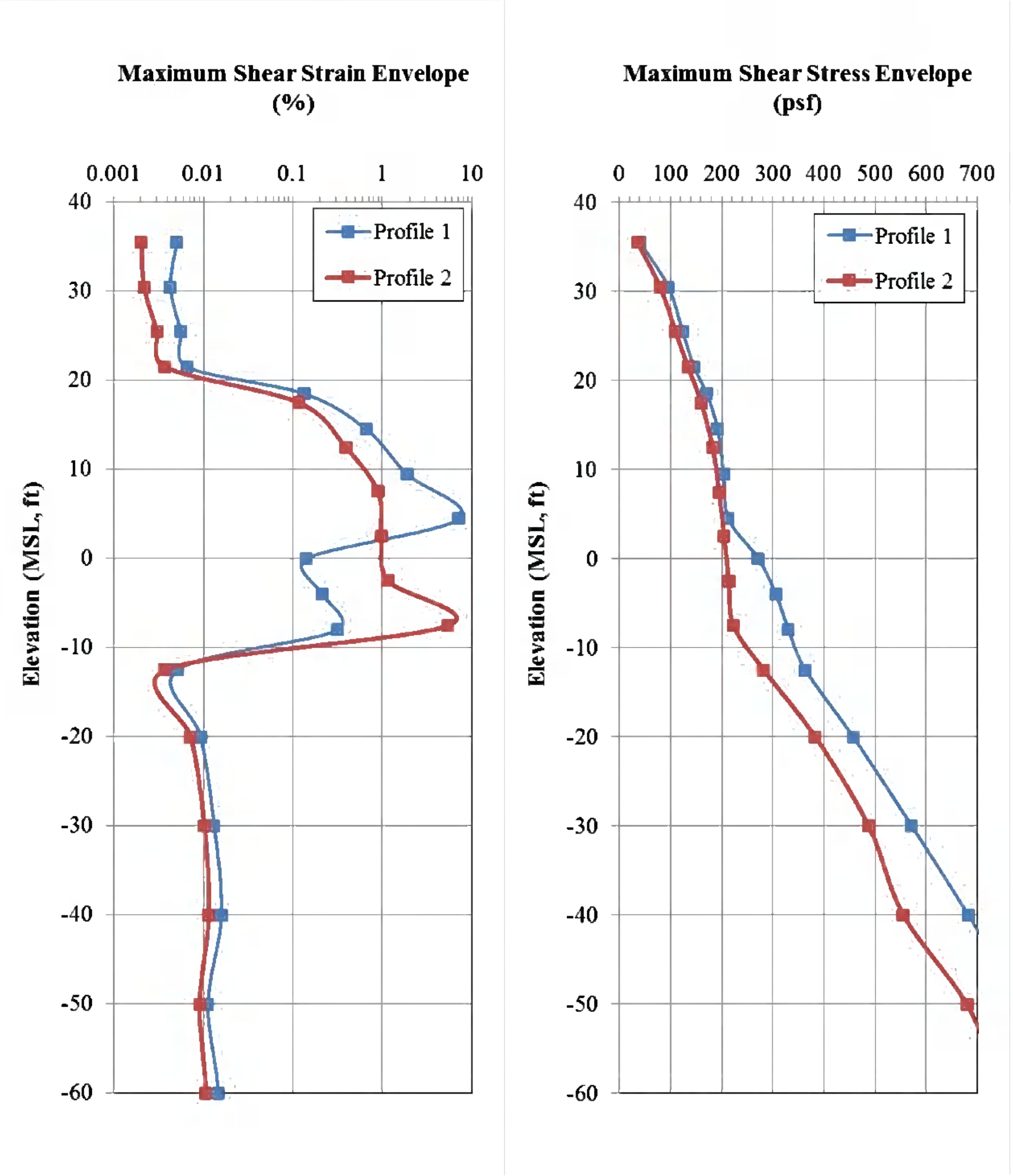
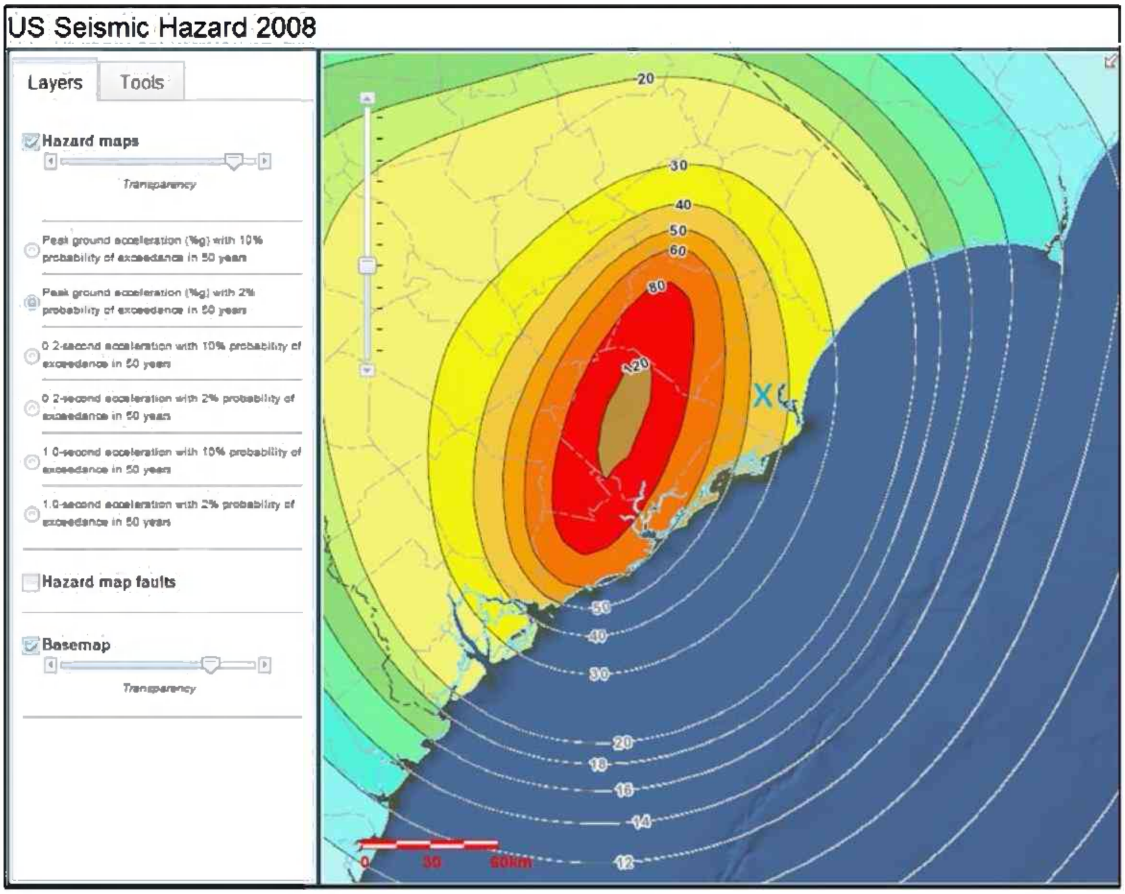


Figure 10. Maximum Shear Strain and Shear Stress Envelopes for Each Profile

Appendix 1

Peak Ground Accelerations from Different Seismic Hazard Maps


US Seismic Hazard 2008

Layers | Tools

☑ Hazard maps

Transparency

○ Peak ground acceleration (%g) with 10% probability of exceedance in 50 years

● Peak ground acceleration (%g) with 2% probability of exceedance in 50 years

○ 0.2-second acceleration with 10% probability of exceedance in 50 years

○ 0.2-second acceleration with 2% probability of exceedance in 50 years

○ 1.0-second acceleration with 10% probability of exceedance in 50 years

○ 1.0-second acceleration with 2% probability of exceedance in 50 years

☐ Hazard map faults

☑ Basemap

Transparency

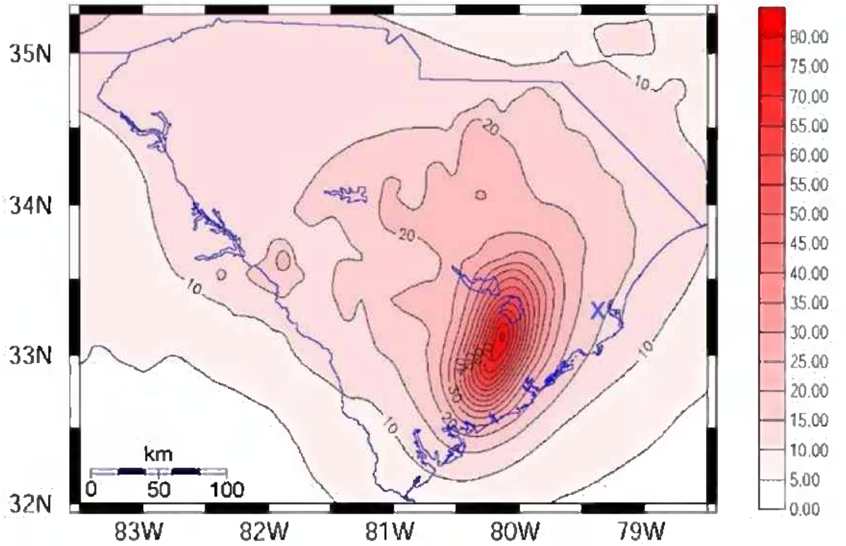
Figure 1-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years (USGS, 2008)

Note:

1. Site PGA based on USGS seismic hazard map (2008) is 0.469g.

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**



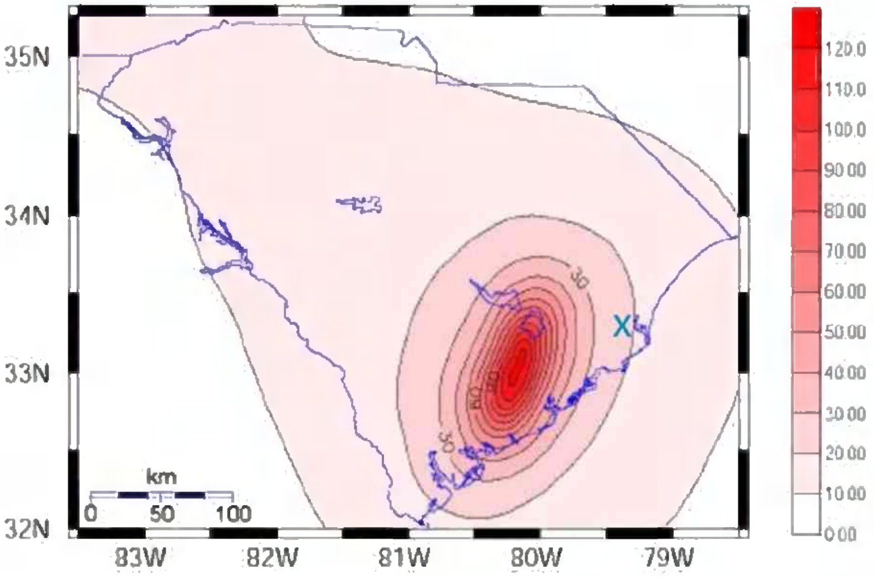


Figure 1-2. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

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.

Appendix 2

SCDOT Seismic Hazard Maps Used for Development of Target Design Spectra

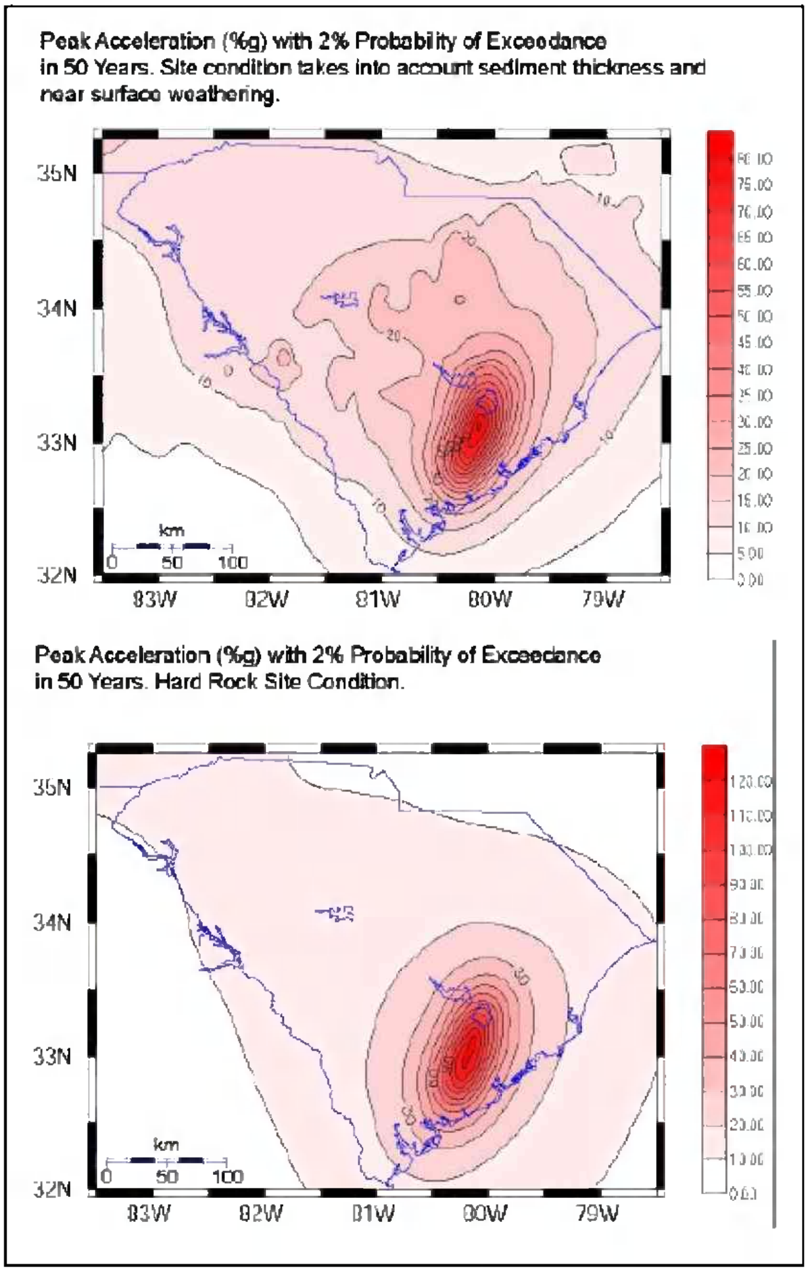
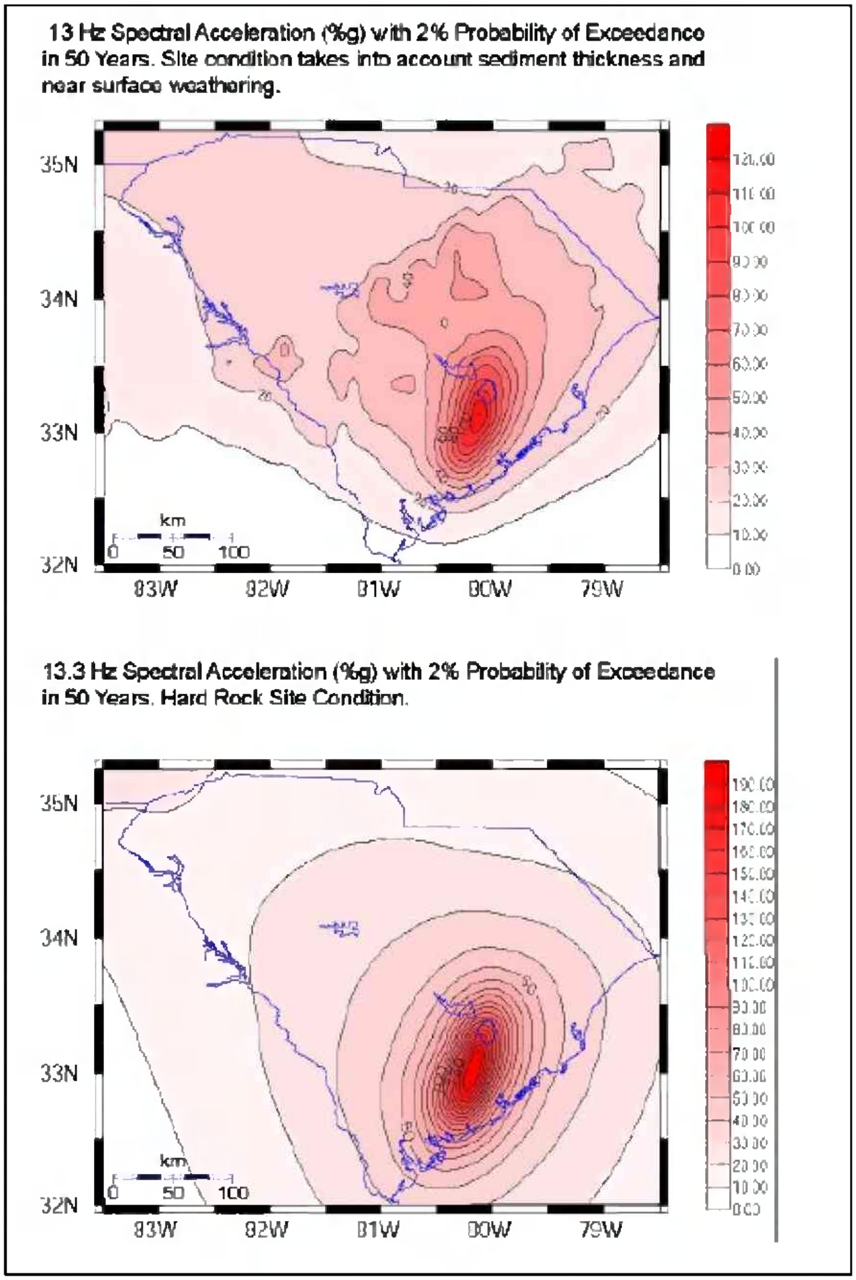


Figure 2-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.



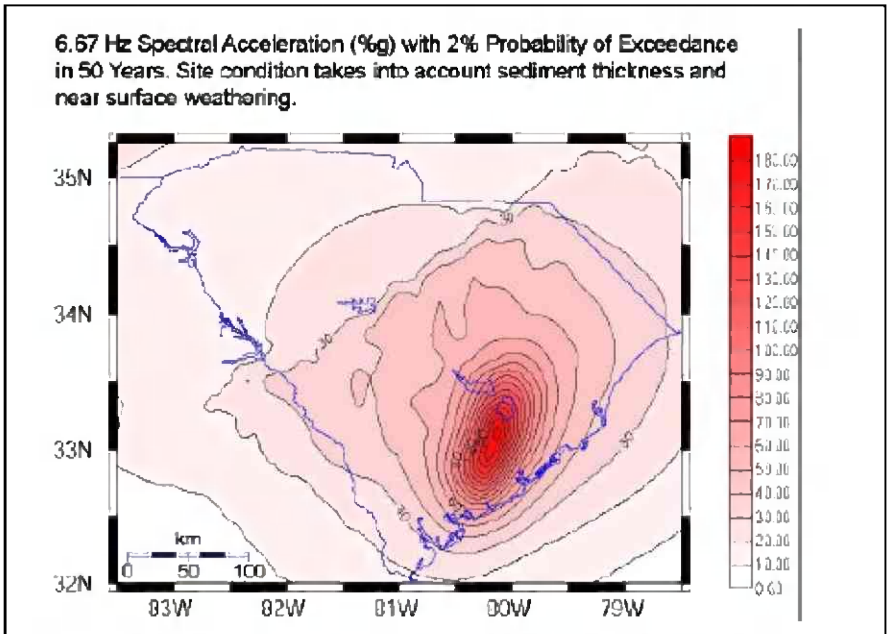
13 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

13.3 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Hard Rock Site Condition.

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.

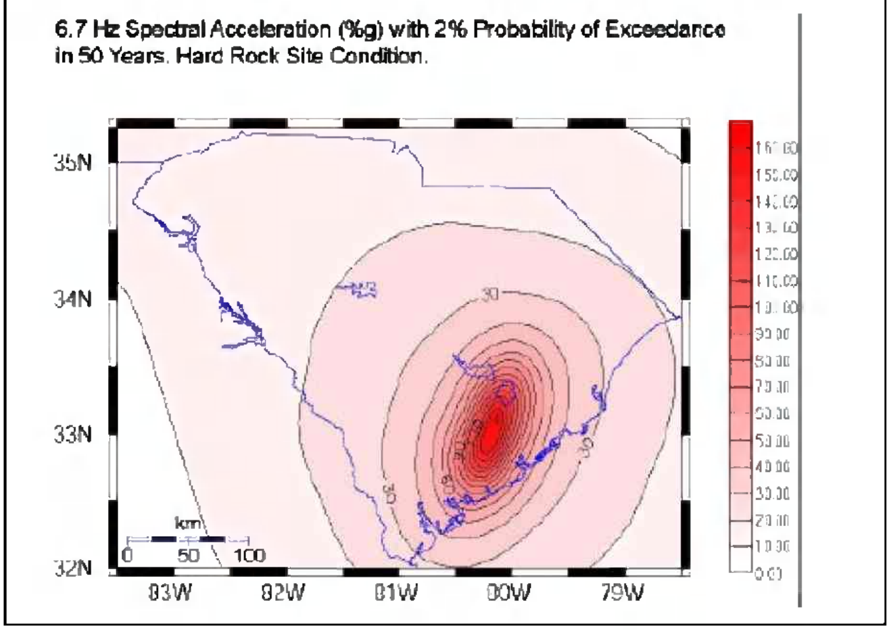
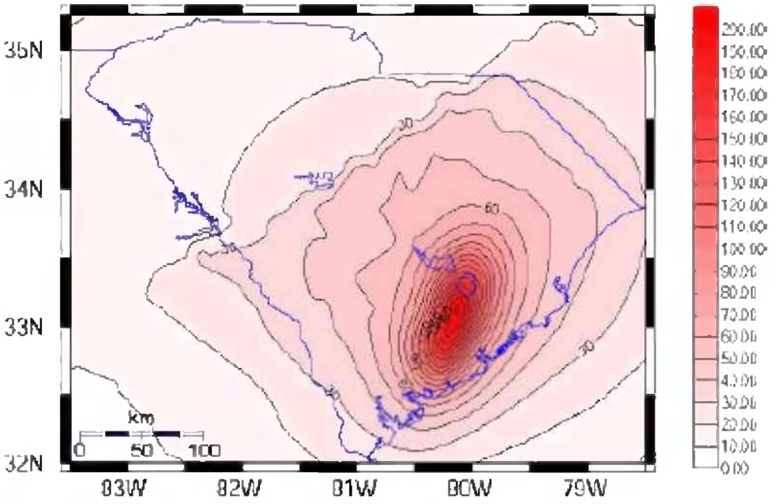


Figure 2-3. Spectral Acceleration (%) for 6.7 Hz (0.15 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

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.

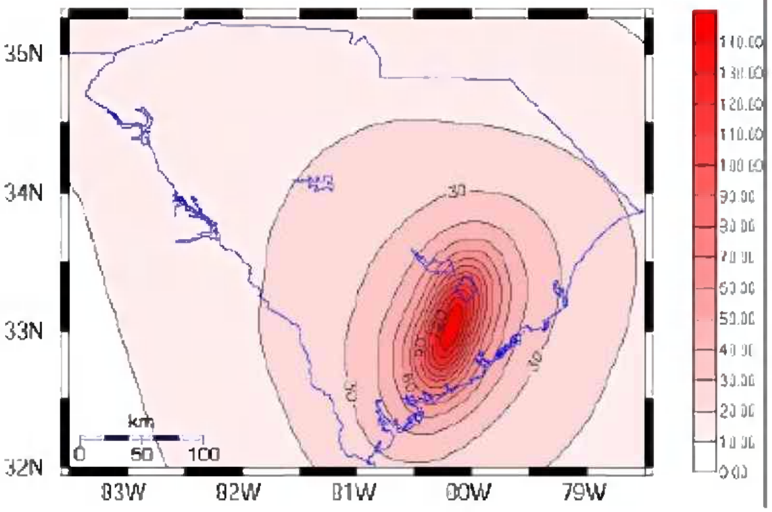
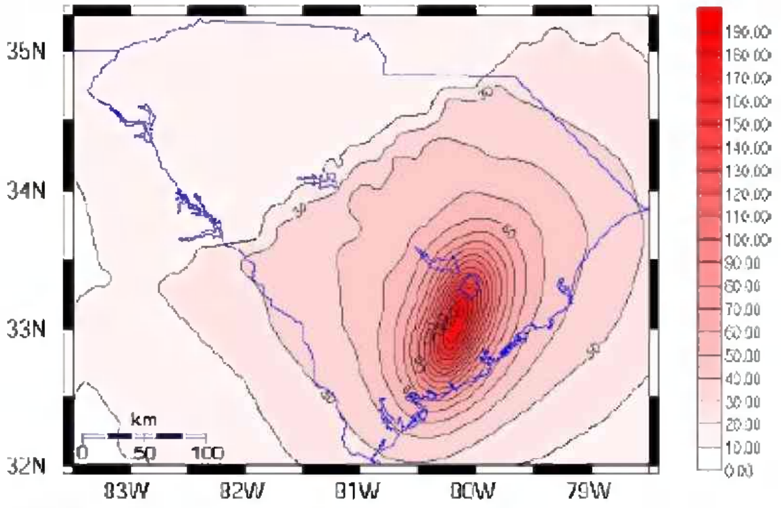


Figure 2-4. Spectral Acceleration (%) for 5 Hz (0.2 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (upper figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

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.

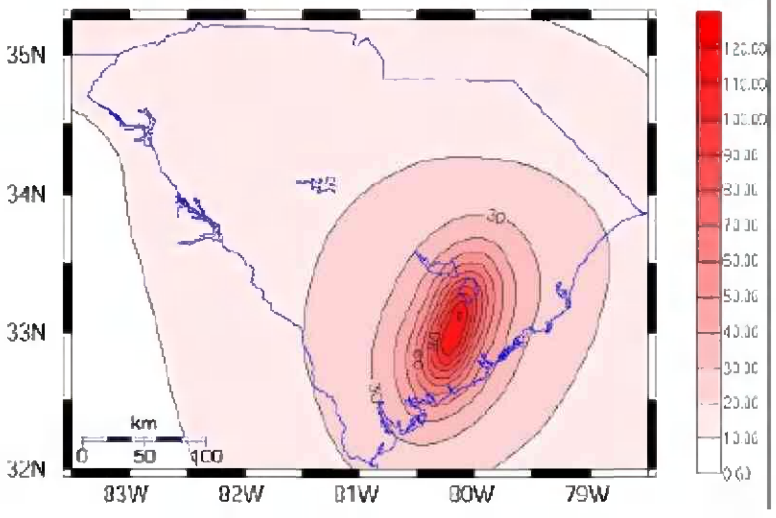
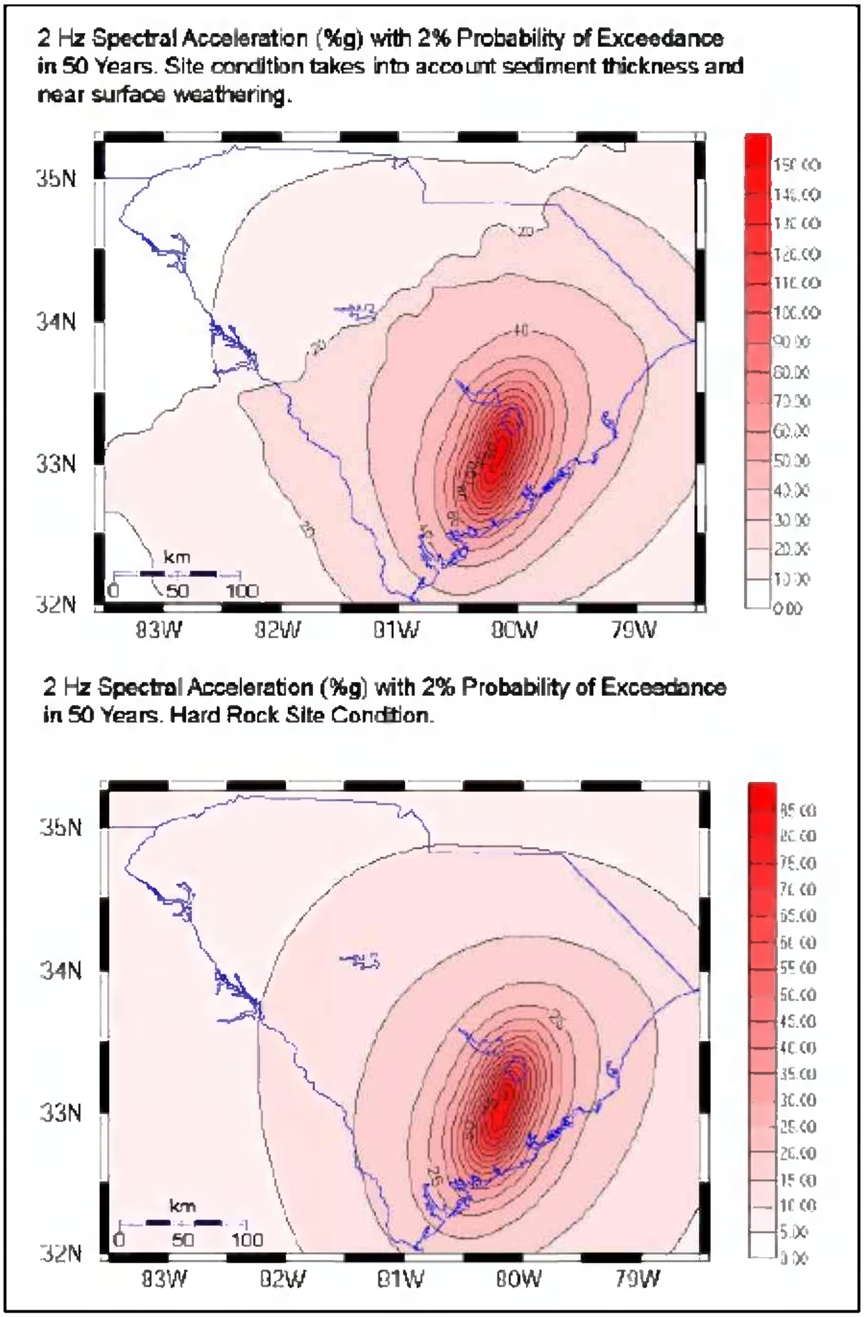


Figure 2-5. Spectral Acceleration (%) for 3.33 Hz (0.3 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.



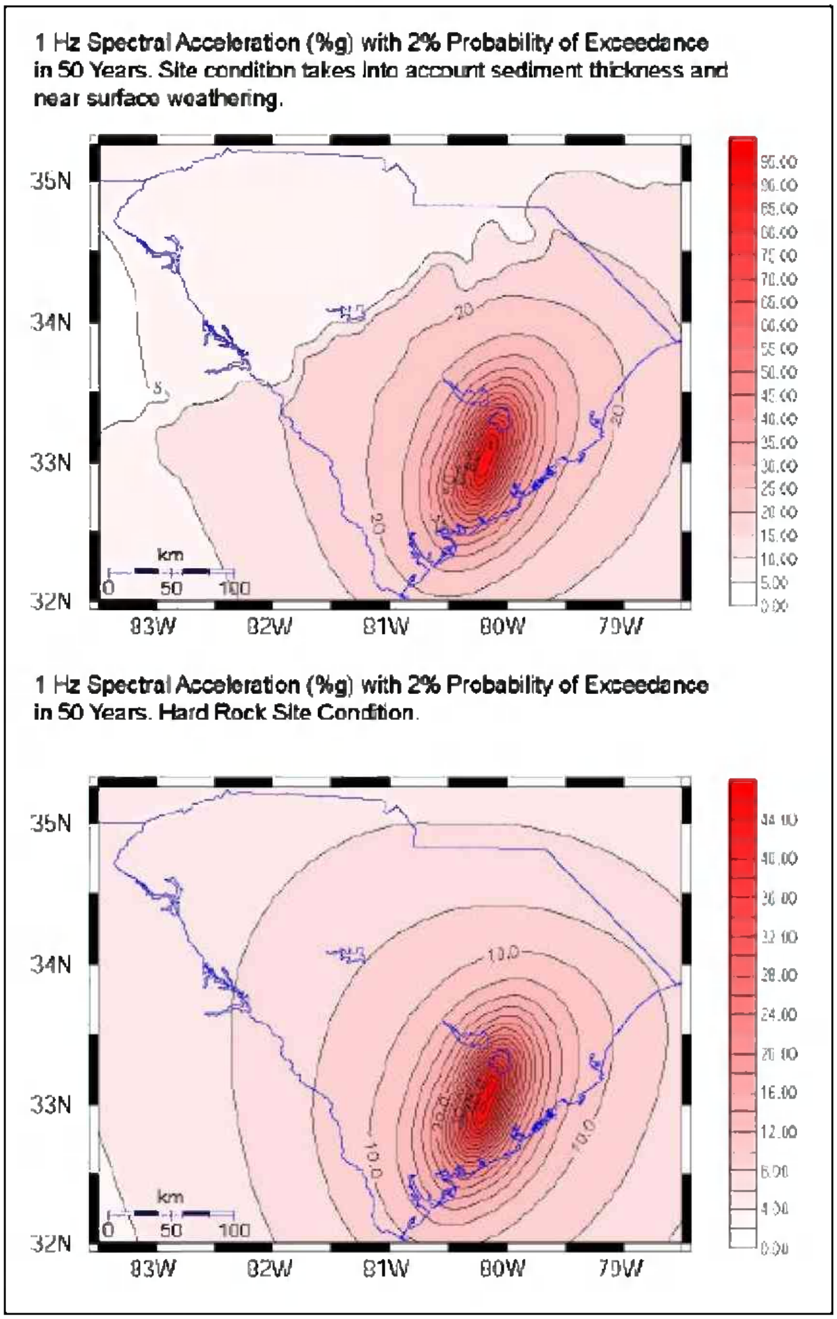
2 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

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.



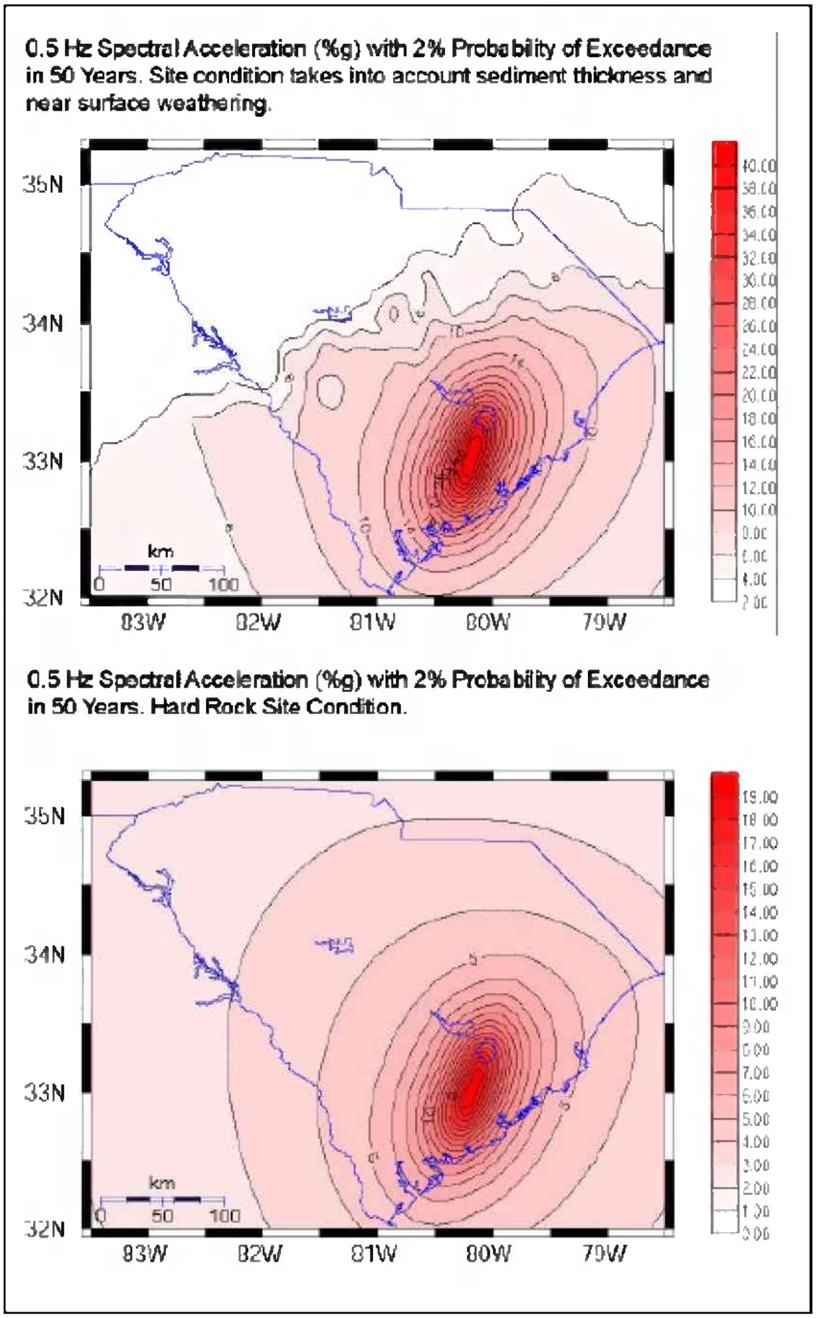
1 Hz Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 Years. Site condition takes into account sediment thickness and near surface weathering.

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

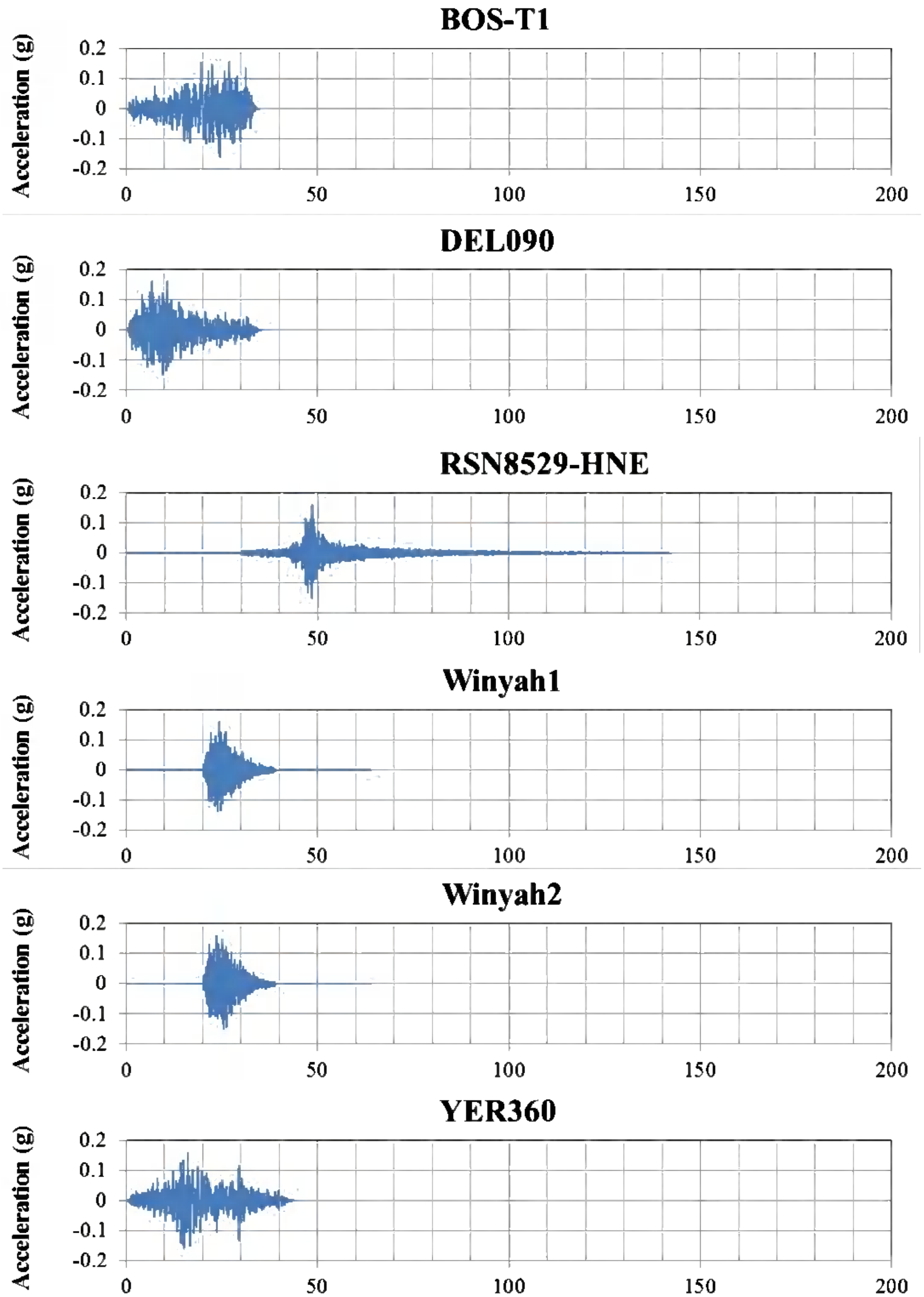


Figure 3-1. Acceleration Time Histories of Selected Earthquake Motions Scaled to PGA of 0.16g

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 foundation soil in Profile 1

(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: clayey soils with PI=75; groundwater table @ 10 ft bgs.

Step 3 – calculate σ_m'@ mid-depth of the layer (42 ft bgs)

$$\sigma_v' = \gamma H - \gamma_w H_w = 125\times18 + 115\times18 + 100\times6 - 62.4\times32 = 2923.2 \text{ psf}$$

$$\sigma_m' = \sigma_v' \,(1+2K'_o)/3 = 2923.2\times(1+2\times0.675)/3 = 2289.8 \text{ psf}$$

($K_o' = 0.6+0.001\times\text{PI} = 0.6+0.001\times75 = 0.675$, 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 foundation soils in Profile 2.

Step 5 – select the parameters α, γ_{r1}, k from Figure 4-4.

$\gamma_{r1} = 0.092\%$, $\alpha = 1.10$, $k = 0.2$

Step 6 – compute the reference strain using SCDOT GDM Equation 12-20 (see Figure 4-3 for the equation).

$$\gamma_r = \gamma_{r1}\,(\sigma_m'/P_a)^k = 0.092\times(2289.8/2089)^{0.2} = 0.0937\%$$

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 $\gamma = 0.001\%$, $G/G_{max} = 1/[1+(0.001/0.0937)] = 0.989$
If $\gamma = 0.01\%$, $G/G_{max} = 1/[1+(0.01/0.0937)] = 0.904$
If $\gamma = 0.1\%$, $G/G_{max} = 1/[1+(0.1/0.0937)] = 0.484$

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.

$D_{min1} = 0.96\%$

Step 6 – compute the small strain material damping, D_{min}, using SCDOT GDM Equation 12-28 (see Figure 4-7 for the equation).

$D_{min} = D_{min1} (\sigma_m'/P_a)^{-0.5k} = 0.96\times(2289.8/2089)^{-0.5\times0.2} = 0.951\%$

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 $\gamma = 0.001\%$, $D = 12.2\times(0.989)^2 - 34.2\times(0.989) + 22.0 + 0.951= 1.06\%$
If $\gamma = 0.01\%$, $D = 12.2\times(0.904)^2 - 34.2\times(0.904) + 22.0 + 0.951= 2.00\%$
If $\gamma = 0.1\%$, $D = 12.2\times(0.484)^2 - 34.2\times(0.484) + 22.0 + 0.951= 9.26\%$

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.

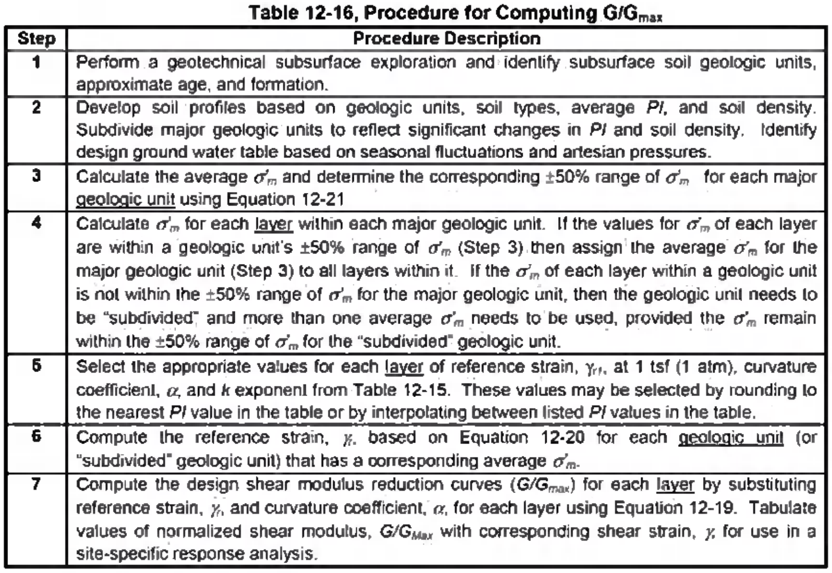


Table 12-16, Procedure for Computing G/G_{max}

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)

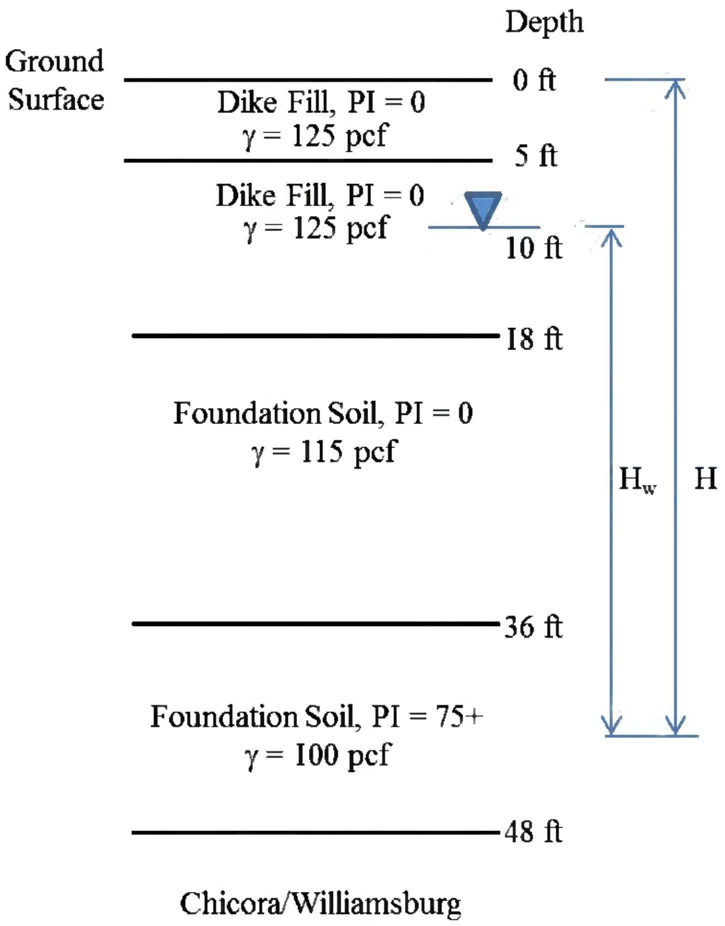
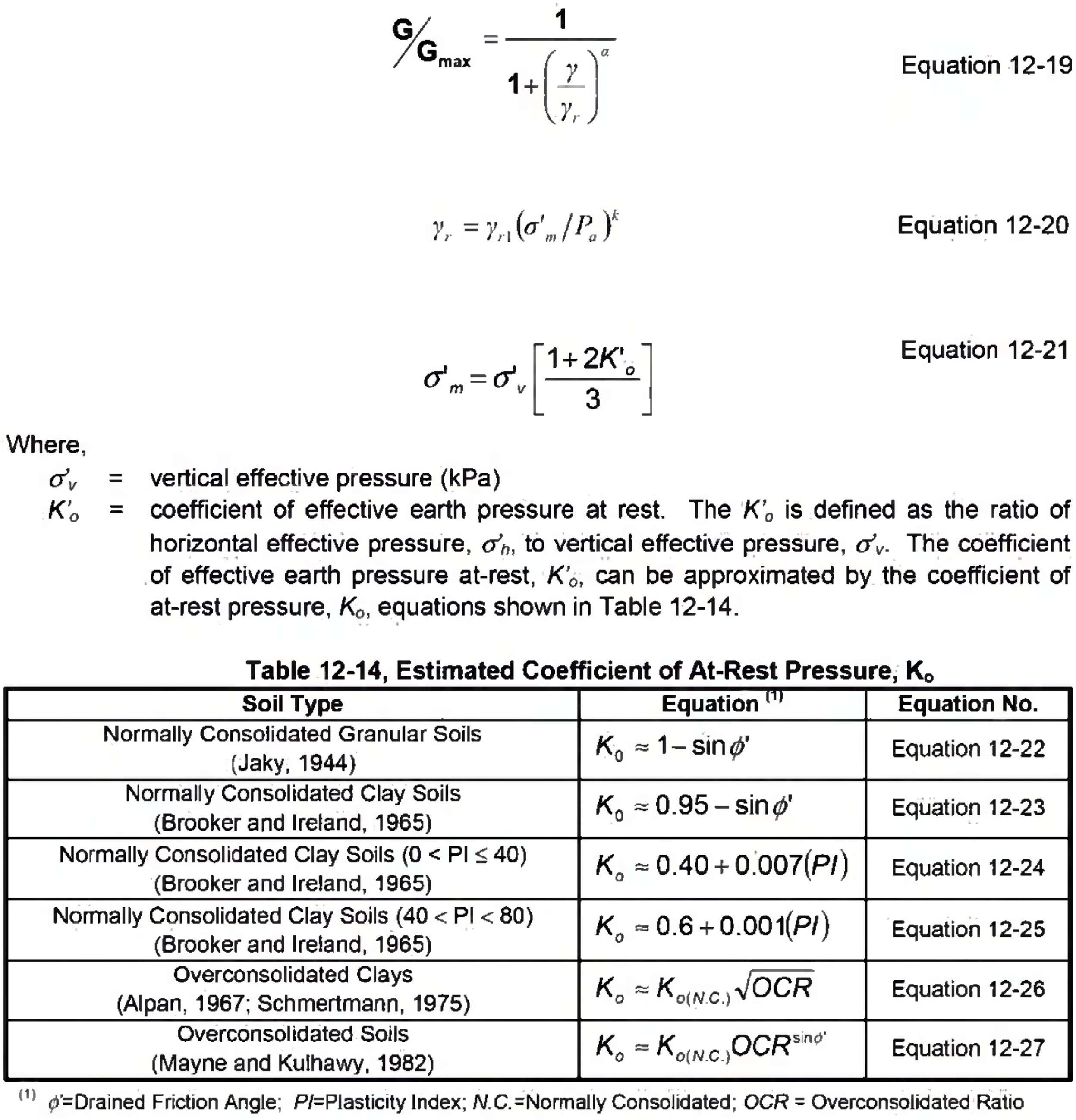


Figure 4-2. Profile 2 for the Example Calculations



$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}$$ 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)

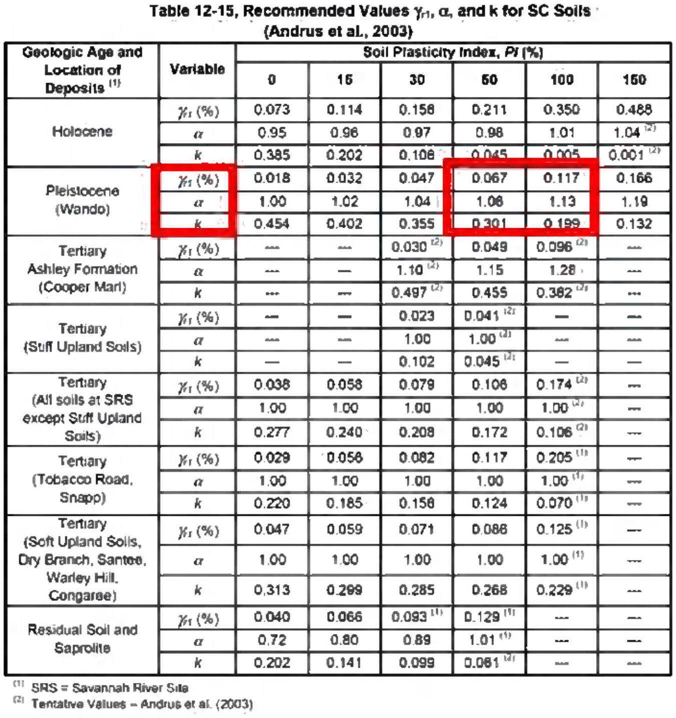


Table 12-15, Recommended Values γ_{r1}, α, and k for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits [1]	Variable	Soil Plasticity Index, *PI* (%)					
		0	15	30	50	100	150
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	0.106	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	0.106	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	0.086	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)

Note:

1. Values for PI=75 were linearly interpolated between values for PI=50 and PI=100.

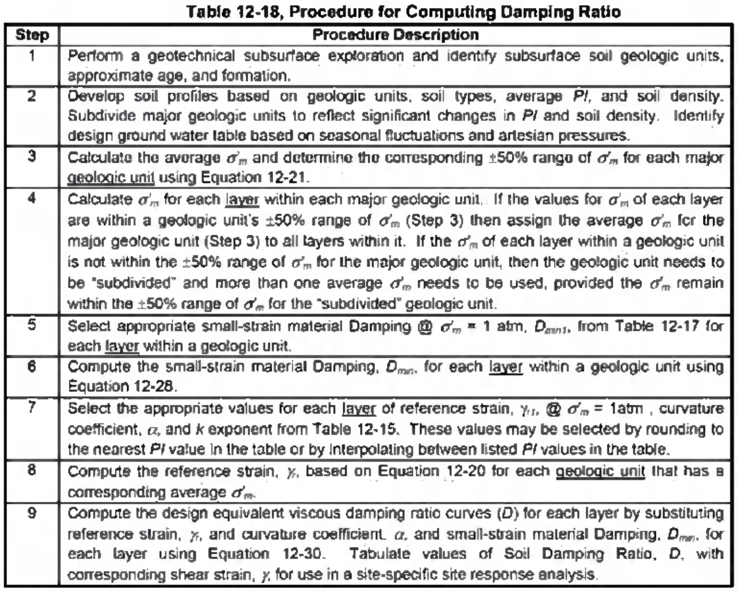


Table 12-18, Procedure for Computing Damping Ratio

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 <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 appropriate small-strain material Damping @ $\sigma'_m = 1$ atm, D_{min1}, from Table 12-17 for each <u>layer</u> within a geologic unit.
6	Compute the small-strain material Damping, D_{min}, for each <u>layer</u> within a geologic unit using Equation 12-28.
7	Select the appropriate values for each <u>layer</u> of reference strain, γ_{r1}, @ $\sigma'_m = 1$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.
8	Compute the reference strain, γ_r, based on Equation 12-20 for each <u>geologic unit</u> 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)

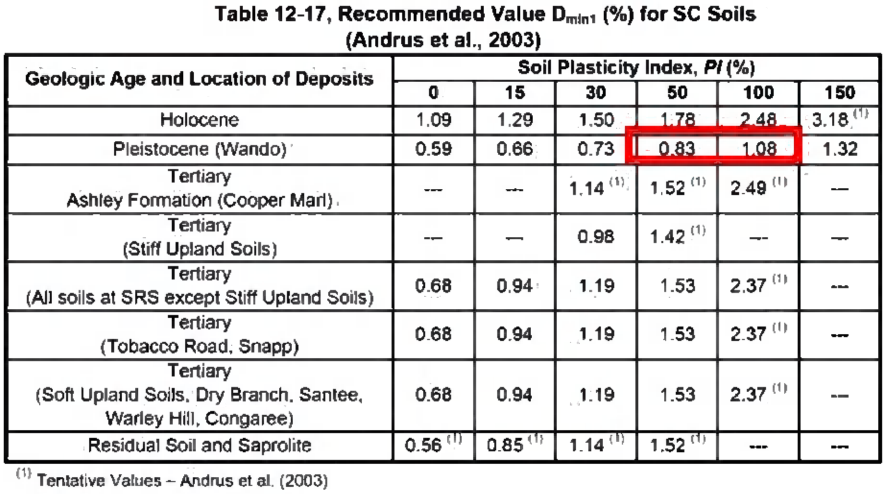


Table 12-17, Recommended Value D_{min1} (%) for SC Soils (Andrus et al., 2003)

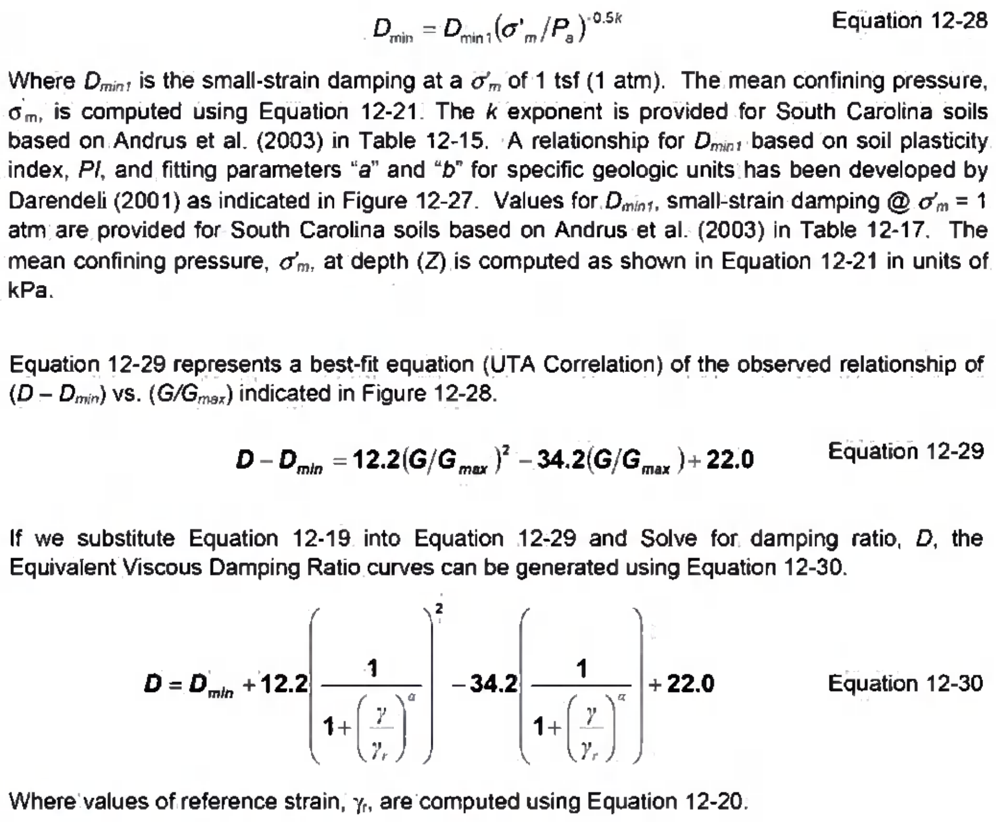
Geologic Age and Location of Deposits	Soil Plasticity Index, *PI* (%)					
	0	15	30	50	100	150
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 (1)
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	---	---	1.14 (1)	1.52 (1)	2.49 (1)	---
Tertiary (Stiff Upland Soils)	---	---	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)

Note:

1. Values for PI=75 were linearly interpolated between values for PI=50 and PI=100.



$$D_{\min} = D_{\min 1}(\sigma'_m/P_a)^{-0.5k} \qquad \text{Equation 12-28}$$

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 @ σ'_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.

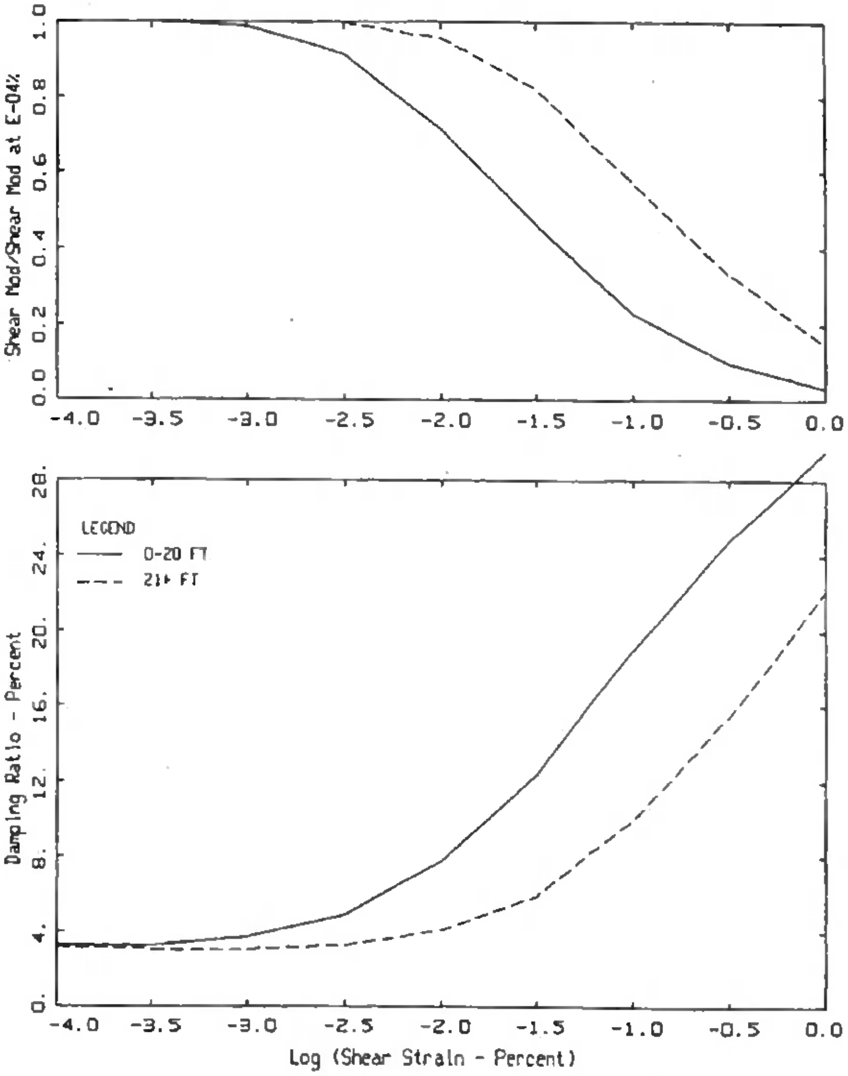
$$D - D_{min} = 12.2(G/G_{max})^2 - 34.2(G/G_{max}) + 22.0 \qquad \text{Equation 12-29}$$

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.

$$D = D_{min} + 12.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right)^2 - 34.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right) + 22.0 \qquad \text{Equation 12-30}$$

Where values of reference strain, γ_r, are computed using Equation 12-20.

Figure 4-7. Equations Needed for Damping Curve Development (SCDOT, 2010)



MODULUS REDUCTION AND DAMPING CURVES FOR ROCK

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

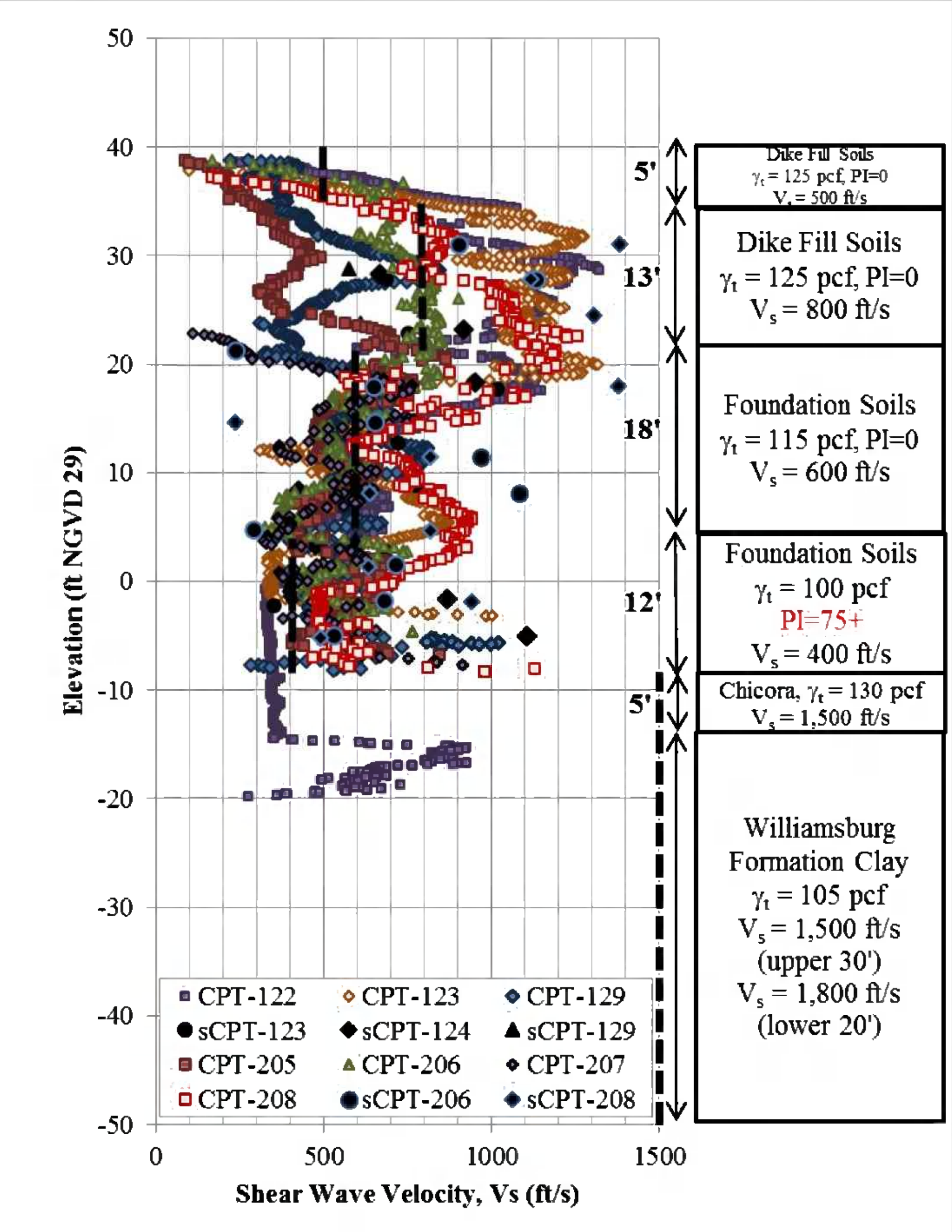


Figure 5-1a. Selected V_s Profile for the Intake/Discharge Canals Dike Model (Profile 1)

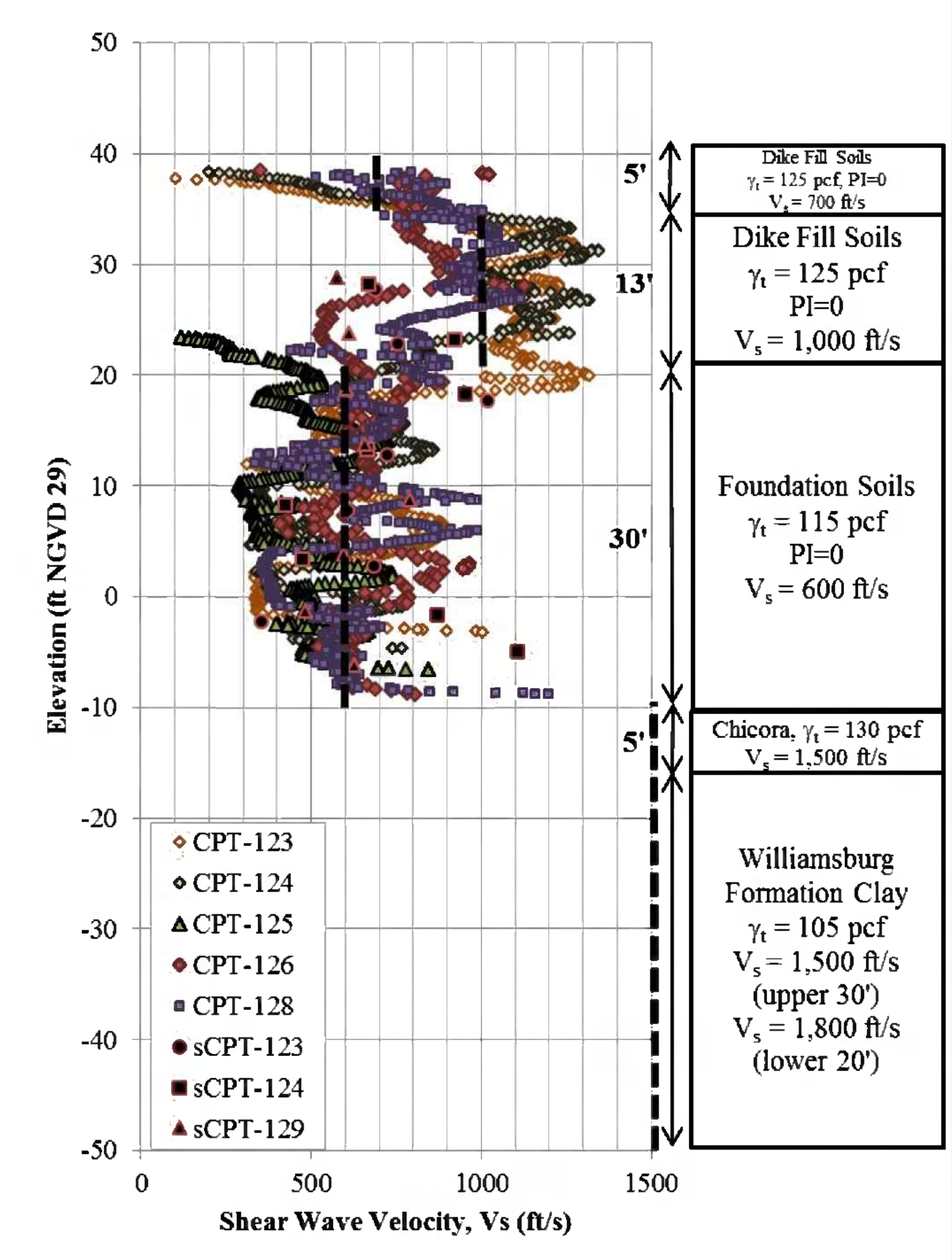


Figure 5-1b. Selected V_s Profile for the Cooling Pond Dike Model (Profile 2)

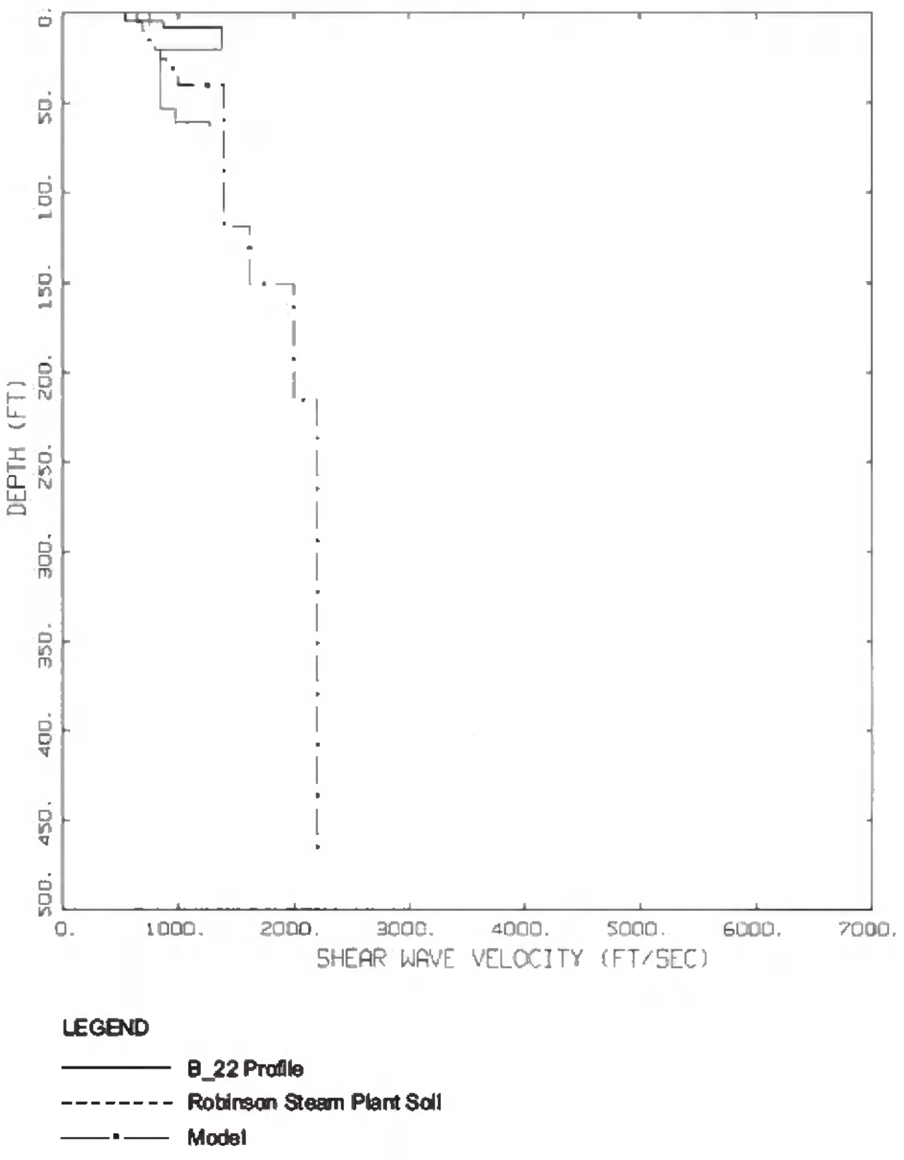


Figure 3-9. Base case shear-wave velocity profile for the Myrtle Beach site response category along with available profiles.

Figure 5-2. Reference V_s Profile for Chicora/Williamsburg Formation (URS, 2001)

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**

Appendix 6

Calculated Acceleration Profiles

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**

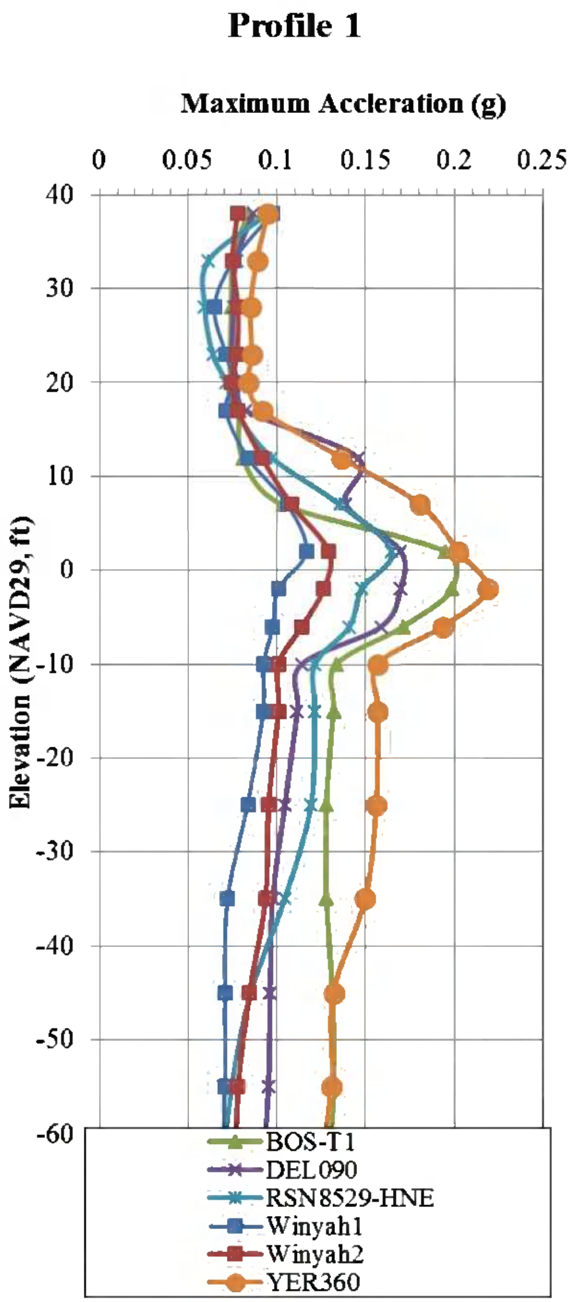


Figure 6-1. Calculated Maximum Acceleration for Profile 1

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.16g.

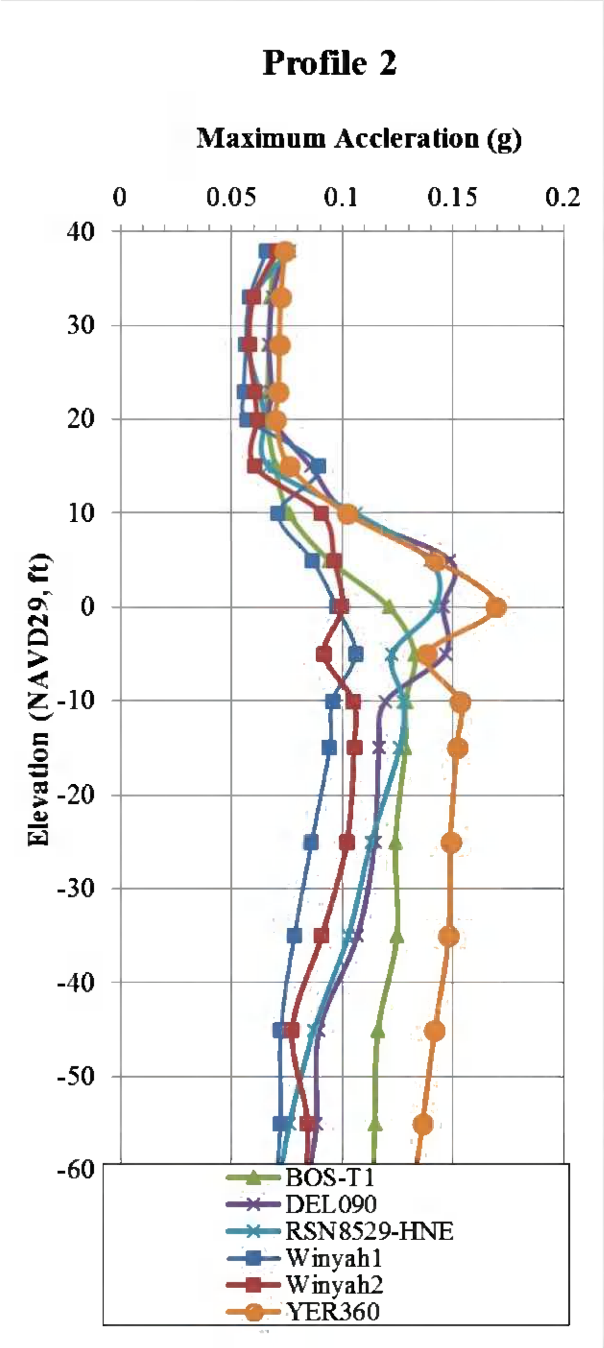


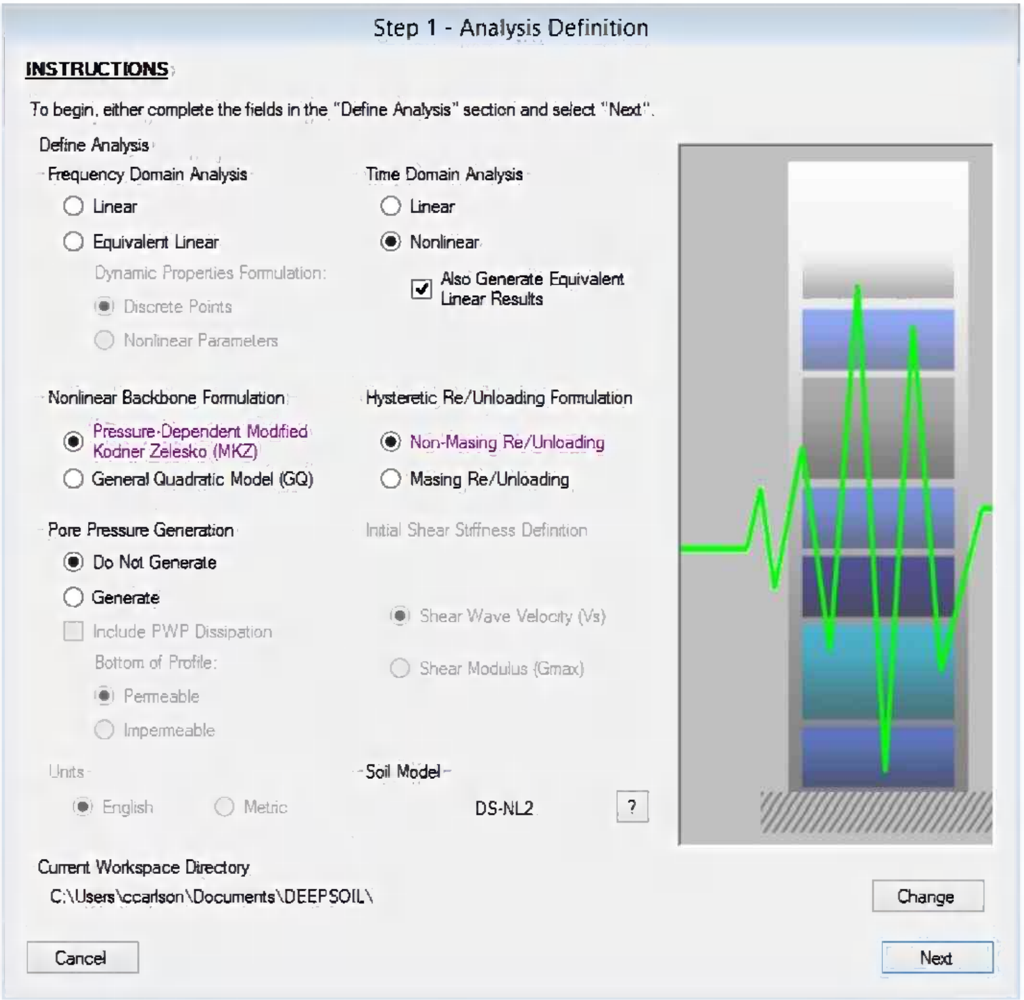
Figure 6-2. Calculated Maximum Acceleration for Profile 2

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

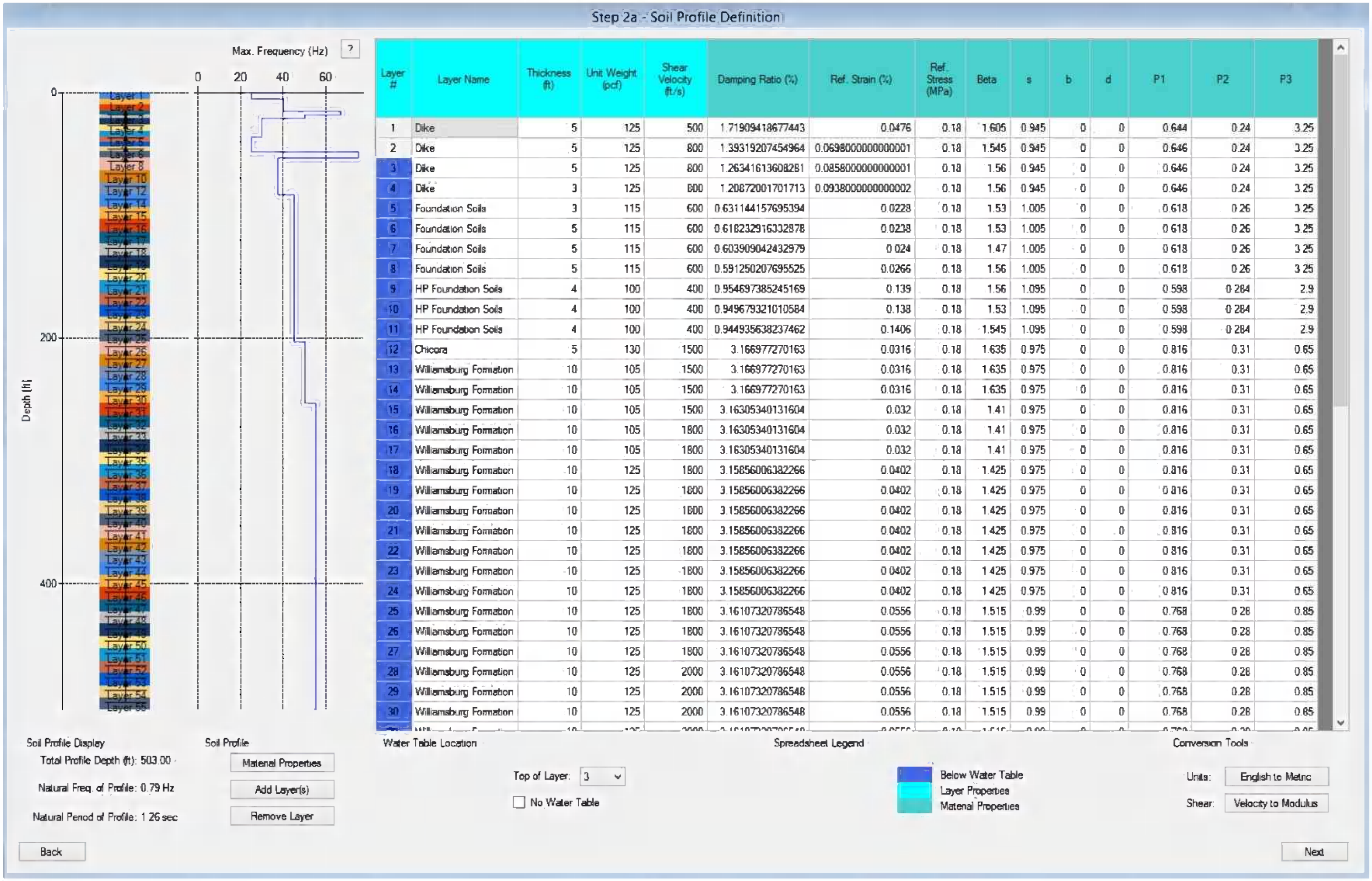
Soil Model: DS-NL2

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	500	1.71909418677443	0.0476	0.18	1.605	0.945	0	0	0.644	0.24	3.25
2	Dike	5	125	800	1.39319207454964	0.0698000000000001	0.18	1.545	0.945	0	0	0.646	0.24	3.25
3	Dike	5	125	800	1.26341613608281	0.0858000000000001	0.18	1.56	0.945	0	0	0.646	0.24	3.25
4	Dike	3	125	800	1.20872001701713	0.0938000000000002	0.18	1.56	0.945	0	0	0.646	0.24	3.25
5	Foundation Soils	3	115	600	0.631144157695394	0.0228	0.18	1.53	1.005	0	0	0.618	0.26	3.25
6	Foundation Soils	5	115	600	0.618232916332878	0.0238	0.18	1.53	1.005	0	0	0.618	0.26	3.25
7	Foundation Soils	5	115	600	0.603909042432979	0.024	0.18	1.47	1.005	0	0	0.618	0.26	3.25
8	Foundation Soils	5	115	600	0.591250207695525	0.0266	0.18	1.56	1.005	0	0	0.618	0.26	3.25
9	HP Foundation Soils	4	100	400	0.954697385245169	0.139	0.18	1.56	1.095	0	0	0.598	0.284	2.9
10	HP Foundation Soils	4	100	400	0.949679321010584	0.138	0.18	1.53	1.095	0	0	0.598	0.284	2.9
11	HP Foundation Soils	4	100	400	0.944935638237462	0.1406	0.18	1.545	1.095	0	0	0.598	0.284	2.9
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): 503.00

Natural Freq. of Profile: 0.79 Hz

Natural Period of Profile: 1.26 sec

Water Table Location

Top of Layer: 3

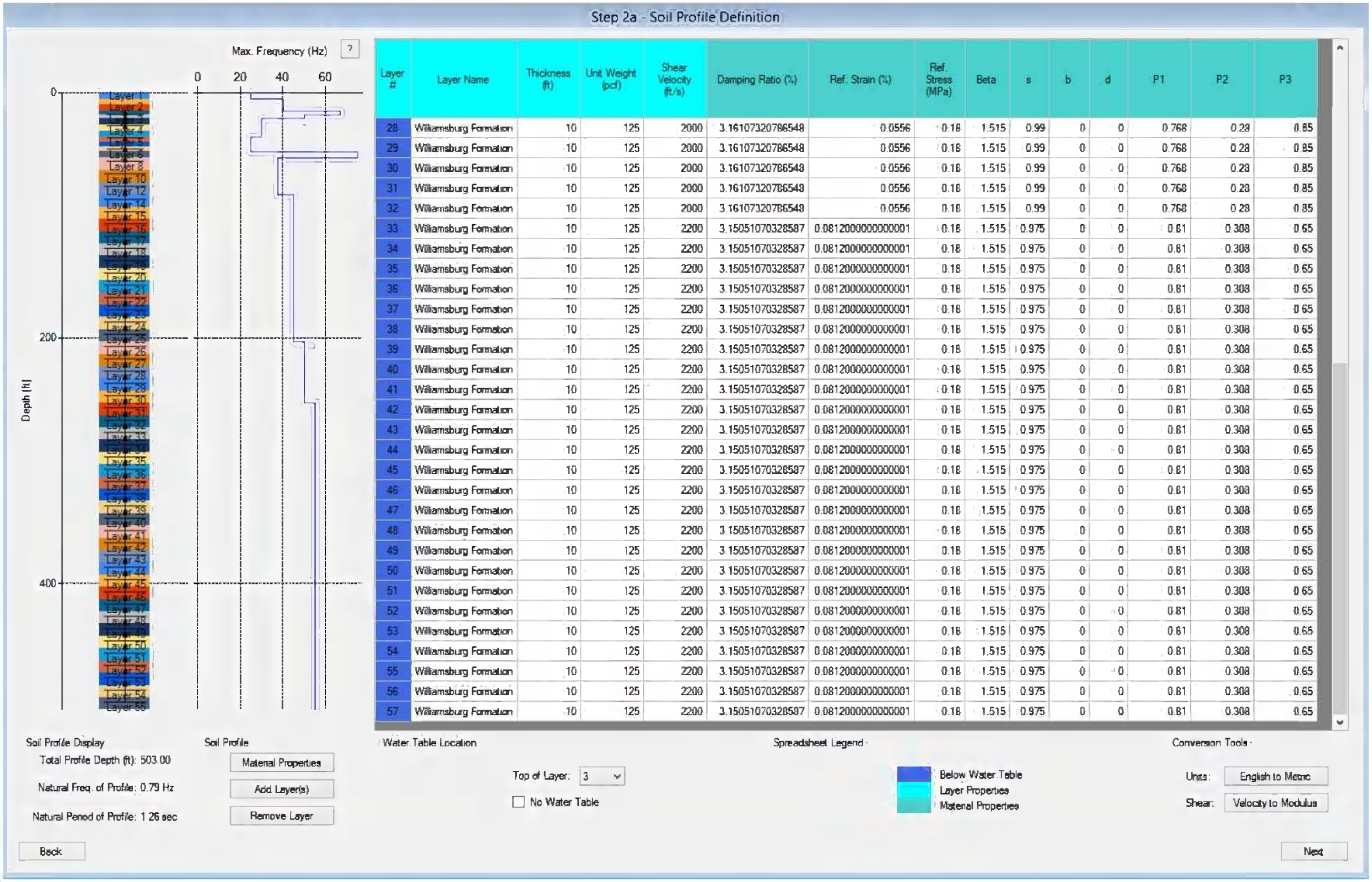
☐ No Water Table

Spreadsheet Legend

🟦 Below Water Table
Layer Properties
Material Properties

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
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): 503.00

Natural Freq. of Profile: 0.79 Hz

Natural Period of Profile: 1.26 sec

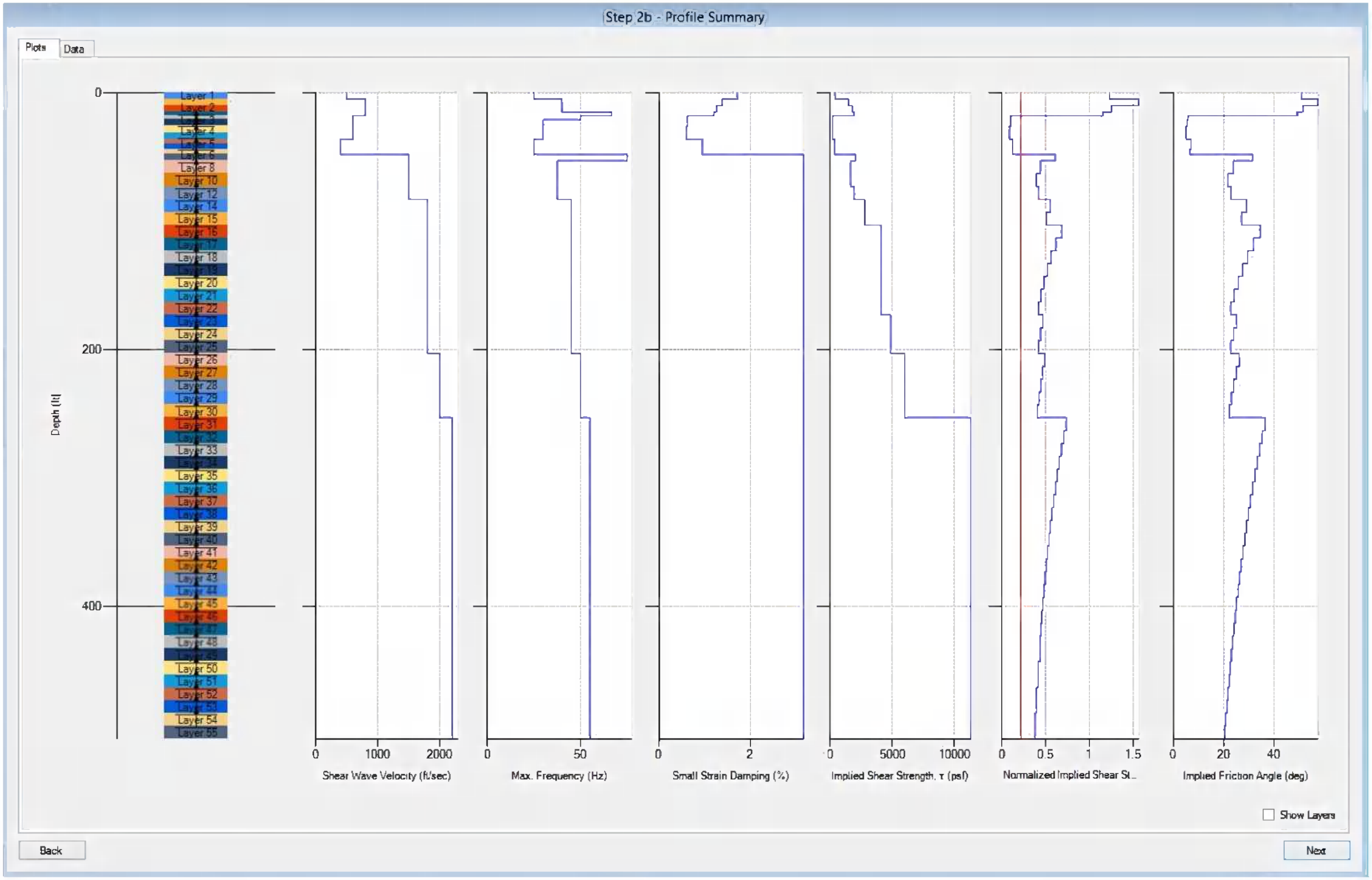
Water Table Location

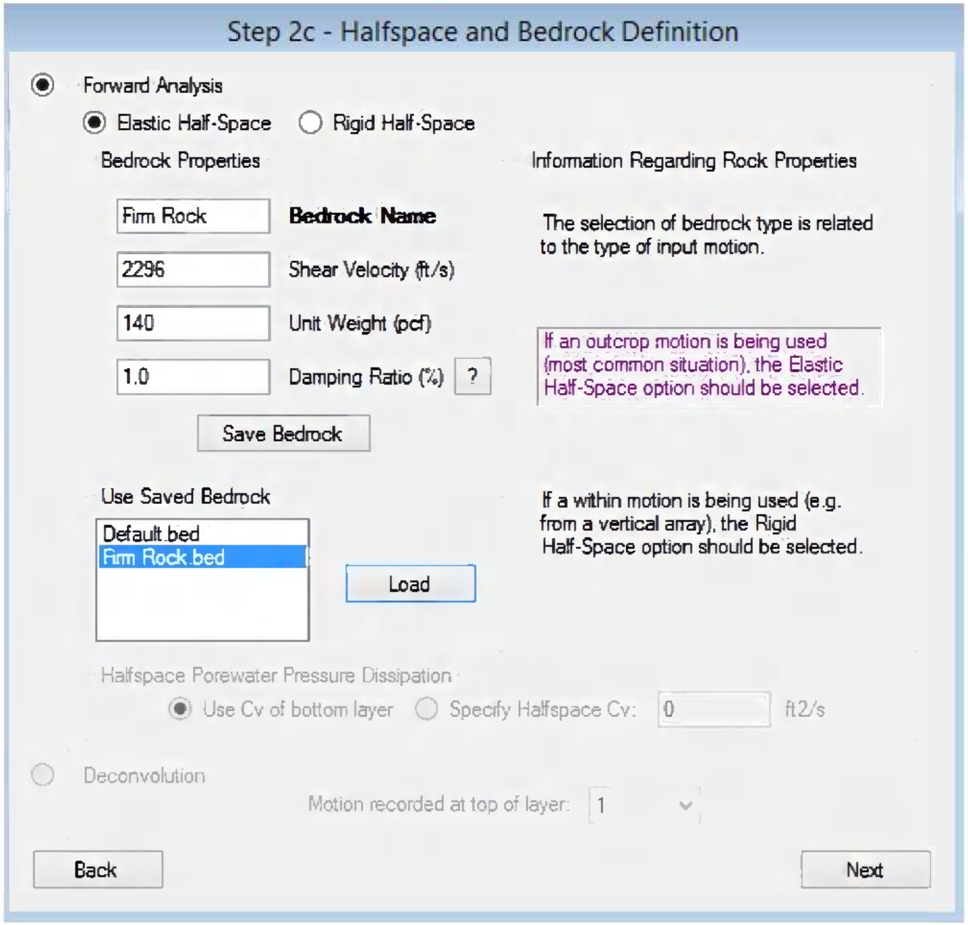
Top of Layer: 3

☐ No Water Table

Spreadsheet Legend

🟦	Below Water Table
🟦	Layer Properties
🟦	Material Properties


Step 2b - Profile Summary
Plots
Data
Depth (ft)
0
200
400
Layer 1
Layer 2
Layer 4
Layer 6
Layer 8
Layer 10
Layer 12
Layer 14
Layer 15
Layer 16
Layer 18
Layer 20
Layer 21
Layer 22
Layer 24
Layer 25
Layer 26
Layer 27
Layer 28
Layer 29
Layer 30
Layer 31
Layer 33
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 48
Layer 50
Layer 51
Layer 52
Layer 53
Layer 54
Layer 55
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
0.5
1
1.5
Normalized Implied Shear St...
0
20
40
Implied Friction Angle (deg)
☐ Show Layers
Back
Next



● Forward Analysis

● Elastic Half-Space ○ Rigid Half-Space

Bedrock Properties

Firm Rock **Bedrock Name**

2296 Shear Velocity (ft/s)

140 Unit Weight (pcf)

1.0 Damping Ratio (%)

Use Saved Bedrock

Default.bed
Firm Rock.bed

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.

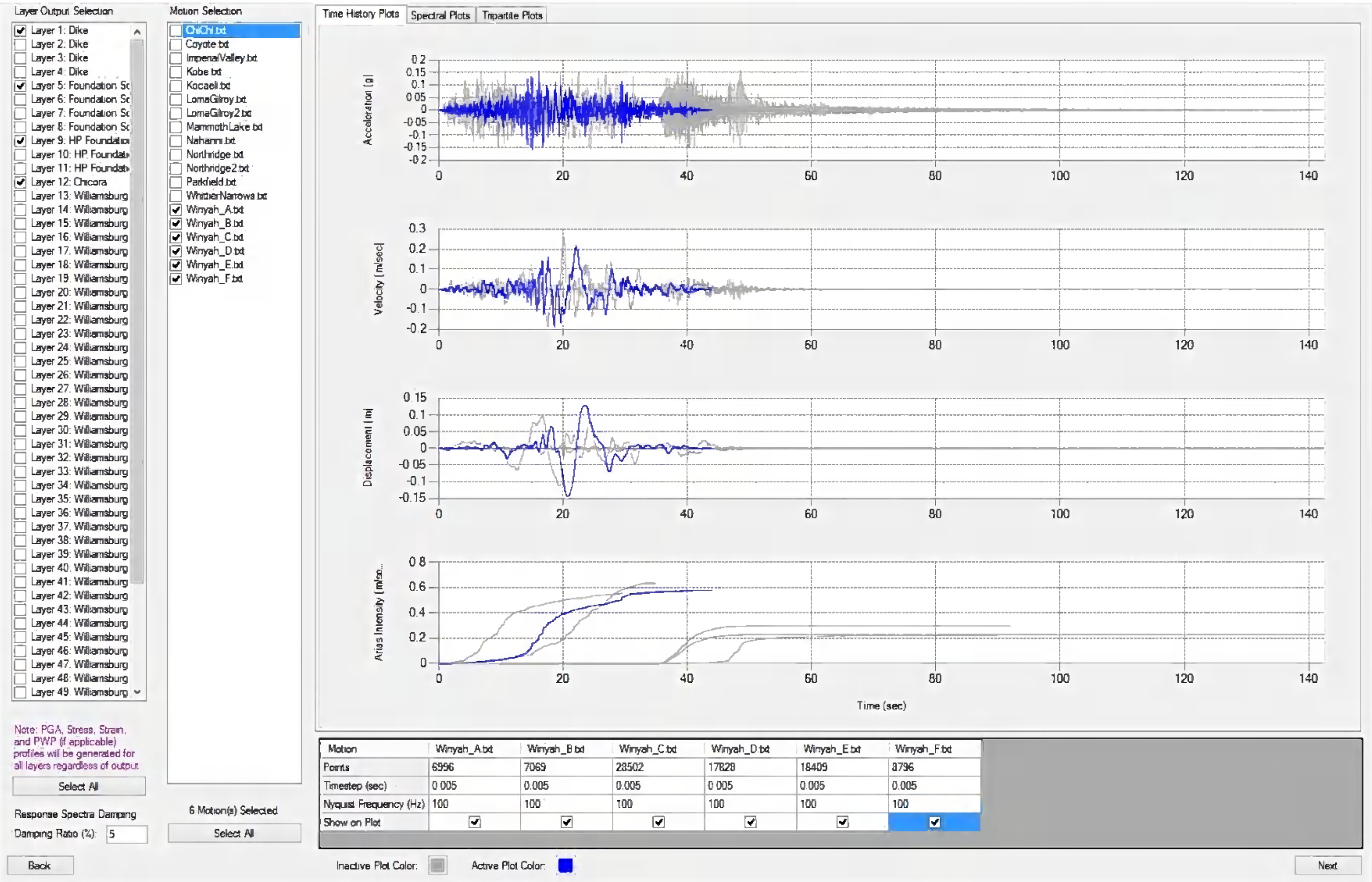
If a within motion is being used (e.g. from a vertical array), the Rigid Half-Space option should be selected.

Halfspace Porewater Pressure Dissipation

● Use Cv of bottom layer ○ Specify Halfspace Cv: 0 ft2/s

○ Deconvolution

Motion recorded at top of layer: 1



Layer Output Selection

- ☑ Layer 1: Dike
- ☐ Layer 2: Dike
- ☐ Layer 3: Dike
- ☐ Layer 4: Dike
- ☑ Layer 5: Foundation Sc
- ☐ Layer 6: Foundation Sc
- ☐ Layer 7: Foundation Sc
- ☐ Layer 8: Foundation Sc
- ☑ Layer 9: HP Foundatio
- ☐ Layer 10: HP Foundat
- ☐ Layer 11: HP Foundat
- ☑ Layer 12: Chicora
- ☐ 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

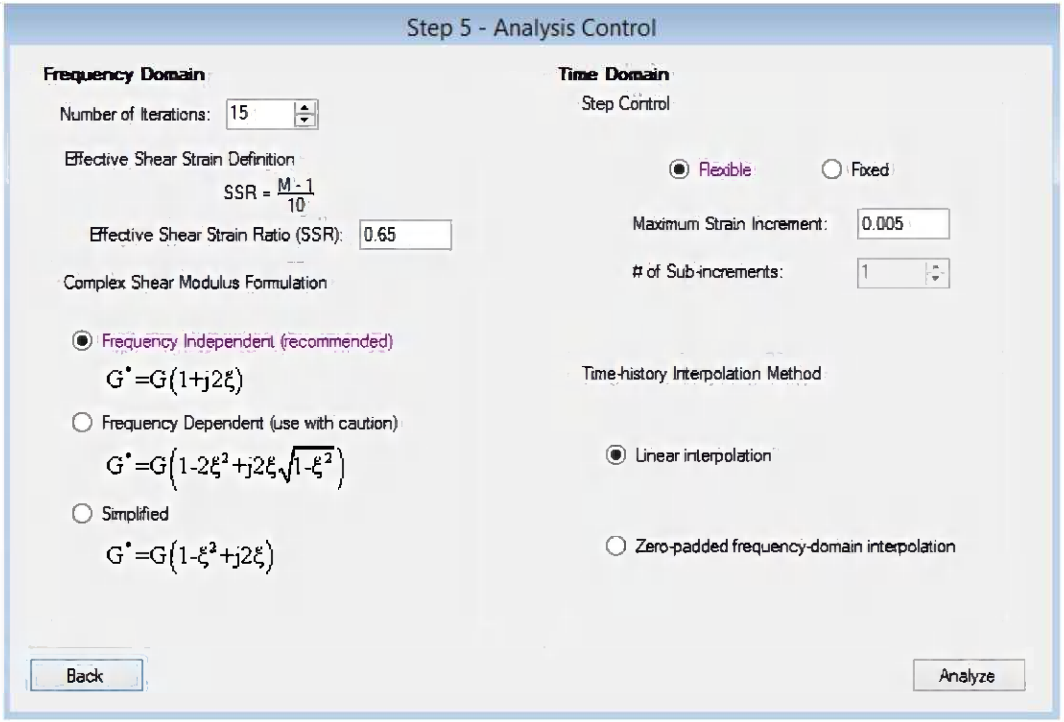
Motion Selection

- ☐ 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	☑	☑	☑	☑	☑	☑

Inactive Plot Color: ▨ Active Plot Color: 🟦



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

$G^* = G(1-\xi^2+j2\xi)$

Time Domain

Step Control

● Flexible ○ Fixed

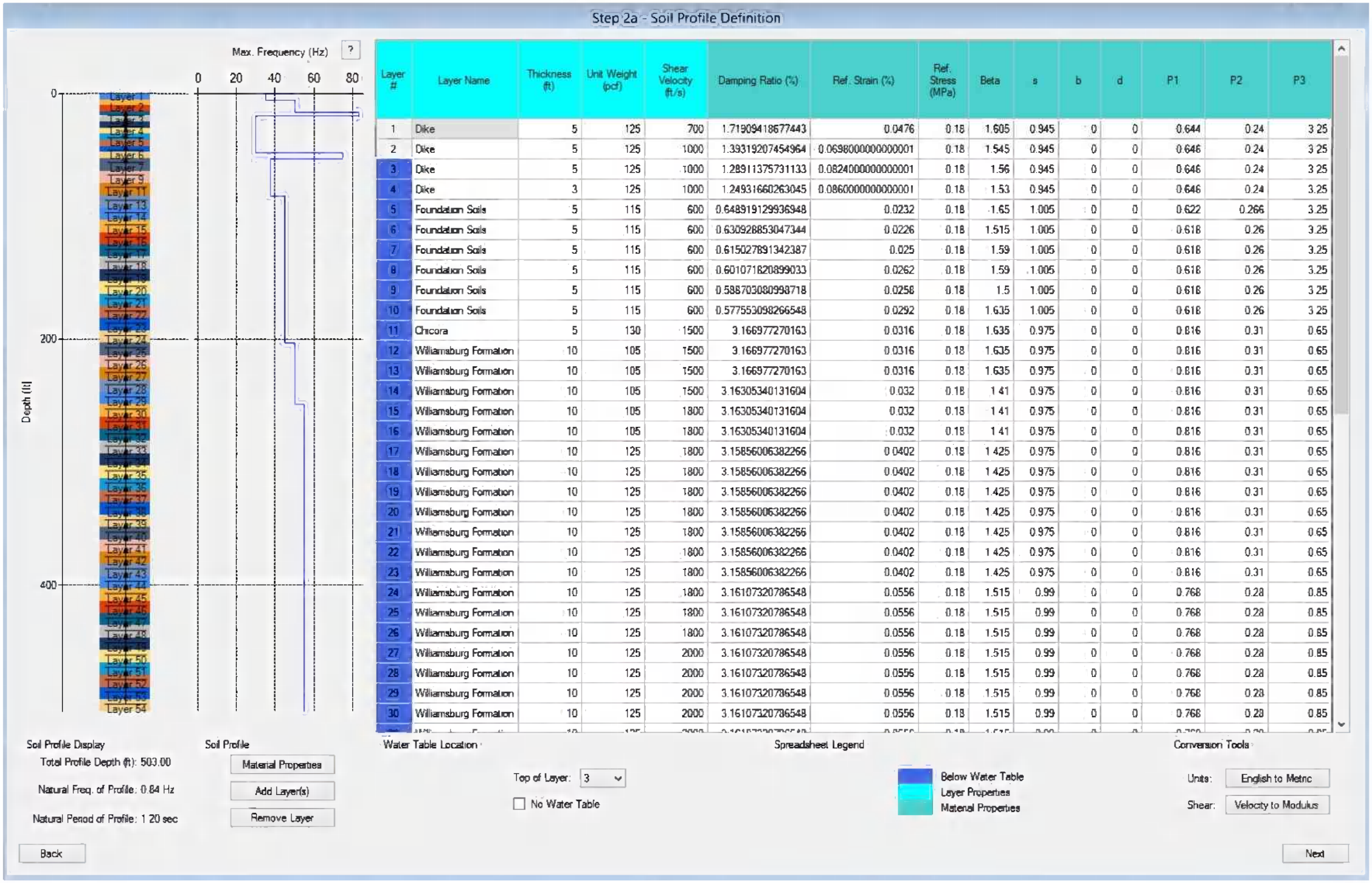
Maximum Strain Increment: 0.005

# of Sub-increments: 1

Time-history Interpolation Method

● Linear interpolation

○ Zero-padded frequency-domain interpolation



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
1	Dike	5	125	700	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	1.28911375731133	0.0824000000000001	0.18	1.56	0.945	0	0	0.646	0.24	3.25
4	Dike	3	125	1000	1.24931660263045	0.0860000000000001	0.18	1.53	0.945	0	0	0.646	0.24	3.25
5	Foundation Soils	5	115	600	0.648919129936948	0.0232	0.18	1.65	1.005	0	0	0.622	0.266	3.25
6	Foundation Soils	5	115	600	0.630928853047344	0.0226	0.18	1.515	1.005	0	0	0.618	0.26	3.25
7	Foundation Soils	5	115	600	0.615027891342387	0.025	0.18	1.59	1.005	0	0	0.618	0.26	3.25
8	Foundation Soils	5	115	600	0.601071820899033	0.0262	0.18	1.59	1.005	0	0	0.618	0.26	3.25
9	Foundation Soils	5	115	600	0.588703080998718	0.0258	0.18	1.5	1.005	0	0	0.618	0.26	3.25
10	Foundation Soils	5	115	600	0.577553098266548	0.0292	0.18	1.635	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): 503.00

Natural Freq. of Profile: 0.84 Hz

Natural Period of Profile: 1.20 sec

Soil Profile

Water Table Location

Top of Layer: 3

☐ No Water Table

Spreadsheet Legend

🟦 Below Water Table

🟦 Layer Properties

🟦 Material Properties

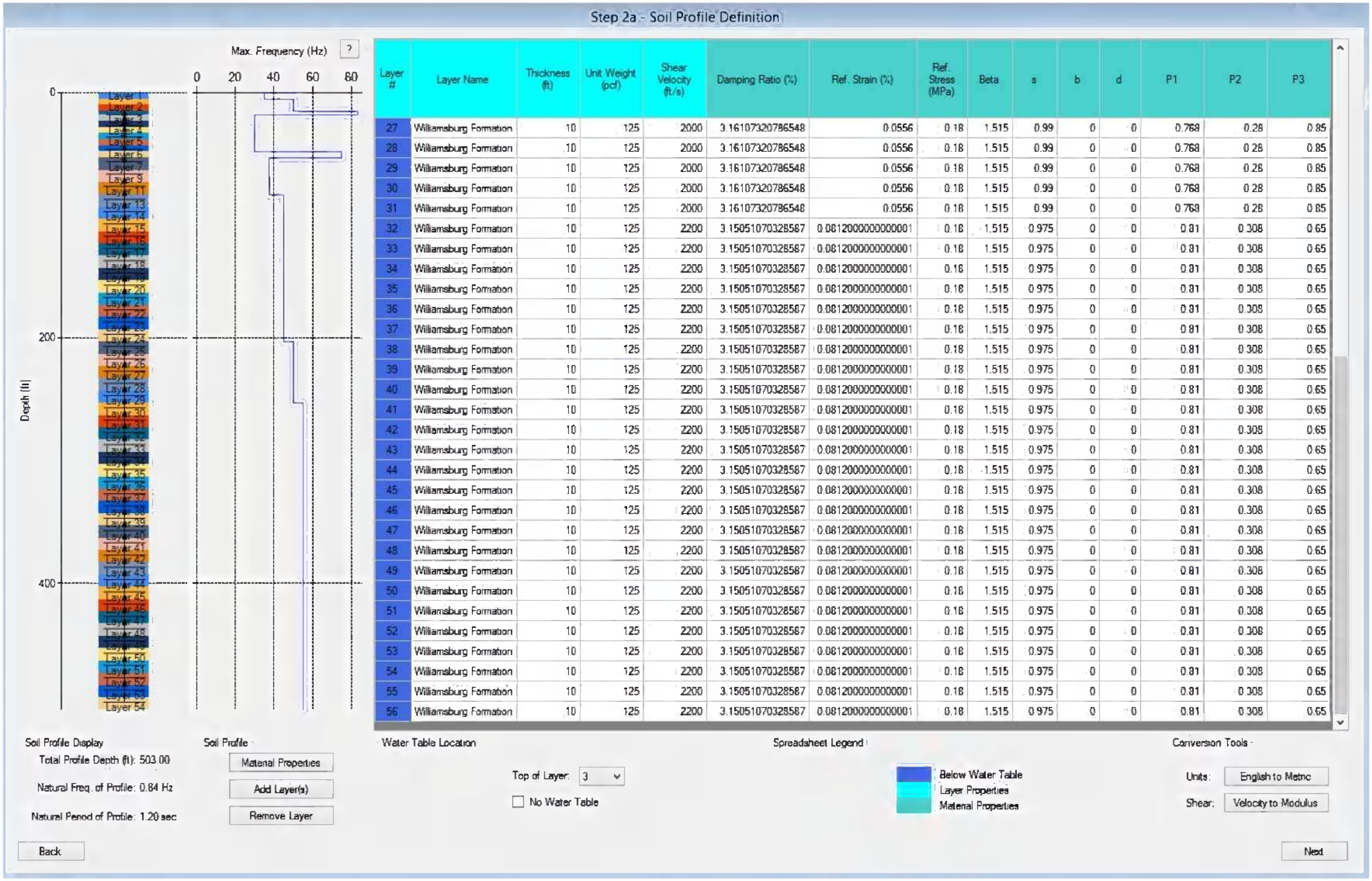
Conversion Tools

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	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
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

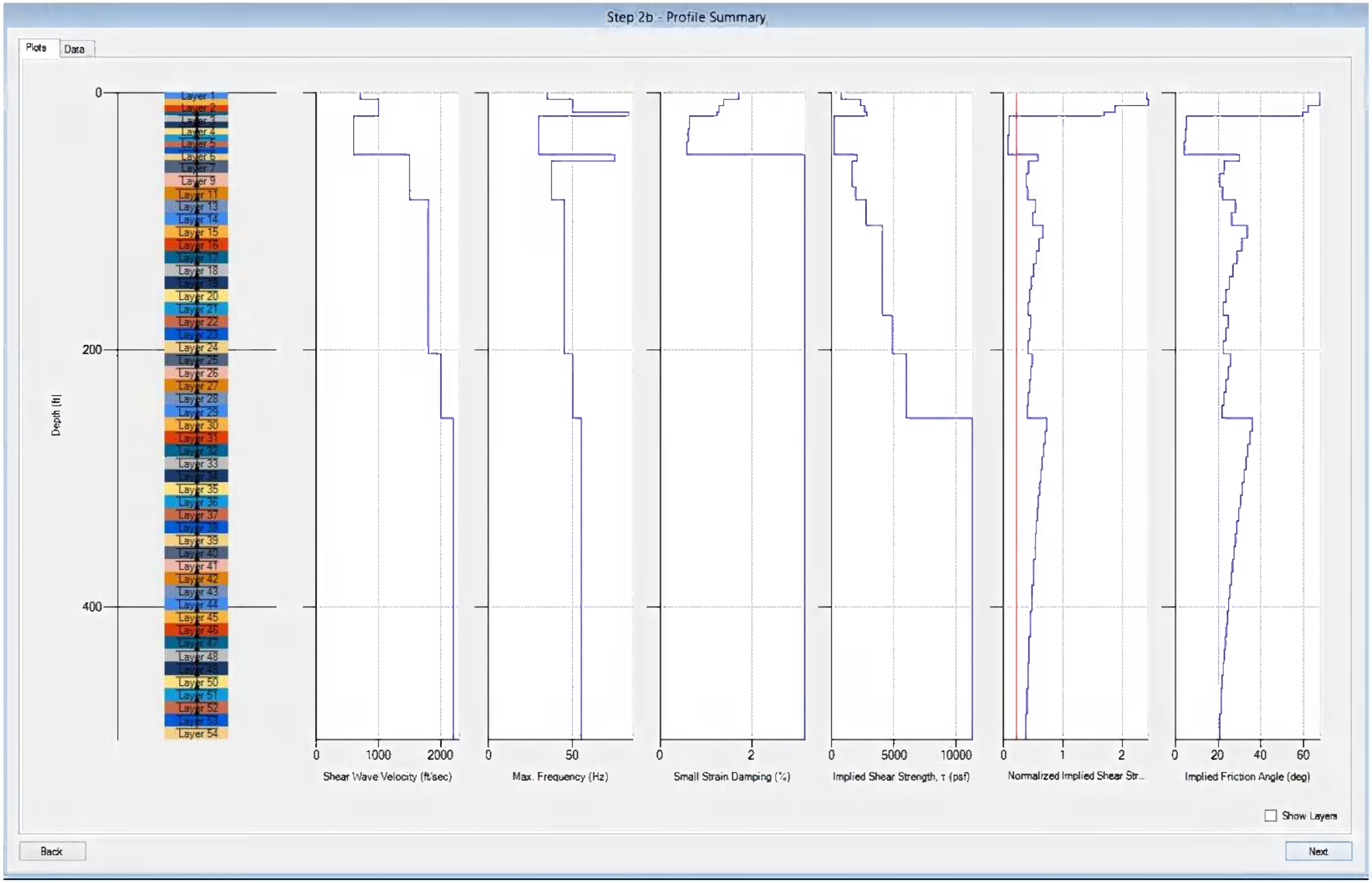
Total Profile Depth (ft): 503.00

Natural Freq. of Profile: 0.84 Hz

Natural Period of Profile: 1.20 sec

Top of Layer: 3

☐ 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 7
Layer 9
Layer 11
Layer 13
Layer 14
Layer 15
Layer 16
Layer 17
Layer 18
Layer 19
Layer 20
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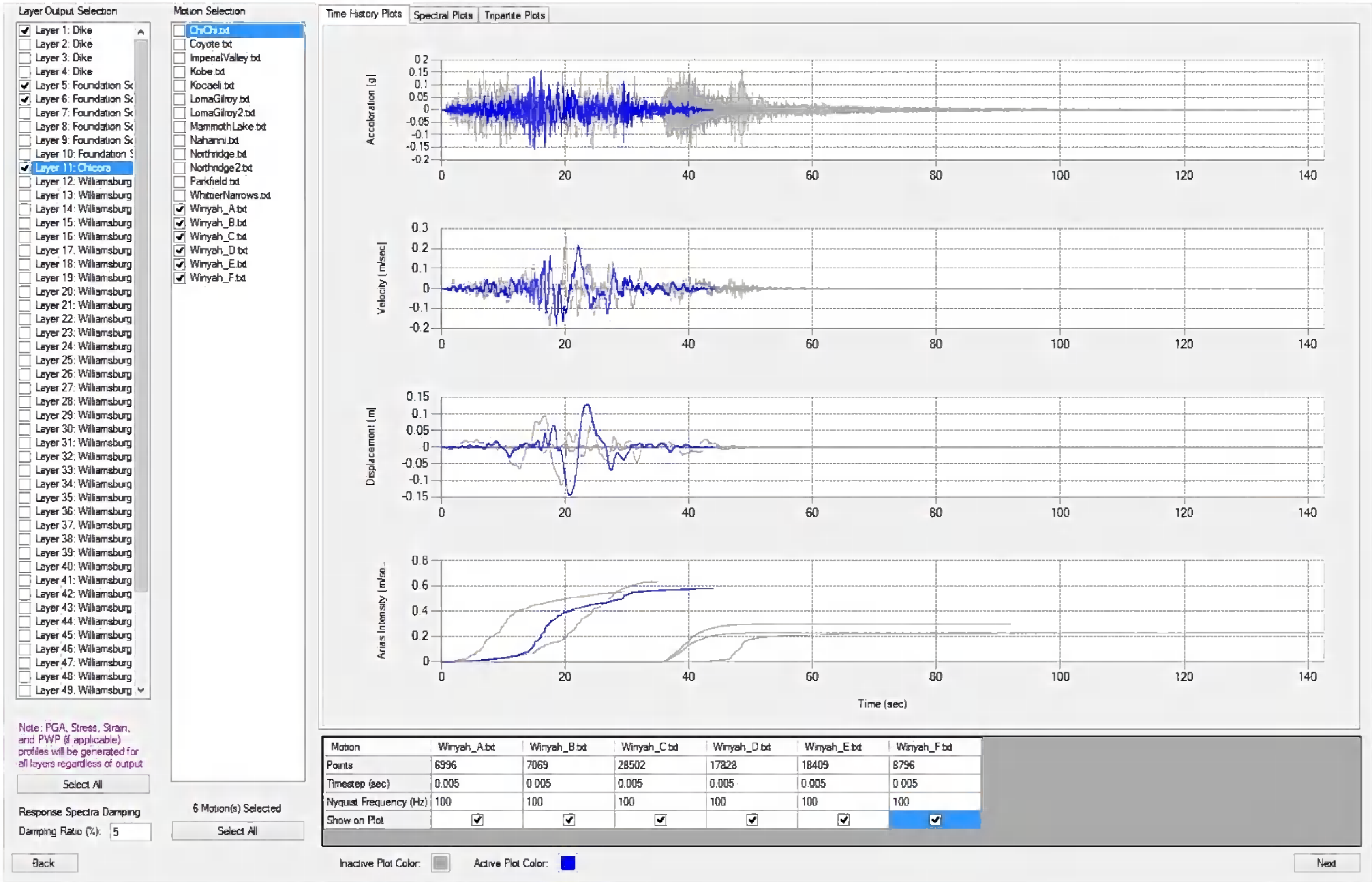
Layer 35
Layer 36
Layer 37
Layer 38
Layer 39
Layer 40
Layer 41
Layer 42
Layer 43
Layer 44
Layer 45
Layer 46
Layer 47
Layer 48
Layer 49
Layer 50
Layer 51
Layer 52
Layer 53
Layer 54
0
1000
2000
Shear Wave Velocity (ft/sec)
0
50
Max. Frequency (Hz)
0
2
Small Strain Damping (%)
0
5000
10000
Implied Shear Strength, τ (psf)
0
1
2
Normalized Implied Shear Str...
0
20
40
60
Implied Friction Angle (deg)
☐ Show Layers
Back
Next



Layer Output Selection

- ☑ Layer 1: Dike
- ☐ Layer 2: Dike
- ☐ Layer 3: Dike
- ☐ Layer 4: Dike
- ☑ Layer 5: Foundation Sc
- ☑ Layer 6: Foundation Sc
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- ☑ Layer 11: Chicora
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- ☐ 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	☑	☑	☑	☑	☑	☑

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ATTACHMENT 7

Liquefaction Potential Analysis

Geosyntec consultants

CALCULATION PACKAGE COVER SHEET

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

TITLE OF PACKAGE: **LIQUEFACTION POTENTIAL ANALYSIS: SOUTH ASH POND**

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

LIQUEFACTION POTENTIAL ANALYSIS: SOUTH ASH POND

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 the South Ash Pond at Winyah Generating Station (WGS or Site). This calculation package is Attachment 7 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: South Ash Pond* (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 safety factor assessment criteria presented in 40 Code of Federal Regulations (CFR) 257.73(e). Ground motions and resulting cyclic shear stresses for the design seismic event are presented in Attachment 6 titled "Seismic Hazard Evaluation and Site Response Analysis: South Ash Pond" (Site Response Package) to the Safety Factor Assessment Report. The potential of soils to liquefy was evaluated for soil borings and cone penetration test (CPT) soundings advanced through the South Ash Pond perimeter dikes based on geotechnical information collected during Geosyntec's 2013 and 2016 geotechnical subsurface investigations. Soil borings and soundings located at the perimeter dike toe will be analyzed during an evaluation of "Unstable Areas" in accordance with the CCR Rule. Details of these investigations are discussed in Attachment 5 titled "Subsurface Stratigraphy and Material Properties: South Ash Pond" (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.

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 at WGS to evaluate the susceptibility of fine-grained soils to cyclic softening. Of the four, fine-grained samples tested within the South Ash Pond area, none was found to be susceptible to cyclic softening under the criteria described in Bray and Sancio (2006).

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 SPT 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 and is normalized with the effective overburden stress. The CSR for a soil interval is calculated as follows:

$$\mathrm{CSR}_{\mathrm{M},\sigma'_{\mathrm{vo}}} = 0.65\,\frac{\tau_{\max}}{\sigma'_{\mathrm{vo}}} \tag{1}$$

where:

$\mathrm{CSR}_{\mathrm{M},\sigma'\mathrm{vo}}$ = Cyclic Stress Ratio due to an earthquake with magnitude, M;
$\tau_{\max}$ = 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:

$$\mathrm{Q} = \left(\frac{\mathrm{q_c}-\sigma_{\mathrm{vo}}}{\mathrm{P_a}}\right)\left(\frac{\mathrm{P_a}}{\sigma'_{\mathrm{vo}}}\right)^{\mathrm{n}} \tag{2}$$

and,

$$\mathrm{F} = \left(\frac{\mathrm{f_s}}{\mathrm{q_c}-\sigma_{\mathrm{vo}}}\right) \times 100\% \tag{3}$$

where:

$\mathrm{q_c}$ = 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:

$$I_c = \left[(3.47 - \log(Q))^2 + (\log(F) + 1.22)^2\right[^{0.5} \qquad (4)$$

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).

$$(N_1)_{60} = N_{meas}C_EC_BC_SC_RC_N \tag{6}$$

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:

$$\mathrm{CRR}_{M=7.5,\sigma'_{vo}=1\,\mathrm{atm}} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right) \qquad (9)$$

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:

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \times \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \qquad (10)$$

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:

$$\mathrm{CRR}_{M=7.5,\,\sigma'_{vo}=1\,\mathrm{atm}} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \qquad (11)$$

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:

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \qquad (12)$$

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 (GDM) 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 for soil borings and CPT soundings overseen by Geosyntec in 2013 and 2016. Computations were limited to soil borings and CPT soundings located through the dike centerline into the foundation soils immediately underlying the perimeter dikes.

Two representative soil profiles of shear wave velocity (V_s) were developed and presented in the Data Package (Attachment 5) from the dike crest to the Chicora stratum. These profiles 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 analyses for the South Ash Pond.

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 cyclic shear stress (τ_{max}) was computed for each ground motion and the maximum value at each depth was calculated 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.

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 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 SCDOT GDM (SCDOT, 2010) and the site-specific laboratory data (Attachment 5). The assigned unit weight is dependent 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 for the range of soil ages observed in the region presented by Leon et al. (2005), and are provided in Table 4 based on Equations 19a and 19b. A K_{DR} was selected from Table 4 and applied to soils in the vicinity of the South Ash Pond perimeter dikes that were evaluated to be of geologic and geotechnical ages older than Holocene age (i.e., foundation soils).

As shown in Figure 1, soils immediately surrounding South Ash Pond 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 1.3 was selected for Pleistocene-aged, foundation soils at WGS. The bottom of dike fill soils at each investigation point was estimated based the elevation of South Ash Pond toe drains as provided in design drawings (Lockwood-Greene, 1978). The dike fill soils were assumed to extend 1 ft below the toe drain elevation for each investigation point. Boring information and the top of foundation soil elevation (or the approximate 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 (percent 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 South Ash Pond footprint. Physical samples are not collected during CPT soundings. 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, as provided in Attachment 5. 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 South Ash Pond 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 subsurface investigation, Soil Consultants, Inc. (SCI), reported that the automatic hammer on the utilized drilling rig had an energy ratio of 89 percent, which was independently evaluated within six months of the investigation. During the 2016 subsurface investigation, Mid-Atlantic Drilling, Inc. (MAD) utilized a drill rig with an energy ratio of 77 percent.

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 against liquefaction triggering for the soil borings and CPT soundings and the approximate base of the perimeter dike structure which was developed from historical drawings (Lockwood-Greene, 1978) are shown in Figures 2 through 6. Figure 2 shows SPT-109, CPT-122, and CPT-123 which are located in the southwest corner of the South Ash Pond. 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 of this Attachment 7.

The liquefaction potential calculation results can be generally summarized as follows:

- The computed FS_{liq} typically exceeded 2.0 within dike fill and foundation soils immediately below the South Ash Pond perimeter dikes.

- Dike fill soils adjacent to CPT-129 and SPT-302 were computed with FS_{liq} ranging from 1.2 to 1.5 between elevations 20 and 30 ft NGVD29.

- A potentially liquefiable zone (i.e., $FS_{liq} < 1.0$) during the design earthquake was computed within dike fill soils based on in-situ measurements from CPT-205. A 1-ft thick potentially liquefiable zone spanning from 25.8 ft to 26.8 ft NGVD29 was computed from CPT-205.

- Based on in-situ measurements from CPT-122, a few thin seams (0.1- to 0.2-ft thick) of potential liquefiable soil during the design earthquake were computed between -18.2 and -19.5 ft NGVD29, which is within foundation soils immediately above the Chicora stratum. Index testing of soil samples collected within this depth interval was not performed during site investigations; thus, the fines content of an overlying sand layer (11 percent) was conservatively assigned to this stratum. It is anticipated that these seams contain a larger percentage fines and greater resistance than conservatively modeled within this calculation package.

- Foundation soils were typically not found to be liquefiable (except as indicated above) beneath the South Ash Pond perimeter dikes. Isolated zones where FS_{liq} between 1.1 and 1.5 were computed between 0.0 and 10.0 ft NGVD29 in the northwest corner of the South Ash Pond.

- 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-302 and CPT-205 at similar elevations).

CONCLUSIONS

Based the liquefaction potential computations presented within this calculation package, a potentially liquefiable zone was computed within dike fill soils (i.e., native soils recompacted to form impounding perimeter dikes) located in the northwest corner of the South Ash Pond. Other than a few isolated seams, soils located within the foundation materials underlying the South Ash Pond perimeter dikes were not found to be liquefiable during the design earthquake. Soil borings and CPT soundings advanced at the downstream toe of the South Ash Pond perimeter dikes were not evaluated within this calculation package and will be included during an evaluation of "Unstable Areas" for the South Ash Ponds. Since liquefiable zones were identified within the perimeter dikes, additional post-liquefaction stability is warranted for the South Ash Pond in the vicinity of CPT-205.

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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): $C_S = 1.1 \quad \text{for} \quad (N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100} \quad \text{for} \quad 10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3 \quad \text{for} \quad (N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Written by: **J. McNash** Date: **10/11/2016** Reviewed by: **J. Colley/M. Zhu/G. Rix** Date: **10/11/2016**

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

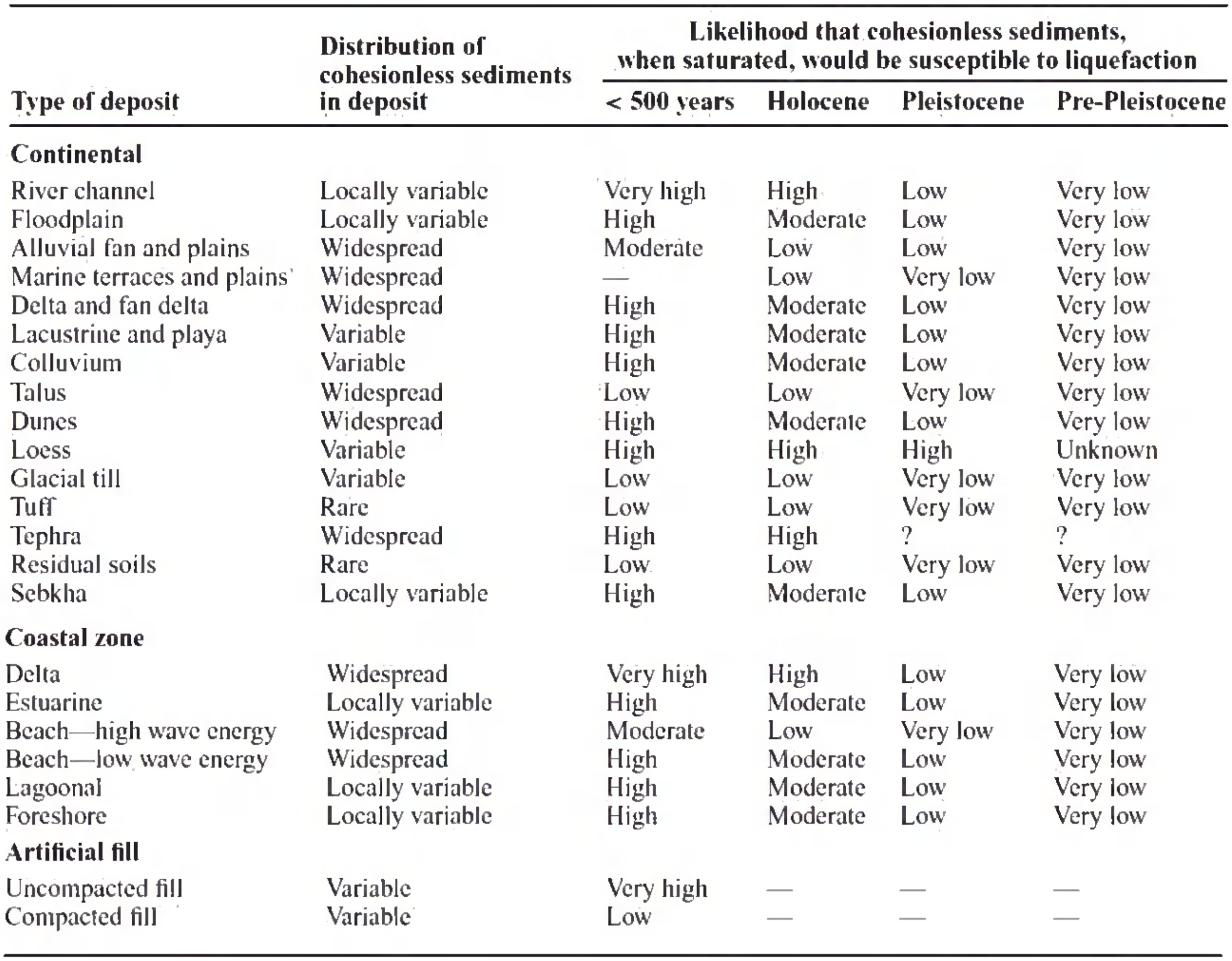
Table 2. Summary of Representative Profiles for the South Ash Pond Dike Centerline

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	41.0	2.5	35.7
7.5	96.1	7.5	79.9
12.5	123.8	12.5	109.9
16.5	146.2	16.5	134.6
19.5	171.3	20.5	160.2
23.5	192.4	25.5	182.3
28.5	203.8	30.5	195.3
33.5	213.1	35.5	204.8
38.0	271.9	40.5	214.0
42.0	306.2	45.5	223.2
46.0	330.8	50.5	282.6
50.5	364.1	58.0	383.3
58.0	457.2	68.0	487.5
68.0	571.7	78.0	555.2
78.0	682.4	88.0	679.3
88.0	789.2	98.0	758.0
98.0	928.9	-	-

Notes:

1. Profile 1 refers to the western perimeter dikes of the South Ash Pond where a thicker zone of clay was observed; while Profile 2 refers to the eastern perimeter dikes where significant zones of clay were not encountered. Development of each profile is discussed within 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.

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 Soil Borings and Soundings Analyzed for Liquefaction Potential

Boring ID	**Relative Location**	**Northing**	**Easting**	**Elevation**	**Dike Base Elevation**	**GWT**	**GWT Source**	**FC Basis**	**τ_{max} Profile**
-	-	**ft**	**ft**	**ft NGVD29**	**ft NGVD29**	**ft NGVD29**	-	-	-
CPT-122	Dike Centerline	546598.0511	2499294.0984	38.82	17.6	30.4	Diss. Test	SPT-303/303A	Profile 1
CPT-123	Dike Centerline	546324.2132	2500128.8799	37.84	20.0	29.8	u_2 Signature	SPT-303/303A	Profile 2
CPT-124	Dike Centerline	546100.3030	2501583.1059	38.39	20.5	24.9	u_2 Signature	SPT-110	Profile 2
CPT-125	Downstream Toe	546044.6532	2501573.1307	23.51	20.5	20.5	u_2 Signature	N/A	Profile 2
CPT-126	Dike Centerline	546463.4354	2502551.6060	38.63	21.1	21.6	u_2 Signature	SPT-110	Profile 2
CPT-128	Dike Centerline	547041.4825	2501352.8951	38.42	22.3	25.9	u_2 Signature	SPT-111	Profile 2
CPT-129	Dike Centerline	547429.4842	2498965.8353	38.88	20.0	25.8	Diss. Test	SPT-302	Profile 1
CPT-130	Downstream Toe	547534.4639	2498976.2141	21.66	19.7	18.2	u_2 Signature	N/A	N/A
CPT-130A	Downstream Toe	547498.2356	2498839.4657	23.08	19.7	17.9	Diss. Test	N/A	N/A
CPT-131	Pond Interior	546940.5550	2499072.2961	42.25	N/A	34.6	Diss. Test	N/A	N/A
CPT-132	Pond Interior	547033.8151	2500786.7646	38.58	N/A	32.6	u_2 Signature	N/A	N/A
CPT-133	Pond Interior	546303.4801	2501507.2779	38.58	N/A	31.0	Diss. Test	N/A	N/A
CPT-204	Dike Centerline	547294.8128	2499937.8582	38.42	20.9	30.4	u_2 Signature	SPT-112	Profile 2
CPT-205	Dike Centerline	547422.5833	2499106.9970	38.88	20.3	29.2	Diss. Test	SPT-302	Profile 1
CPT-206	Dike Centerline	547384.8761	2498800.1583	38.88	19.7	29.2	u_2 Signature	SPT-302	Profile 1
CPT-207	Downstream Toe	547241.3967	2498631.3734	23.08	17.0	17.7	Diss. Test	N/A	N/A
CPT-208	Dike Centerline	547121.6171	2498742.0069	37.39	17.0	27.4	u_2 Signature	SPT-109	Profile 1
SPT-109	Dike Centerline	546898.7338	2498876.8972	37.39	16.5	32.3	Borehole	SPT-109	Profile 1
SPT-110	Dike Centerline	546029.9130	2502059.5414	38.72	21.0	27.7	Borehole	SPT-110	Profile 2
SPT-111	Dike Centerline	546949.6779	2502267.1691	39.41	21.7	29.3	Borehole	SPT-111	Profile 2
SPT-112	Dike Centerline	547207.9843	2500600.5880	37.66	21.5	32.4	Borehole	SPT-112	Profile 2
SPT-113	Pond Interior	546933.0676	2499058.1034	42.27	N/A	33.9	Borehole	N/A	N/A
SPT-302	Dike Centerline	547422.0662	2498943.1317	38.88	20.0	29.2	Borehole	SPT-302	Profile 1
SPT-303	Dike Centerline	546607.9500	2499254.4342	38.82	17.6	33.4	Borehole	SPT-303	Profile 1
SPT-303A	Dike Centerline	546618.8098	2499250.9024	38.82	17.6	33.4	Borehole	N/A	N/A

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 the design elevation of the nearest toe drain (Lockwood-Greene, 1978) minus 1 ft.
3. SPT-113 and CPT-131 through CPT-133 were performed in the interior of the South Ash Pond and were terminated at the bottom of the impoundment. Liquefaction potential was not computed for the CCR materials.
4. 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).
5. The GWT elevation for SPT-302, which was advanced using mud rotary wash techniques, was selected as 9.64 ft bgs (based on CPT-205) as the bentonite slurry prevented the water within the borehole from reaching an equilibrium condition within 24 hours.

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

FIGURES

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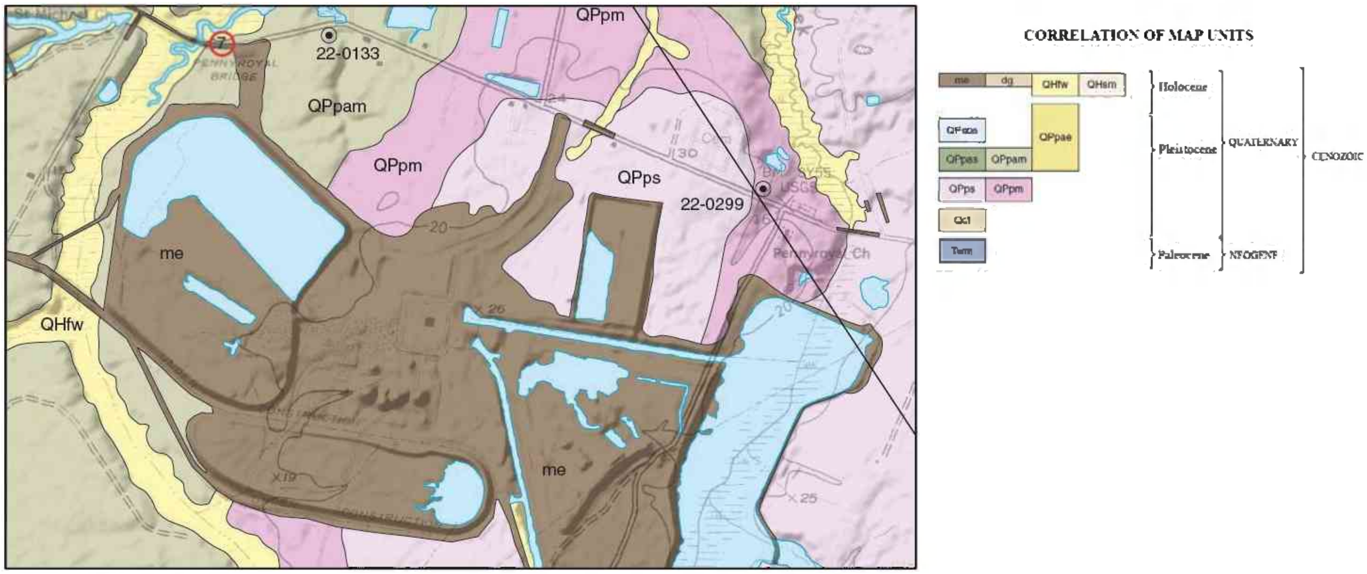

QPpam **Estuarine deposits or marine deposits or both (Pleistocene)** – In its lower part, quartz and phosphatic sand, medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse grained with trace amounts of very fine grained heavy minerals. Lower part 10 to 20 feet thick. 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 the South Ash Pond
(Map taken from SC Department of Natural Resources: Geological Survey, 2012)

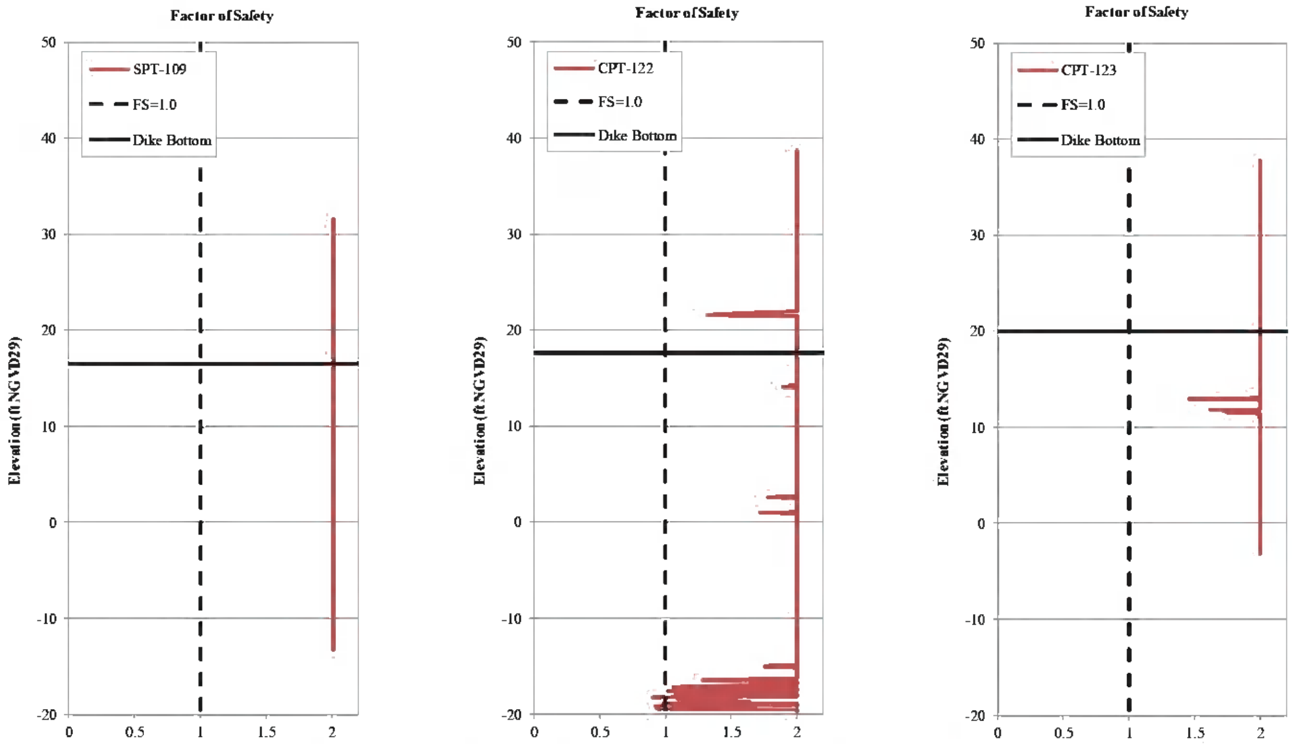

Figure 2. Liquefaction Results for Dike and Foundation Soils for SPT-109, CPT-122, and CPT-123

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

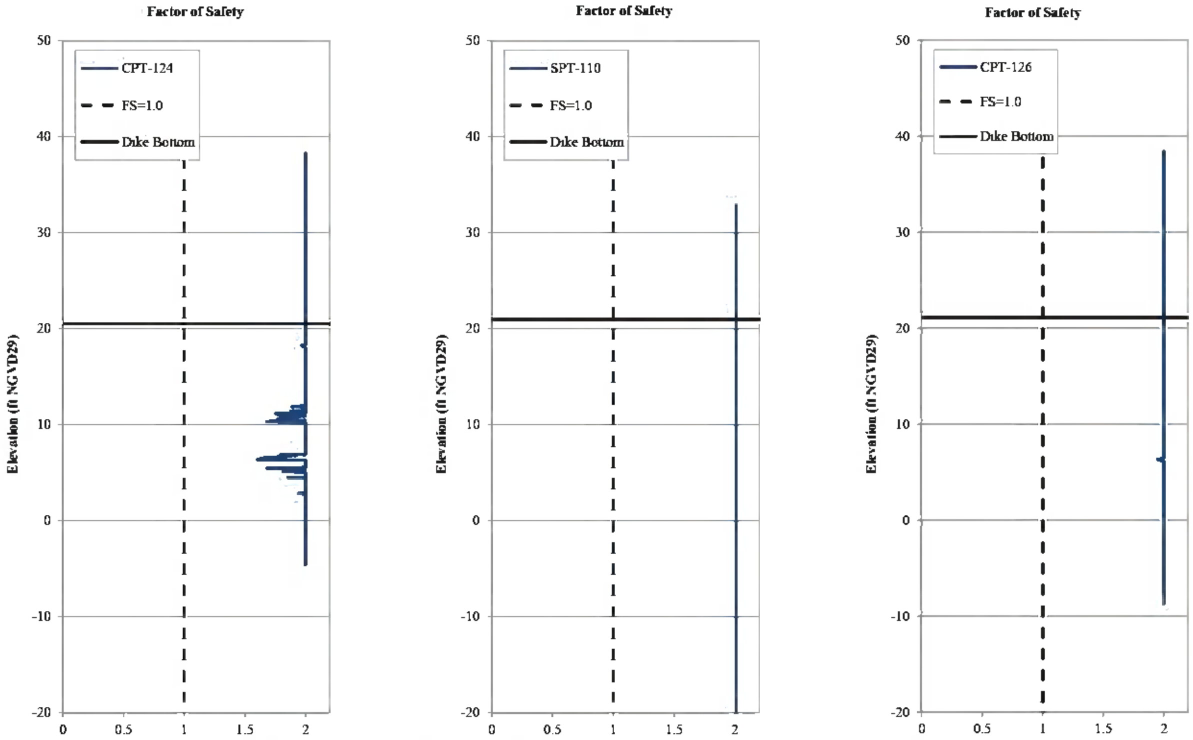

Figure 3. Liquefaction Results for Dike and Foundation Soils for CPT-124, SPT-110, and CPT-126

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

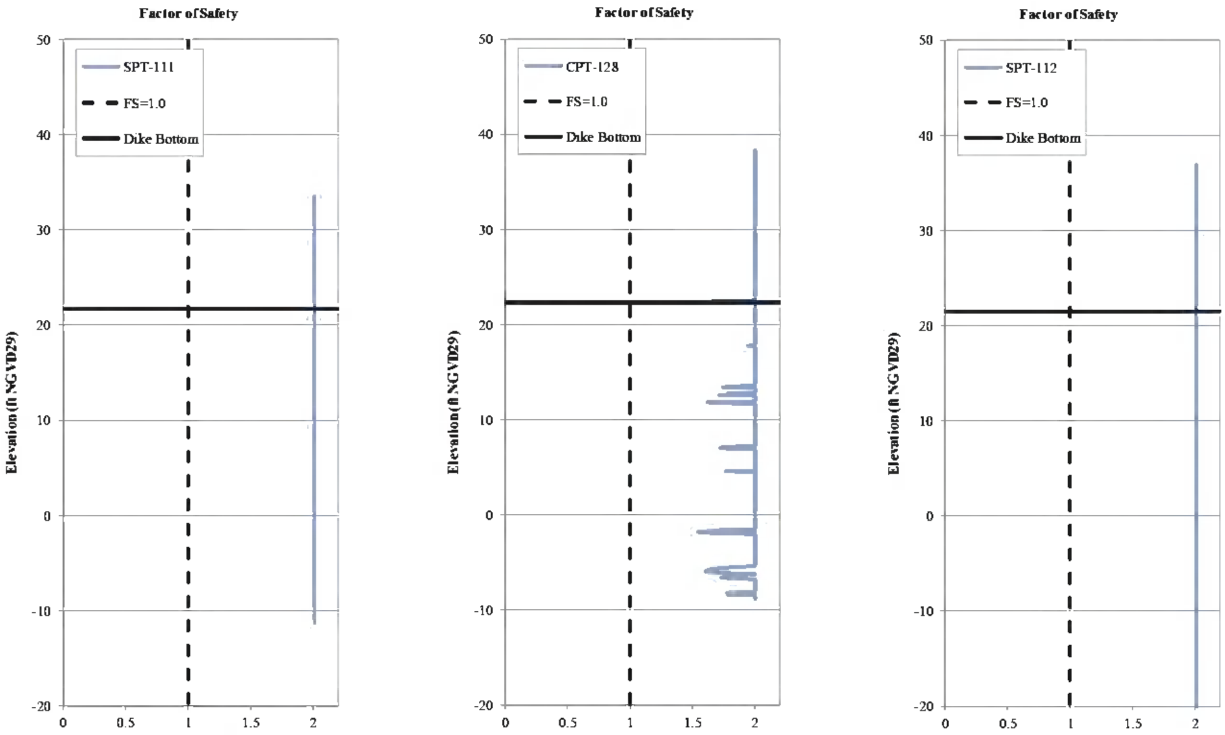

Figure 4. Liquefaction Results for Dike and Foundation Soils for SPT-111, CPT-128, and SPT-112

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

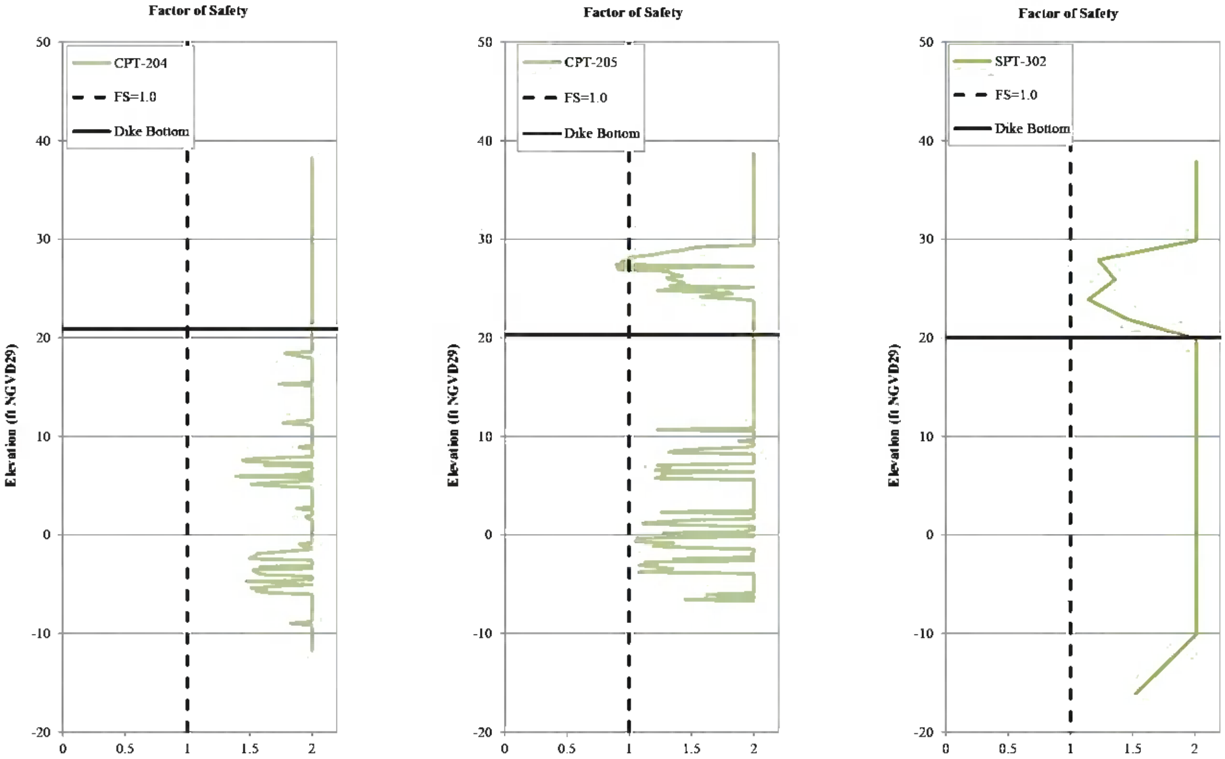

Figure 5. Liquefaction Results for Dike and Foundation Soils for CPT-204, CPT-205, and SPT-302

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

Written by: **J. McNash** Date: **10/11/2016** Reviewed by: **J. Colley/M. Zhu/G. Rix** Date: **10/11/2016**

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

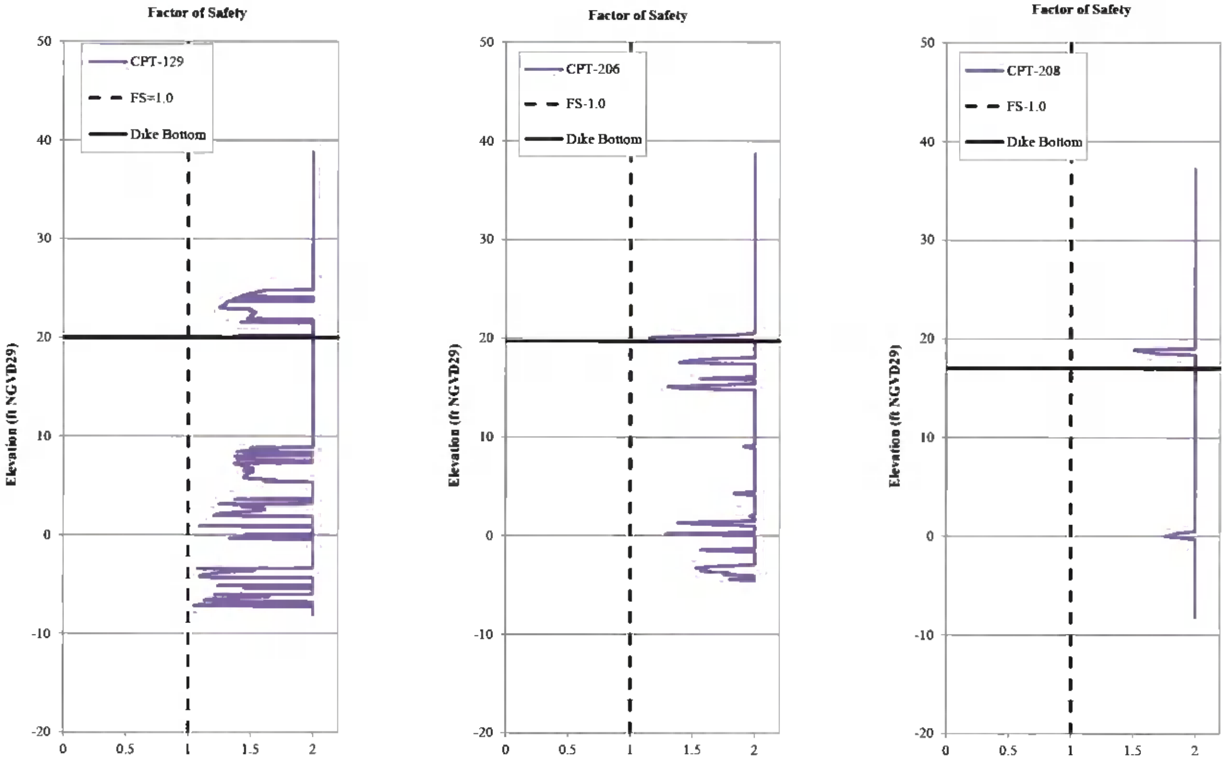

Figure 6. Liquefaction Results for Dike and Foundation Soils CPT-129, CPT-206, and CPT-208

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

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

Appendix 1

MathCAD® Example Calculation

CPT - based Liquefaction Analysis

$\text{BoringID} := \text{"CPT\_122"}$

Site Parameters:

Age Correction Factor of Pleistocene Soils: $K_{dr} := 1.3$

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $\text{Prof} := \text{"Profile1"}$

$$\text{CyclicStress} := \left|\begin{array}{l} \text{READEXCEL("SAP\_Profile\_1.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile1"} \\ \text{READEXCEL("SAP\_Profile\_2.xlsx")} \ \text{ if } \ \text{Prof} = \text{"Profile2"} \end{array}\right.$$

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:

$\text{Full} := \text{READEXCEL}(\text{concat}(\text{BoringID}, \text{".xlsx"}))$

$\text{Data} := \text{submatrix}(\text{Full}, 2, \text{rows}(\text{Full}) - 1, 0, \text{cols}(\text{Full}) - 1)$

$\text{depth} := \text{Data}^{\langle 0 \rangle} \cdot \text{ft} \qquad \text{qc} := \text{Data}^{\langle 1 \rangle} \cdot \text{tsf} \qquad f_s := \text{Data}^{\langle 2 \rangle} \text{tsf} \qquad u_2 := \text{Data}^{\langle 3 \rangle} \text{tsf} \qquad \text{Fines} := \text{Data}^{\langle 4 \rangle}$

Simple counter used in the Algorithm: $i := 0 .. \text{rows}(\text{Data}) - 1$

Tip net area ratio: $a := 1$ Correction applied when converting Hogentogler Data (.cpt) to Excel (.xls) format.

Boring Information Inputs:

Boring Elevation: $\text{Elevation} := 38.82\text{ft}$ NGVD 29

Groundwater Depth at Time of Boring (TOB): $\text{GWT}_b := 8.46\text{ft}$ bgs

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $\text{Elev}_h := 17.6\text{ft}$ 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:

$$\text{class}_{1983} := \left| \begin{array}{l} \text{for } \ i \in 0 .. \text{rows}(qt) - 1 \\ \quad \left| \begin{array}{l} \text{class}_i \leftarrow 5 \\ \text{class}_i \leftarrow 4 \ \text{ if } \ \dfrac{qt_i}{100\cdot kPa} \geq s04\left(Rf_i\right) \\ \text{class}_i \leftarrow 3 \ \text{ if } \ \dfrac{qt_i}{100\cdot kPa} \geq s03\left(Rf_i\right) \\ \text{class}_i \leftarrow 2 \ \text{ if } \ \dfrac{qt_i}{100\cdot kPa} \geq s02\left(Rf_i\right) \\ \text{class}_i \leftarrow 1 \ \text{ if } \ \dfrac{qt_i}{100\cdot kPa} \geq s01\left(Rf_i\right) \end{array} \right. \\ \text{class} \end{array} \right.$$

$$\gamma 1 := \left| \begin{array}{l} \text{for } \ i \in 0 .. \text{rows}(qt) - 1 \\ \quad \text{for } \ m \in 1 .. 5 \\ \quad\quad \gamma 1_i \leftarrow \gamma_m \ \text{ if } \ \text{class}_{1983_i} = m \\ \gamma 1 \end{array} \right.$$

▲ 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:

$$u_{0_i} := \left| \begin{array}{l} \left(\text{depth}_i - GWT_b\right)\cdot\gamma_{water} \ \text{ if } \ \text{depth}_i > GWT_b \\ 0 \ \text{ otherwise} \end{array} \right.$$

Calculating Total and Effective Overburden Pressure

$$\sigma_{v0_i} := \left| \begin{array}{l} \left(\text{depth}_i - \text{depth}_{i-1}\right)\cdot\left(\dfrac{\gamma 1_i + \gamma 1_{i-1}}{2}\right) + \sigma_{v0_{i-1}} \ \text{ if } \ i > 0 \\ \text{depth}_i\cdot\gamma 1_0 \ \text{ otherwise} \end{array} \right. \qquad \sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}}\cdot 100$$

▲ 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:

```
$class_{2010} :=$ for $i \in 0..\text{rows}(Q_t) - 1$
                     $class_i \leftarrow 2$
                     $class_i \leftarrow 3$  if $2.95 < I_{c_i} \le 3.6$
                     $class_i \leftarrow 4$  if $2.60 < I_{c_i} \le 2.95$
                     $class_i \leftarrow 5$  if $2.05 < I_{c_i} \le 2.60$
                     $class_i \leftarrow 6$  if $1.31 < I_{c_i} \le 2.05$
                     $class_i \leftarrow 7$  if $I_{c_i} \le 1.31$
                 class
```

Assigning unit weight based on soil classification:

```
$\gamma_{fin} :=$ for $i \in 0..\text{rows}(Q_t) - 1$
                    for $m \in 1..9$
                        $\gamma2_i \leftarrow \gamma_m$  if $class_{2010_i} = m$
                $\gamma2$
```

▲ Refined soil classification and assigning unit weights

▼ Applying Robertson (2010) values for remaining calculations:

$\gamma := \gamma_{fin}$ $\quad\quad$ $class := class_{2010}$

Final Static Pore Pressure Calculation for CPT interpretation:

$$u_{0_i} := \begin{vmatrix} \left(depth_i - GWT_b\right)\cdot\gamma_{water} \ \text{ if } \ depth_i > GWT_b \\ 0 \ \text{ otherwise} \end{vmatrix}$$

Total and Effective Overburden Pressure Final Calculation for CPT interpretaiton:

$$\sigma_{v0_i} := \begin{vmatrix} \left(depth_i - depth_{i-1}\right)\cdot\left(\frac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} \ \text{ if } \ i > 0 \\ depth_0\cdot\gamma_0 \ \text{ otherwise} \end{vmatrix} \qquad \sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}}\cdot 100$$

$$Q_i := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}}$$

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 $i \in 0..\mathrm{rows}(qt) - 1$
        $C_{N_i} \leftarrow 1.7$
    for $i \in 0..\mathrm{rows}(qt) - 1$
        while $c < 500$
            $q_{c1_i} \leftarrow C_{N_i} \cdot qt_i$
            $q_{c1N_i} \leftarrow \frac{q_{c1_i}}{1\mathrm{atm}}$
            $C_{N_i} \leftarrow \min\left[1.7, \left(\frac{1\mathrm{atm}}{\sigma_{v0eff_i}}\right)^{1.338-0.249\cdot\left(\max\left(21,\min\left(q_{c1N_i},254\right)\right)\right)^{0.264}}\right]$
            $c \leftarrow c + 1$
        $c \leftarrow 0$
    $\left(\frac{q_{c1}}{psf} \quad q_{c1N}\right)$
```

$$q_{c1} := \left(q_{c1\_it}{}^{\langle 0\rangle}\right)_0 psf \qquad q_{c1N} := \left(q_{c1\_it}{}^{\langle 1\rangle}\right)_0$$

▲ Overburden corrected tip resistance calculations

▼ Compute CRR with Overburden, MSF, and Kdr Corrections

Compute the CRR (M_w=7.5, 1 atm) based on the CPT values:

Cyclic Resistance Ratio (CRR):

$i := 0..\mathrm{rows}(qc) - 1$

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: $q_{c1Ncs_i} := q_{c1N_i} + \Delta q_{c1N_i}$

$$CRR_i := \left|\begin{array}{l} \exp\left[\dfrac{q_{c1Ncs_i}}{540} + \left(\dfrac{q_{c1Ncs_i}}{67}\right)^2 - \left(\dfrac{q_{c1Ncs_i}}{80}\right)^3 + \left(\dfrac{q_{c1Ncs_i}}{114}\right)^4 - 3\right] \ \text{if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} < 211 \\ 2.0 \ \text{ if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} > 211 \\ 2.0 \ \text{ otherwise} \end{array}\right.$$

Overburden Correction Factor (Kσ) for Sands:

$$C_{\sigma_i} := \min\left[\frac{1}{37.3 - 8.27\cdot\left(\min\left(q_{c1N_i}, 211\right)\right)^{0.264}}, 0.3\right]$$

$$K_{\sigma_i} := \left|\begin{array}{l} \min\left(1 - C_{\sigma_i}\cdot\ln\left(\dfrac{\sigma_{v0eff_i}}{1\text{tsf}}\right), 1.1\right) \ \text{ if } I_{c_i} \le 2.60 \\ 1.0 \ \text{ otherwise} \end{array}\right.$$

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

$$FS_i := \left|\begin{array}{l} 2.00 \ \text{ if } \ \text{depth}_i < GWT_b \\ \min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.00\right) \ \text{ otherwise} \end{array}\right.$$

▲ 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

$\text{BoringID} := \text{"SPT-109"}$

Site Parameters:

Age Correction Factor: $K_{dr} := 1.3$ (Geosyntec, 2013)

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $\text{Prof} := \text{"Profile1"}$

$$\text{CyclicStress} := \left| \begin{array}{l} \text{READEXCEL("SAP\_Profile\_1.xlsx")} \ \ \text{if Prof} = \text{"Profile1"} \\ \text{READEXCEL("SAP\_Profile\_2.xlsx")} \ \ \text{if Prof} = \text{"Profile2"} \end{array} \right.$$

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:

$$\text{Full} := \text{READEXCEL(concat(BoringID, ".xlsx"))}$$

$$\text{Data} := \text{submatrix}(\text{Full}, 2, \text{rows}(\text{Full}) - 1, 0, \text{cols}(\text{Full}) - 1)$$

$\text{depth} := \text{Data}^{\langle 0 \rangle} \cdot \text{ft}$ $\quad N_{blows} := \text{Data}^{\langle 1 \rangle}$ $\quad \text{Class} := \text{Data}^{\langle 2 \rangle}$ $\quad \text{Fines} := \text{Data}^{\langle 3 \rangle}$ $\quad \text{USCS} := \text{Data}^{\langle 4 \rangle}$

Boring Information:

Boring Elevation: $\text{Elevation} := 37.39\text{ft}$ NGVD29

Groundwater Depth: $\text{GWT} := 5.1\text{ft}$ bgs

Boring Diameter: $\text{Diameter} := 4$ inches

Holocene Elevation: $\text{Elev}_h := 16.50\text{ft}$ NGVD29

Energy Calibration: $\text{ER} := 88$ % (SCI, 2014)

Sampling Method: $C_S := 1.0$

$\text{RodDepth} := \text{depth} + 5\text{ft}$ (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 ≤ 4.0
       | 1.05  if  4.0 < Diameter < 6.0
       | 1.15  otherwise
```

```
C_R := | for  i ∈ 0 .. rows(depth) − 1
       |     | rod_i ← 0.75  if  RodDepth_i ≤ 13ft
       |     | rod_i ← 0.85  if  13ft < RodDepth_i ≤ 20ft
       |     | rod_i ← 0.95  if  20ft < RodDepth_i ≤ 33ft
       |     | rod_i ← 1  otherwise
       | rod
```

Compute N_{60}:

```
N_60 := | for  i ∈ 0 .. rows(depth) − 1
        |     x_i ← C_B·C_E·C_S·N_blows_i·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	$\gamma_1 := 100\text{pcf}$
2. Loose Sands (N_{blows} <10)	$\gamma_2 := 105\text{pcf}$
3. Medium Dense Sands (10<N_{blows} <30)	$\gamma_3 := 115\text{pcf}$
4. Dense Sands	$\gamma_4 := 120\text{pcf}$
5. Soft Clays	$\gamma_5 := 100\text{pcf}$
6. Chicora Member	$\gamma_6 := 130\text{pcf}$
7. Williamsburg Formation Clay	$\gamma_7 := 105\text{pcf}$

Relate depth to elevation to screen unit weights for Williamsburg Formation Clay

$\text{Elev} := (\text{Elevation} - \text{depth})$ $\quad$ $\text{WMElev} := -8\text{ft}$ $\quad$ (Approx. Top of Williamsburg Formation Clay)

```
$\text{Class}_2 :=$ for $i \in 0 .. \text{rows}(\text{depth}) - 1$
        $y_i \leftarrow 7$
        $y_i \leftarrow 1$ if $\text{Class}_i = \text{"SILT"}$
        $y_i \leftarrow 2$ if $\text{Class}_i = \text{"SAND"} \wedge N_{\text{blows}_i} \le 10$
        $y_i \leftarrow 3$ if $\text{Class}_i = \text{"SAND"} \wedge 10 < N_{\text{blows}_i} < 30$
        $y_i \leftarrow 4$ if $\text{Class}_i = \text{"SAND"} \wedge 30 < N_{\text{blows}_i}$
        $y_i \leftarrow 5$ if $\text{Class}_i = \text{"CLAY"} \wedge \text{Elev}_i > \text{WMElev}$
        $y_i \leftarrow 6$ if $\text{Class}_i = \text{"CHICORA"}$
    $y$
```

Assign unit weight based on soil classification:

```
$\gamma_{\text{fin}} :=$ for $i \in 0 .. \text{rows}(\text{depth}) - 1$
        for $m \in 1 .. 7$
            $\gamma 2_i \leftarrow \gamma_m$ if $\text{Class}_{2_i} = m$
    $\gamma 2$
```

$\gamma := \gamma_{\text{fin}} \qquad \gamma_{\text{water}} := 62.4\text{pcf}$

$i := 0 .. \text{rows}(\text{depth}) - 1$

Final Static Pore Pressure Calculation:

$$u_{0_i} := \begin{cases} \left(\text{depth}_i - \text{GWT}\right) \cdot \gamma_{\text{water}} & \text{if } \text{depth}_i > \text{GWT} \\ 0 & \text{otherwise} \end{cases}$$

Total and Effective Overburden Pressure Final Calculation:

$$\sigma_{v0_i} := \begin{cases} \left(\text{depth}_i - \text{depth}_{i-1}\right) \cdot \left(\frac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} & \text{if } i > 0 \\ \text{depth}_0 \cdot \gamma_0 & \text{otherwise} \end{cases} \qquad \sigma_{v0\text{eff}} := \sigma_{v0} - u_0$$

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]:

$$MSF_i := \min(1.80, 6.9\cdot\exp(-0.25\cdot M) - 0.058)$$

$$CRR3_i := CRR2_i\cdot MSF_i$$

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

$$FS_i := \left| \begin{array}{ll} 2.01 \ \text{if Class}_i = \text{"CHICORA"} & \\ 2.01 \ \text{if Class}_i = \text{"CLAY"} & \\ 2.01 \ \text{if depth}_i < GWT & \\ \min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.01\right) & \text{otherwise} \end{array} \right.$$

-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" , "-" , "-" , "-")

$$\text{Export} := \text{augment}\left(\frac{\text{depth}}{\text{ft}}, \frac{\text{Elev}}{\text{ft}}, N_{160}, \text{Class}, FS\right)$$

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: SOUTH ASH POND**

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SAFETY FACTOR ASSESSMENT: SOUTH ASH POND

INTRODUCTION

This calculation package was prepared as Attachment 8 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: South Ash Pond* (Safety Factor Assessment Report) and presents the slope stability analyses for the South Ash Pond perimeter dikes at Winyah Generating Station (WGS). The South Ash Pond is a 76-acre surface impoundment that 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, the South Ash Pond 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 South Ash Pond perimeter dikes achieve the safety factor (also referred to as "factor of safety") criteria described within §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."*

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., circular slip surface mode) and non-rotational (i.e., 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, subsurface soil stratigraphy, phreatic surface elevation, external loading conditions, and 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: South Ash Pond* (Site Response Package) of the 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 an allowable displacement (u) in which the perimeter dikes are considered stable (from Figure 7 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 ≥ 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 (Hynes-Griffin and Franklin, 1984).

Liquefaction Slope Stability

Liquefaction stability was assessed using a the static slope stability approach outlined above, but applying residual shear strengths of liquefied sands during the design earthquake as computed in Attachment 7: *Liquefaction Potential Analysis: South Ash Pond* (Liquefaction Package) of the Safety Factor Assessment Report.

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 South Ash Pond perimeter dikes are approximately 24 feet (ft) in height, with a crest elevation of approximately 38.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) and toe elevation of approximately 24.0 ft NGVD29. The upstream and downstream side slopes range from 3 horizontal to 1 vertical (3H:1V) in the east to 4H:1V in the west; the dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012). To the north, east, and south of the perimeter dikes, a shallow drainage swale has been excavated inside the railroad loop and drains to the sump located to the west of the South Ash Pond.

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 South Ash Pond perimeter dikes (i.e. north, south, east, and west). The external geometry of each cross section was based on a topographic survey prepared by Thomas and Hutton (2012). Topographic contours were modeled as a triangular-irregular-network (TIN) surface within the computer program AutoCAD®. 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 is 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 and 2016 subsurface investigations. 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. Cross Section A also includes riprap buttress material placed against the downgradient dike slope and across the adjacent shallow drainage swale. Further discussion on the development of subsurface conditions can be found in Attachment 5: *Subsurface Stratigraphy and Material Properties: South Ash Pond* (Data Package) of the Safety Factor Assessment Report.

Water Levels

The CCR Rule requires the evaluation of safety factors considering static and seismic slope stability analyses under long-term "Maximum Normal Storage Pool" conditions and static and liquefaction slope stability analyses under short-term "Maximum Surcharge Pool" conditions. As described within the Attachment 1: *Hydrologic and Hydraulic Analysis: South Ash Pond* (H&H Package) of the Safety Factor Assessment Report, the surface water level in the South Ash Pond is maintained at an elevation of 28.73 ft NGVD29 by a concrete riser structure with a top stop log (Thomas and Hutton, 2016). The riser structure discharges eastward through a reinforced concrete pipe into the Discharge Canal (Lockwood Greene, 1978). An operating level of 28.73 ft NGVD29 was selected as the "Maximum Normal Storage Pool" for the South Ash Pond. Because the South Ash Pond has been classified as a "Low Hazard Potential" surface impoundment (Geosyntec, 2016), the 100-yr rainfall event with a rainfall duration of 72 hours was selected as the Inflow Design Flood (IDF), as required by §257.73(d)(1)(B). The maximum surface water elevation within the South Ash Pond during and after the IDF was computed in the H&H Package as 31.8 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 u_2 signatures and sounding dissipation tests, 24-hour depth to water measurements in soil borings, and observed dike toe drain performance in 2013. In both the "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions, the phreatic surface through the South Ash Pond perimeter dikes was assumed to reach steady-state conditions.

Final Cross Section Geometry

The final geometric models 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 the South Ash Pond. Table 1 provides a summary of the material properties selected for each evaluated cross section as a part of this safety factor assessment. The interpretation and selection of properties for each cross section are shown on Figures 7 through 11 for Cross Sections A through E, respectively.

Drained shear strength parameters for cross section-specific dike fill and sandy foundation soil were predominantly developed from in-situ measurements (i.e., SPT N-values, etc.) for each section.

It was assumed that seismic waves generated during a potential seismic event would load dike fill and foundation soils rapidly enough to develop elevated pore pressures and induce an undrained loading condition within the clayey soils, with cyclic degradation of strength for all materials. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the selected shear strength values were reduced by 20 percent for the seismic safety factor case to account for potential cyclic degradation during an earthquake at the Site.

Seismic Loading and Allowable Displacement

An evaluation of the seismic hazard for WGS and the site response analysis for the South Ash Pond perimeter dikes is presented in the Site Response Package of the Safety Factor Assessment Report. Within that package, six ground motions for WGS were evaluated for two representative dike soil profiles for the South Ash Pond, and 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 the South Ash Pond indicated that the typical critical depth of the anticipated slip surface is located up to 30 ft below the dike crest. Thus, the maximum MHEA at the anticipated critical slip surface was selected assuming the critical slip surface is located at 10, 27, 17.5, 16.5, and 2.5 ft below the dike crest for Cross Sections A through E, respectively. The maximum MHEA from the six ground motions at the critical slip surface depth was selected to compute the k_h during pseudo-static analyses. The MHEA for each ground motion and representative soil profile to an approximate depth of 100 ft bgs is provided in Table 2. MHEA values of 0.092g, 0.062g, 0.066g, 0.066 g, and 0.114g were 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 (in.) (30.5 centimeters (cm)) was selected for the South Ash Pond 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 12. Thus, k_h values of 0.046 and 0.031 were computed based on Profile 1 for Cross Sections A and B, respectively, and k_h values of 0.033, 0.033, and 0.057 were computed based on Profile 2 for Cross Sections C, D, and E, respectively.

Residual Shear Strength

The liquefaction potential analysis for the South Ash Pond perimeter dikes is presented in Attachment 7: *Liquefaction Potential Analysis: South Ash Pond* (Liquefaction Package) of this Safety Factor Assessment Report. A thin liquefiable zone spanning from 25.8 to 26.8 ft NGVD29 was computed within dike fill soils at CPT-205 near Cross Section A. Residual shear strength was computed by the correlation presented in Idriss and Boulanger (2008), and is reiterated as follows:

$$\frac{S_r}{\sigma'_{vo}} = \exp\left[\frac{q_{c,1,N,CS}}{24.5} - \left(\frac{q_{c,1,N,CS}}{61.7}\right)^2 + \left(\frac{q_{c,1,N,CS}}{106}\right)^3 - 4.42\right[\qquad (1)$$

where:

S_r = residual shear strength (psf);
σ'_{vo} = vertical effective stress (psf); and
$q_{c,1,N,CS}$= $q_{c,1,N}$ corrected for fines content (+ $\Delta q_{c,1,N\text{-}rl}$).

The correction for tip resistance ($q_{c,1,N}$) is described in the Data Package (Attachment 5); but a separate correction for the influence of fines content is applied when computing the residual shear strength of liquefied soils ($\Delta q_{c,1,N\text{-}rl}$). Table 3 provides the $\Delta q_{c,1,N\text{-}rl}$ for varying levels of fines content. Figure 13 presents the computed residual shear strengths for the computed liquefiable zone from CPT-205. A S_r of 30 psf was selected for the liquefied soil zone modeled within Cross Section A.

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 4. Computed safety factors were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. For the cases presented herein, the critical cross sections, i.e., the sections with the lowest computed safety factor, were found to be Cross Section A for static and liquefaction conditions and Cross Section B for seismic conditions. Figures 14 through 20 depict the safety factors for Cross Sections A and B. While both rotational mode (i.e., circular slip surface mode) and non-rotational (i.e., block slip surface mode) were considered in the analyses, block-type failures were consistently more critical for the failure modes of concern.

CONCLUSIONS

Based on the assumptions, analyses, and results presented within this calculation package, the South Ash Pond at WGS meets the periodic safety factor requirements described within the CCR Rule for existing CCR surface impoundments.

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Written by: **A. Brewster / A. Brown** Date: **10/12/2016** Reviewed by: **B. Gross** Date: **10/12/2016**

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

TABLES

Table 1. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters		Undrained Parameters[1]	
		φ′ (°)	c′ (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill	120[2]	27 to 36[3]	0	-	-
Dike Fill (Liquefaction)	120	0	30		
Foundation Soils (Clayey)	94[2]	15	300	Varies[4]	300
Foundation Soils (Clayey Sands)	123[2]	30 to 32[3]	0	-	-
Chicora	130[2]	50[2]	0	-	-
Williamsburg Formation Clay	105[2]	50[2]	0	-	-
Fly Ash	100[2]	34[2]	0	-	-
Riprap Buttress	150	45	0	-	-

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only.
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 explained 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 11.
4. The selected undrained strength ratio (S_u/σ'_{vo}) varies between locations and ranges from 0.25 to 0.70 for the selected cross sections. Interpretation of in-situ results applied in the selection is provided in Figures 7 through 11. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in the Data Package.

Table 2. Maximum Equivalent Horizontal Acceleration (MHEA) from Site Response Analysis for South Ash Pond Perimeter Dikes

Representative Perimeter Dike Profile 1							Representative Perimeter Dike Profile 2						
Depth (ft)	Maximum Equivalent Horizontal Acceleration (g)						Depth (ft)	Maximum Equivalent Horizontal Acceleration (g)					
	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360		BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360
2.5	0.097	0.097	0.100	0.094	0.091	0.131	2.5	0.106	0.090	0.078	0.071	0.080	0.114
7.5	0.074	0.080	0.064	0.073	0.076	0.102	7.5	0.074	0.075	0.066	0.061	0.070	0.085
12.5	0.062	0.073	0.059	0.067	0.070	0.079	12.5	0.057	0.066	0.060	0.055	0.062	0.070
16.5	0.060	0.070	0.058	0.062	0.066	0.071	16.5	0.054	0.063	0.056	0.054	0.058	0.065
18	0.060	0.071	0.055	0.061	0.060	0.073	18	0.057	0.065	0.053	0.057	0.054	0.066
19.5	0.059	0.069	0.058	0.060	0.065	0.071	20.5	0.054	0.063	0.054	0.054	0.057	0.063
23.5	0.058	0.065	0.053	0.057	0.061	0.067	23	0.054	0.058	0.050	0.052	0.051	0.061
28.5	0.056	0.058	0.052	0.054	0.054	0.059	25.5	0.053	0.058	0.050	0.050	0.052	0.059
33.5	0.053	0.051	0.050	0.050	0.050	0.053	30.5	0.050	0.051	0.045	0.047	0.046	0.053
36	0.061	0.051	0.050	0.049	0.051	0.057	35.5	0.047	0.046	0.043	0.043	0.043	0.048
38	0.060	0.050	0.049	0.048	0.050	0.060	40.5	0.044	0.042	0.040	0.040	0.040	0.044
42	0.062	0.049	0.045	0.046	0.048	0.060	45.5	0.041	0.039	0.036	0.037	0.039	0.041
46	0.062	0.049	0.042	0.045	0.047	0.059	48	0.047	0.039	0.033	0.037	0.038	0.046
48	0.063	0.046	0.038	0.045	0.046	0.054	50.5	0.047	0.039	0.035	0.038	0.038	0.045
50.5	0.062	0.046	0.039	0.045	0.045	0.062	58	0.056	0.043	0.040	0.041	0.038	0.050
58	0.063	0.049	0.043	0.047	0.044	0.068	68	0.062	0.047	0.050	0.046	0.042	0.055
68	0.067	0.053	0.050	0.050	0.042	0.074	78	0.062	0.051	0.055	0.049	0.045	0.061
78	0.071	0.056	0.055	0.051	0.045	0.078	88	0.066	0.054	0.056	0.049	0.047	0.068
88	0.074	0.058	0.058	0.050	0.048	0.080	98	0.068	0.061	0.060	0.042	0.049	0.068
98	0.076	0.058	0.059	0.047	0.049	0.085	108	0.073	0.060	0.059	0.042	0.048	0.073
108	0.076	0.058	0.059	0.045	0.050	0.088	118	0.073	0.059	0.058	0.043	0.049	0.077

Notes:

1. Cross Section A and Cross Section B (similar in subsurface stratigraphy and location to Profile 1) were found to have depths to the critical slip surface of approximately 10 ft and 27 ft, respectively. MHEA values of 0.092g and 0.062g were selected for Cross Section A and Cross Section B, respectively.
2. Cross Section C, Cross Section D, and Cross Section E (similar in subsurface stratigraphy and location to Profile 2) were found to have depths to the critical slip surface of approximately 17.5 ft, 16.5 ft, and 2.5 ft, respectively. MHEA values of 0.066g, 0.066 g, and 0.114g were selected for Cross Section C, Cross Section D, and Cross Section E, respectively.

Table 3. Tip Resistance Adjustment to Compute Residual Shear Strength by CPT (Idriss and Boulanger, 2008)

Fines Content (% Passing No. 200 Sieve)	$\Delta q_{c,1,N\text{-}rl}$
10	10
25	25
50	45
75	55

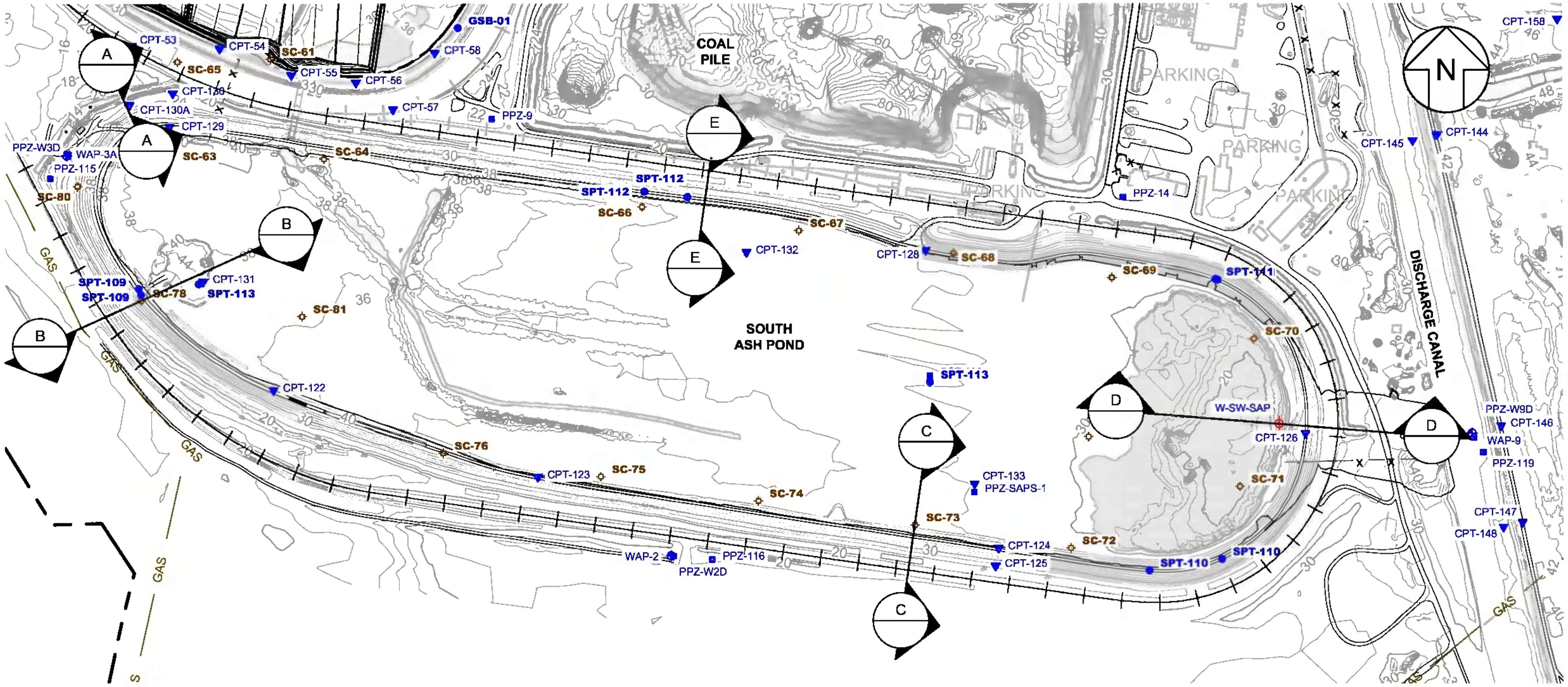
Table 4. Summary of Safety Factor Analysis Results

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	*1.69*	1.81	1.96	2.05	1.90
Static FS- Maximum Surcharge Pool	1.40	*1.69*	1.71	1.82	2.04	1.90
Seismic - Maximum Normal Storage Pool	1.00	1.09	*1.04*	1.12	1.28	1.26
Liquefaction Slope Stability[1]	1.20	*1.32*	N/A	N/A	N/A	N/A

Notes:

1. The liquefaction potential analysis for Cross Section A is presented in the Liquefaction Package. The liquefaction safety factors for Cross Sections B, C, D, and E were not evaluated as embankment soils were not found to be liquefiable (Liquefaction Package).
2. The lowest computed safety factor for each analysis case is *italicized*. Critical FS's for Cross Sections A and B were found to contain the lowest computed FS's and are shown in Figures 14 through 20.
3. Only critical failure surfaces passing through the perimeter dikes were considered.

FIGURES



LEGEND

10	EXISTING MAJOR GRADE CONTOUR
	EXISTING RAILROAD
A A	PROPOSED SECTION
▨	EXISTING PONDED WATER (28.73 FT NGVD)
⌖ W-SW-SAP	EXISTING STAFF GAUGE
⌖ SC-64	BORING BY OTHERS
▼ CPT-05	GEOSYNTEC CONE PENETRATION TEST
● GSB-01 ● SPT-109	GEOSYNTEC SOIL BORING
◆ WAP-2	MONITORING WELL
■ PPZ-9, PPZ-W2D	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

WGS - SOUTH ASH POND
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-F0028-023

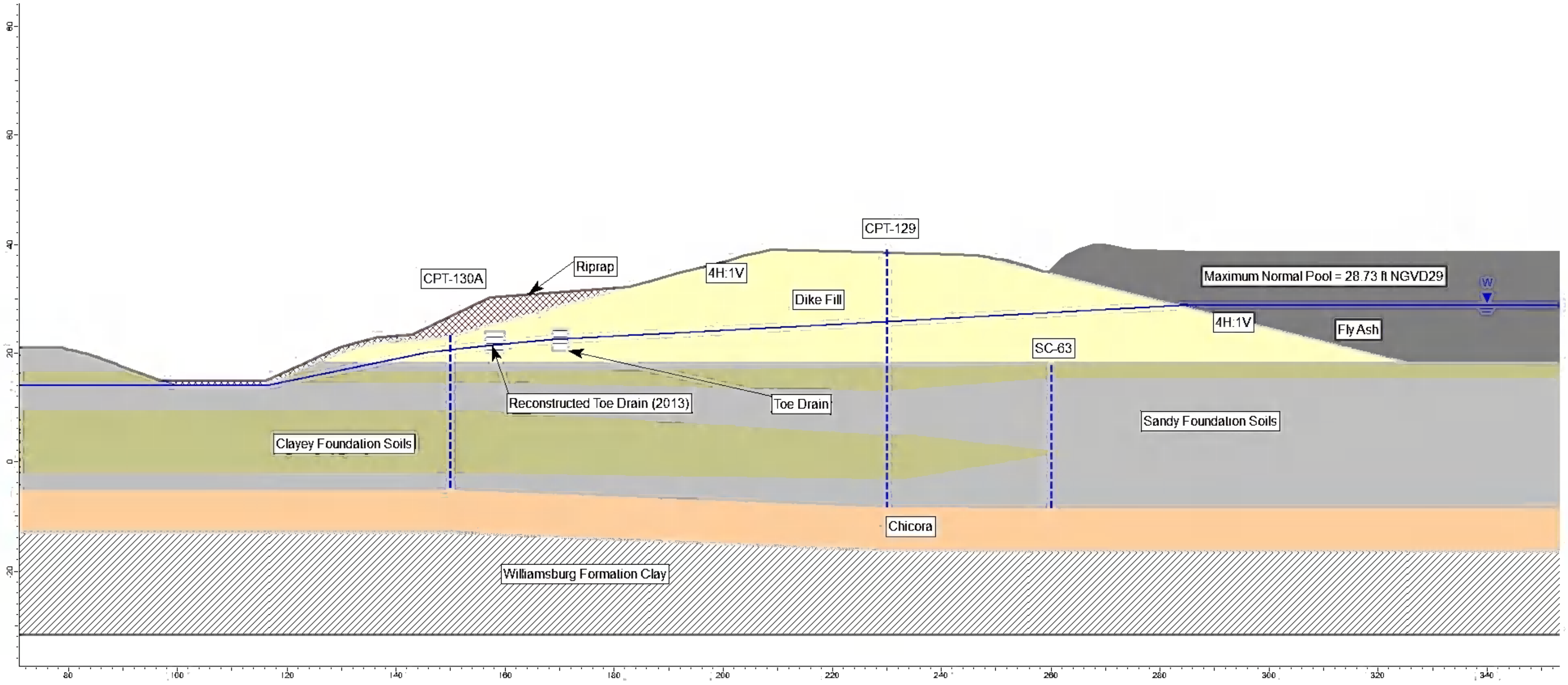


Figure 2. Cross Section A Geometry during Maximum Normal Storage Pool Conditions (as implemented within SLIDE®)

Note:

1. A riprap buttress was constructed against the downgradient dike slope and across the adjacent drainage swale in the vicinity of the computed liquefiable zone (Attachment 7).

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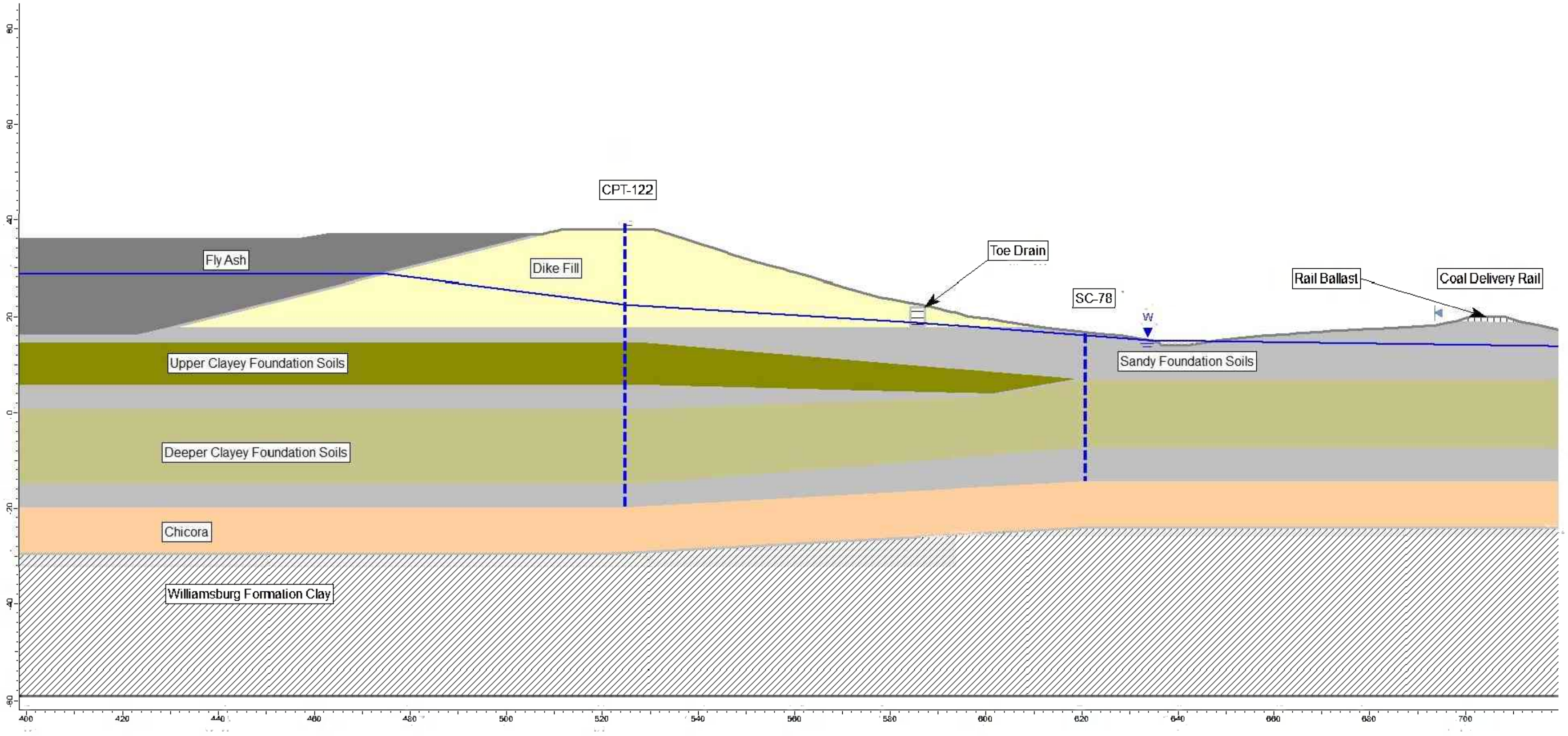

Figure 3. Cross Section B Geometry during Maximum Normal Storage Pool Conditions (as implemented within SLIDE®)

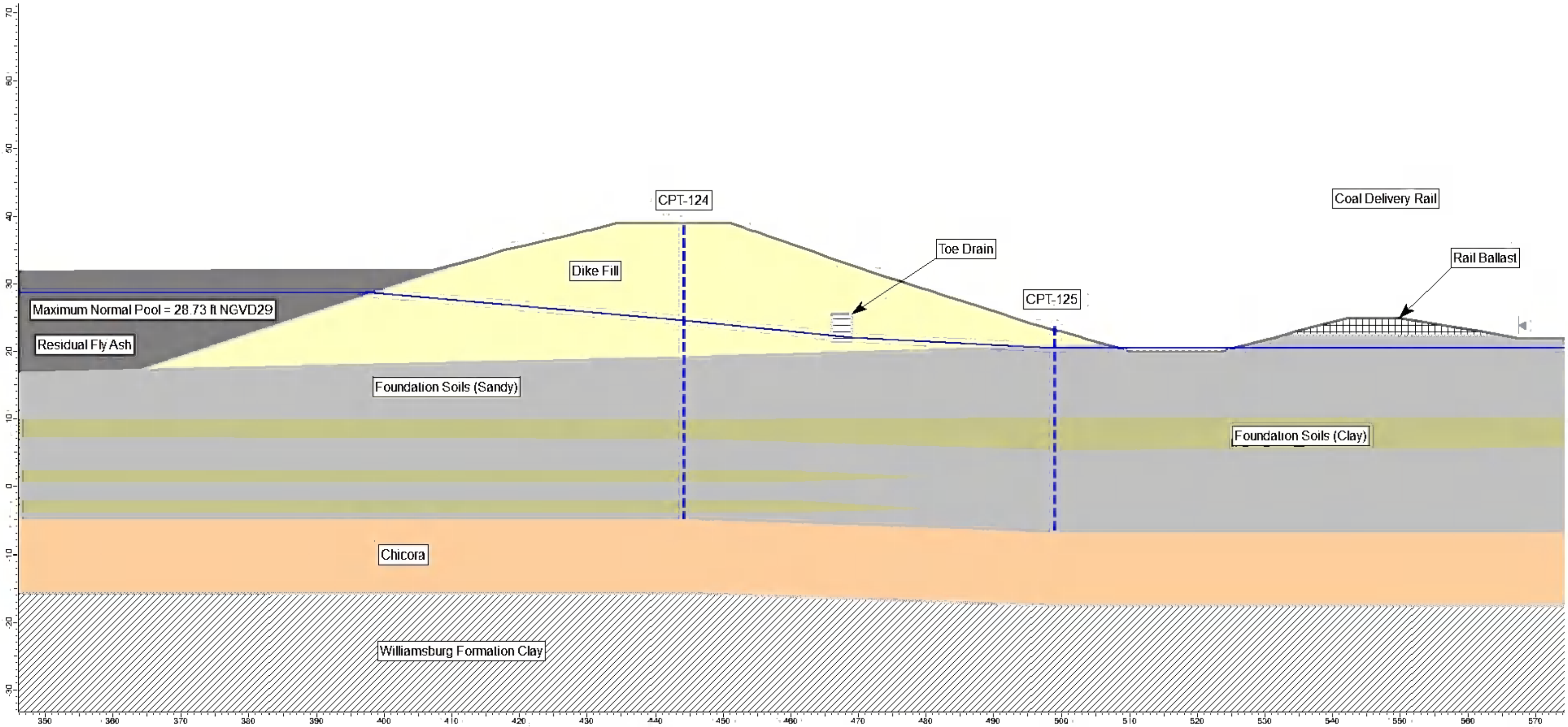

Figure 4. Cross Section C Geometry during Maximum Normal Storage Pool Conditions (as implemented within SLIDE®)

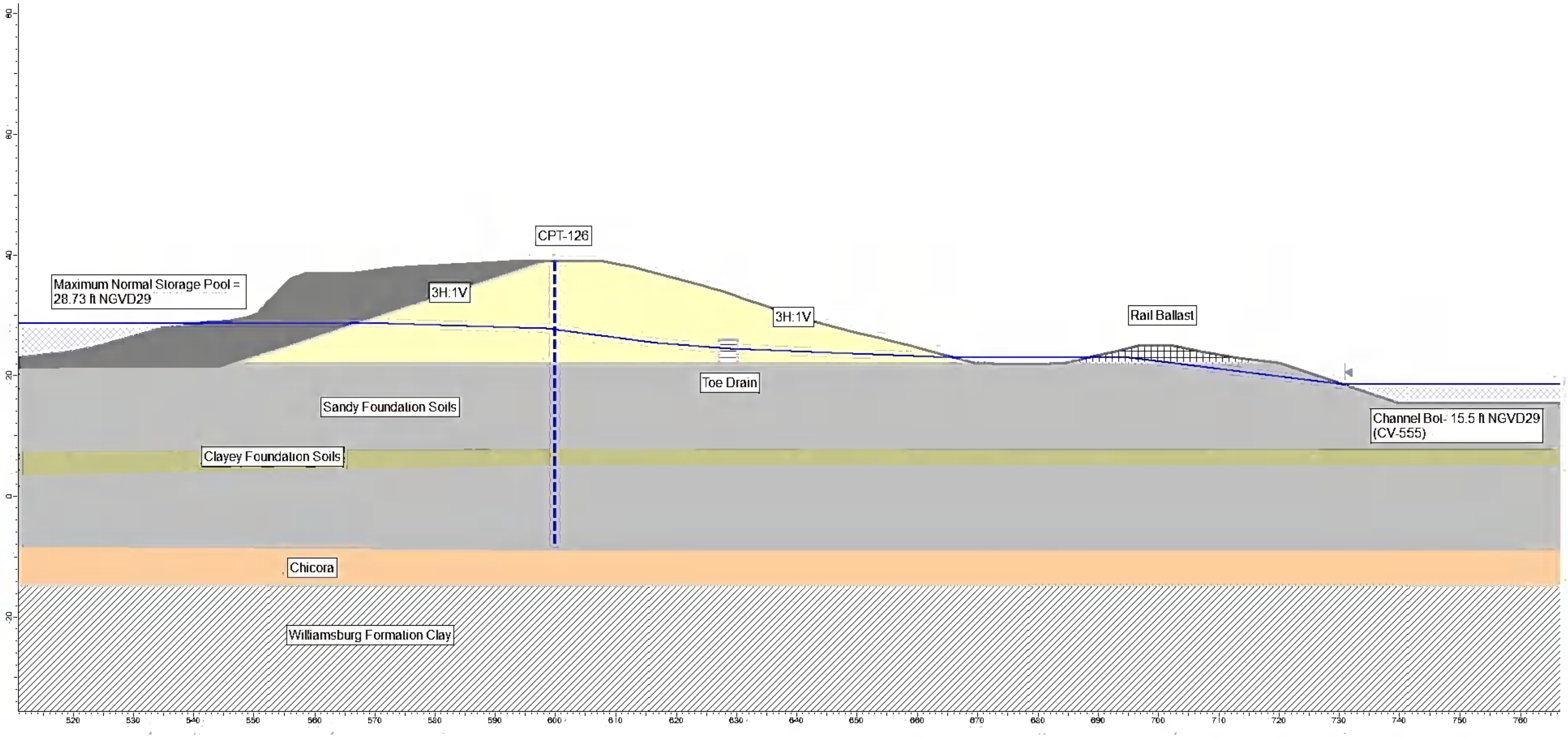

Figure 5. Cross Section D Geometry during Maximum Normal Storage Pool Conditions (as implemented within SLIDE®)

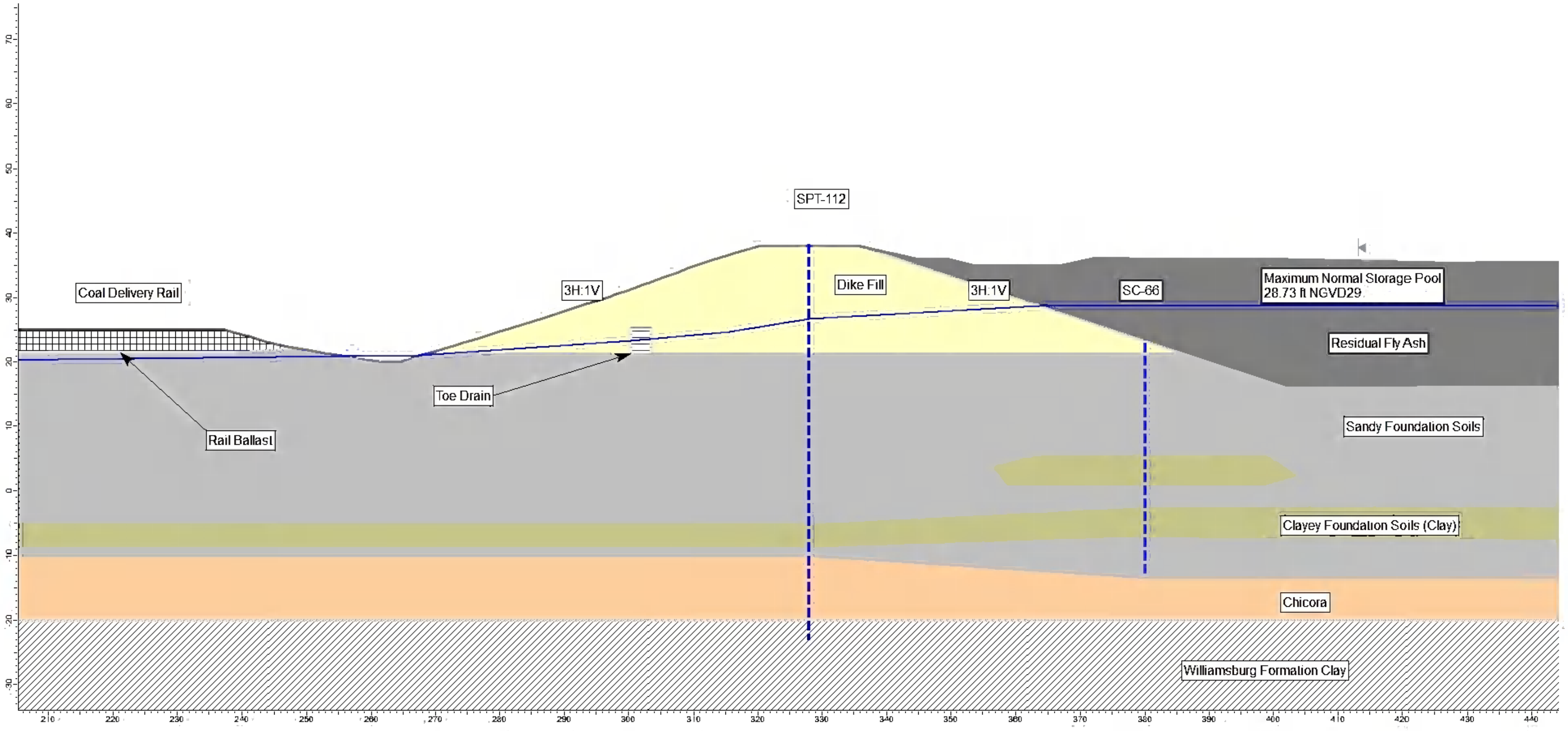

Figure 6. Cross Section E Geometry during Maximum Normal Storage Pool Conditions (as implemented within SLIDE®)

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

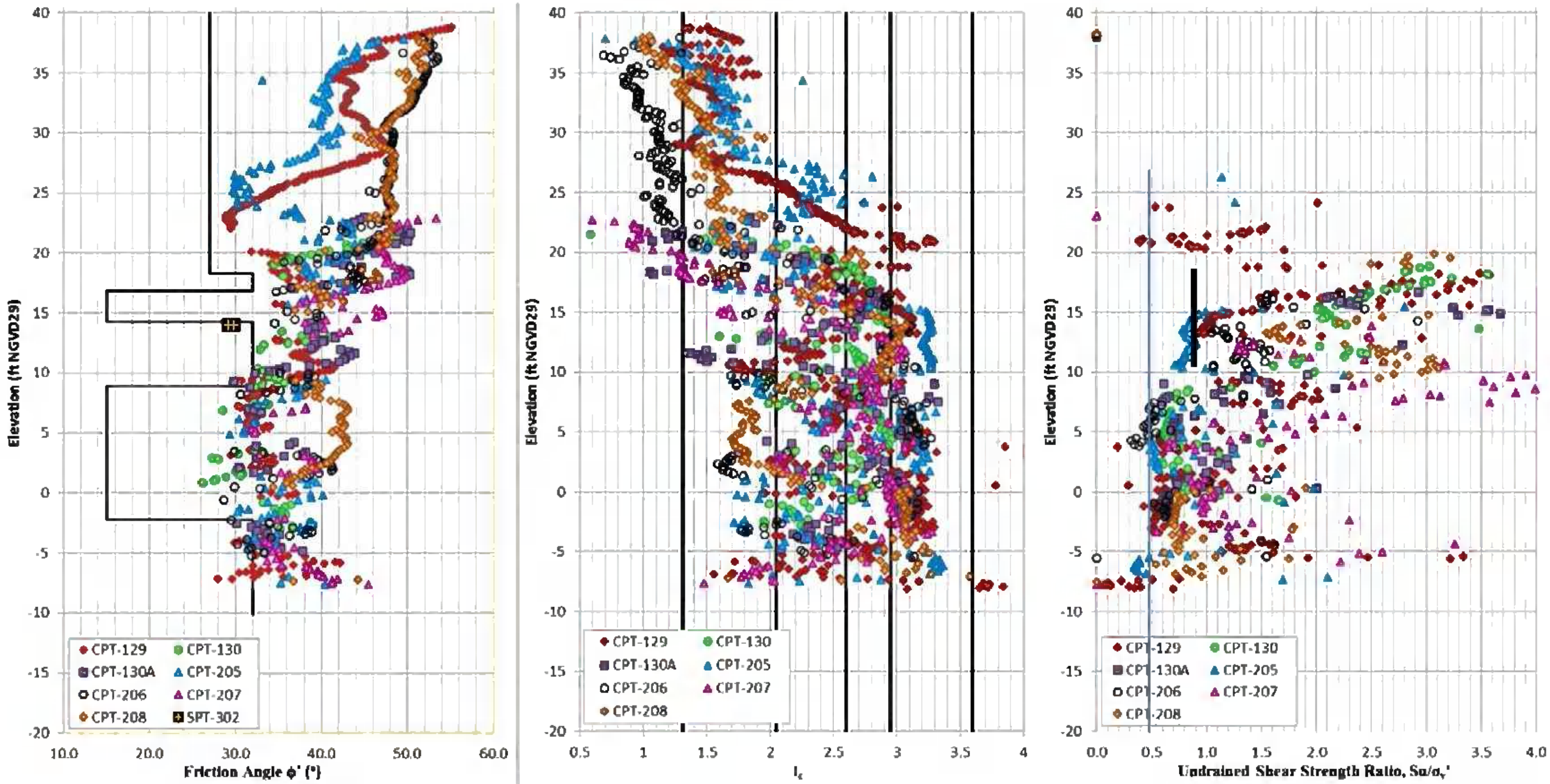

Figure 7. Subsurface Stratigraphy and Shear Strength Model for Cross Section A

Notes:

1. Clayey foundation soils were modeled with a $\phi'=15°$ and a $c'=300$ psf during static slope stability and with 80% of the $S_u/\sigma'_v = 0.5$ (i.e., $S_u/\sigma'_v = 0.4$) and 80% of the $S_{u,min} = 300$ psf (i.e. $S_{u,min} = 240$ psf) during pseudo-static stability analysis (i.e., seismic safety factor).
2. The dike fill soils were conservatively modeled as a single material with $\phi' = 27°$.
3. The sandy foundation soils were modeled with $\phi' = 32°$.

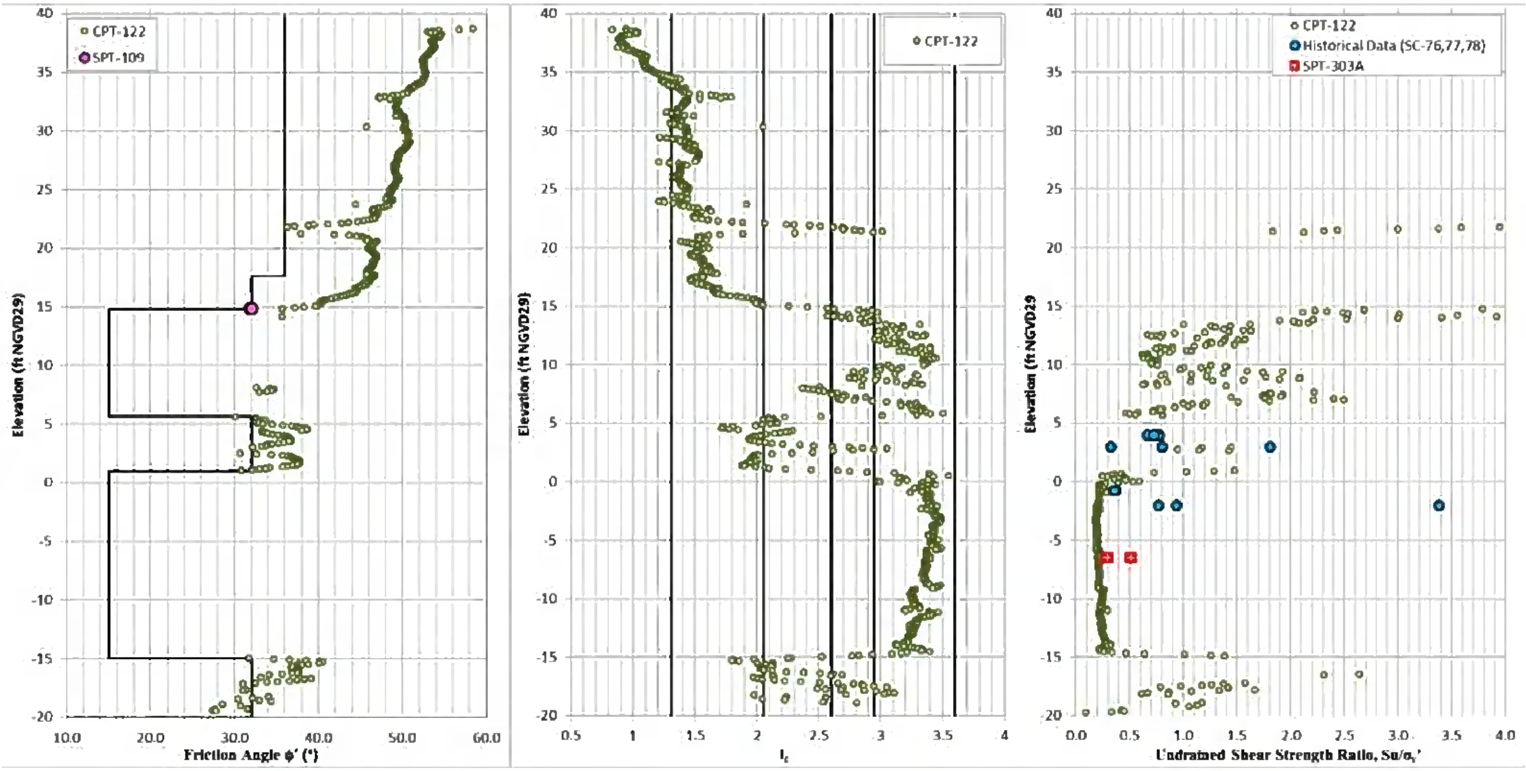

Figure 8. Subsurface Stratigraphy and Shear Strength Model for Cross Section B

Notes:

1. Clayey foundation soils (both upper clayey foundation soils and deeper clayey foundation soils) were modeled with a ϕ′=15° and a c′=300 psf during static slope stability. For pseudo-static stability analysis (i.e. seismic safety factor), the upper clayey foundation soils from 14.8 ft to 5.6 ft NGVD29 were modeled with 80% of the $S_u/\sigma'_v = 0.70$ (i.e. $S_u/\sigma'_v = 0.56$) and 80% of the $S_{u,min}$=300 psf (i.e. $S_{u,min}$=240 psf), and the deeper clayey foundation soils from 1.0 ft to -15.0 ft NGVD29 were modeled with 80% of the $S_u/\sigma'_v = 0.25$ (i.e. $S_u/\sigma'_v = 0.20$) and 80% of the $S_{u,min}$=300 psf (i.e. $S_{u,min}$=240 psf).
2. The sandy foundation soils were modeled with ϕ′ = 32°, and the dike fill soils were modeled with ϕ′ = 36°.

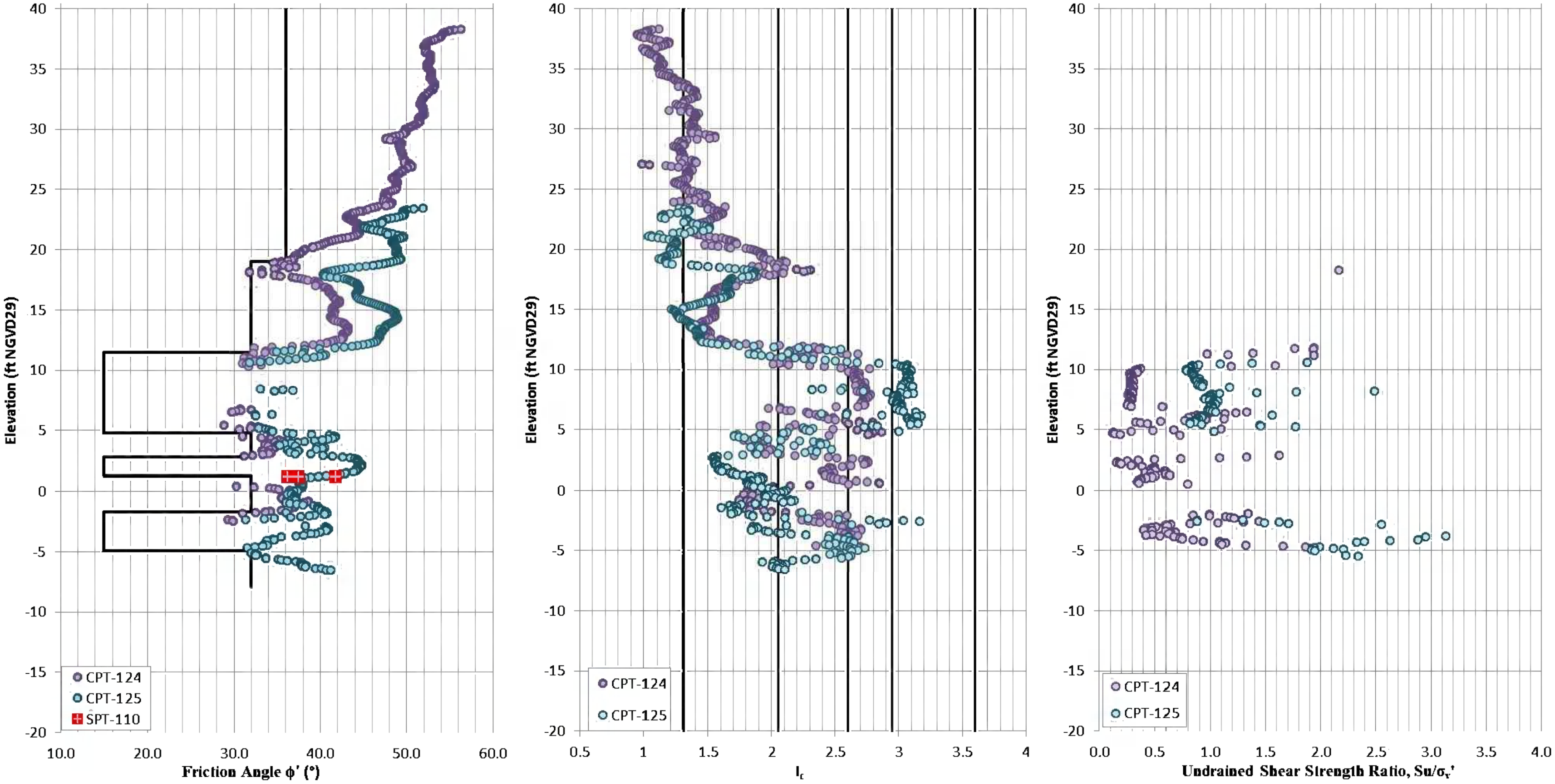

Figure 9. Subsurface Stratigraphy and Shear Strength Model for Cross Section C

Notes:

1. Clayey foundation soils were modeled with a ϕ'=15° and a c'=300 psf during static slope stability and with 80% of the S_u/σ'_v = 0.3 (i.e., S_u/σ'_v = 0.24) and 80% of the $S_{u,min}$ = 300 psf (i.e. $S_{u,min}$ = 240 psf) during pseudo-static stability analysis (i.e., seismic safety factor).
2. The sandy foundation soils were modeled with ϕ' = 32°, and the dike fill soils were modeled with ϕ' = 36°.

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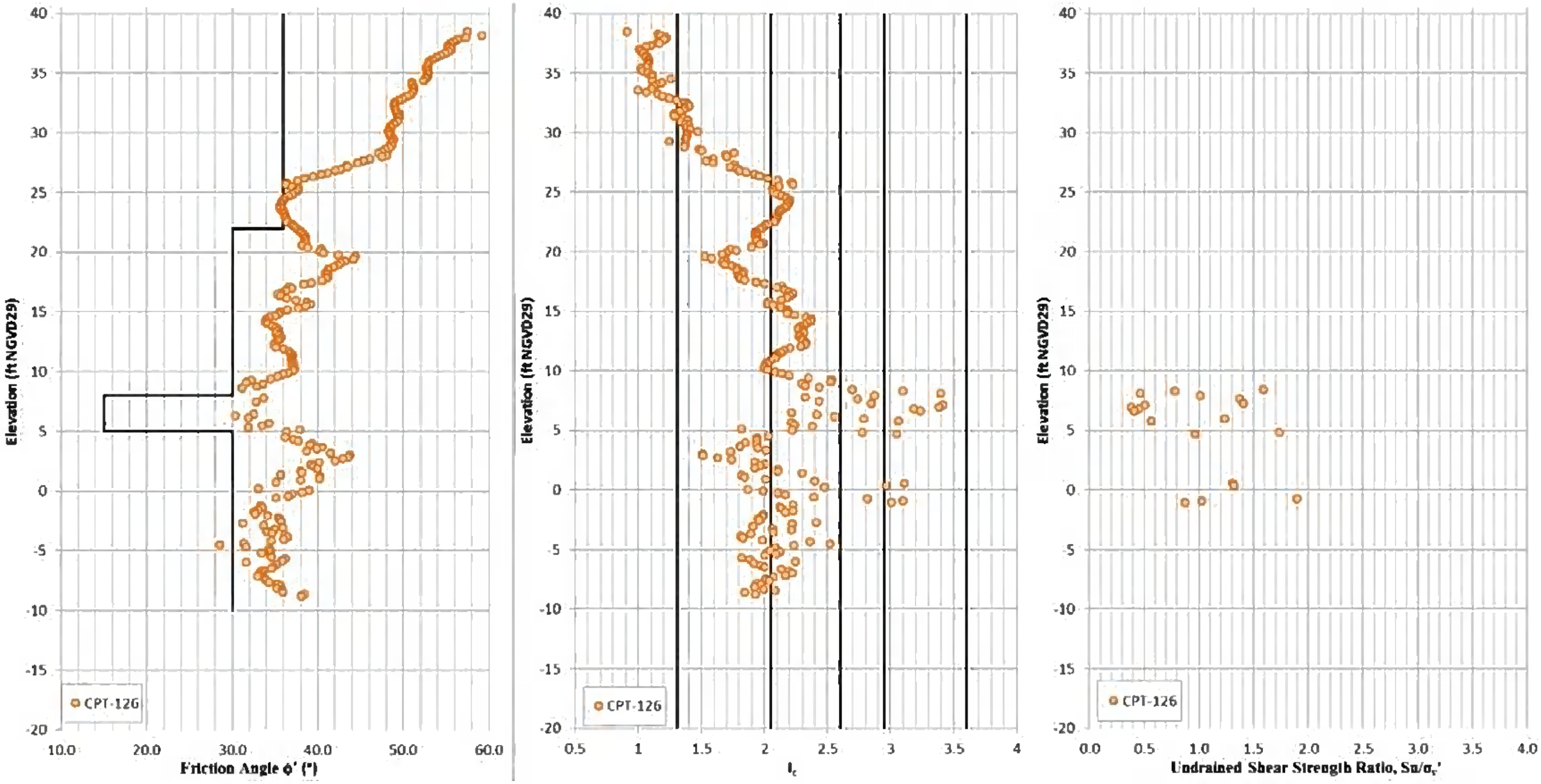

Figure 10. Subsurface Stratigraphy and Shear Strength Model for Cross Section D

Notes:

1. Clayey foundation soils were modeled with a ϕ′=15° and a c′=300 psf during static slope stability and with 80% of the $S_u/\sigma'_v = 0.3$ (i.e., $S_u/\sigma'_v = 0.24$) and 80% of the $S_{u,min} = 300$ psf (i.e. $S_{u,min} = 240$ psf) during pseudo-static stability analysis (i.e., seismic safety factor).
2. The sandy foundation soils were modeled with ϕ′ = 30°, and the dike fill soils were modeled with ϕ′ = 36°.

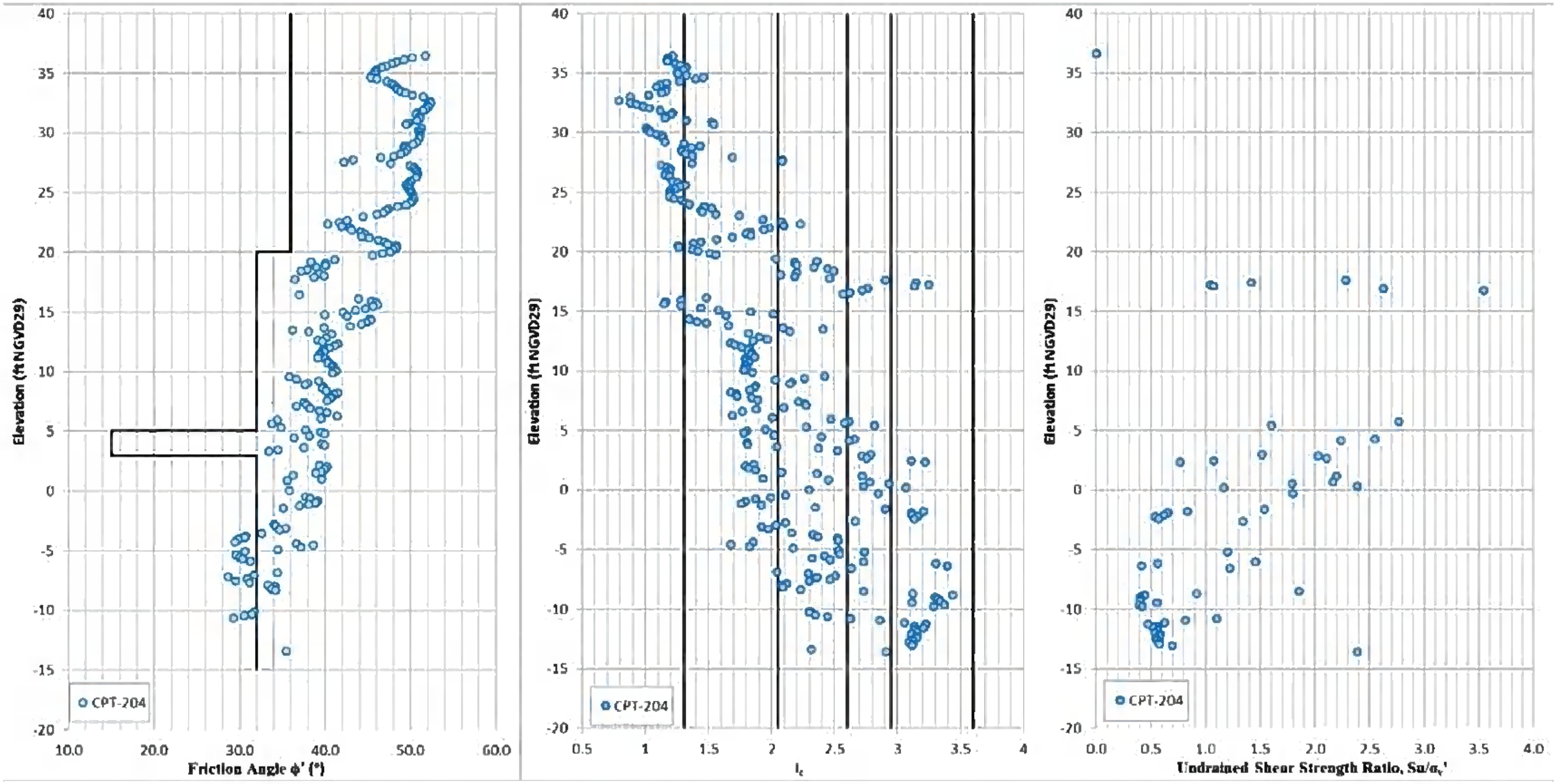

Figure 11. Subsurface Stratigraphy and Shear Strength Model for Cross Section E

Notes:

1. Clayey foundation soils were modeled with a $\phi'=15^\circ$ and a $c'=300$ psf during static slope stability and with 80% of the $S_u/\sigma'_v = 0.3$ (i.e., $S_u/\sigma'_v = 0.24$) and 80% of the $S_{u,min} = 300$ psf (i.e. $S_{u,min} = 240$ psf) during pseudo-static stability analysis (i.e., seismic safety factor).
2. The sandy foundation soils were modeled with $\phi' = 32^\circ$, and the dike fill soils were modeled with $\phi' = 36^\circ$.

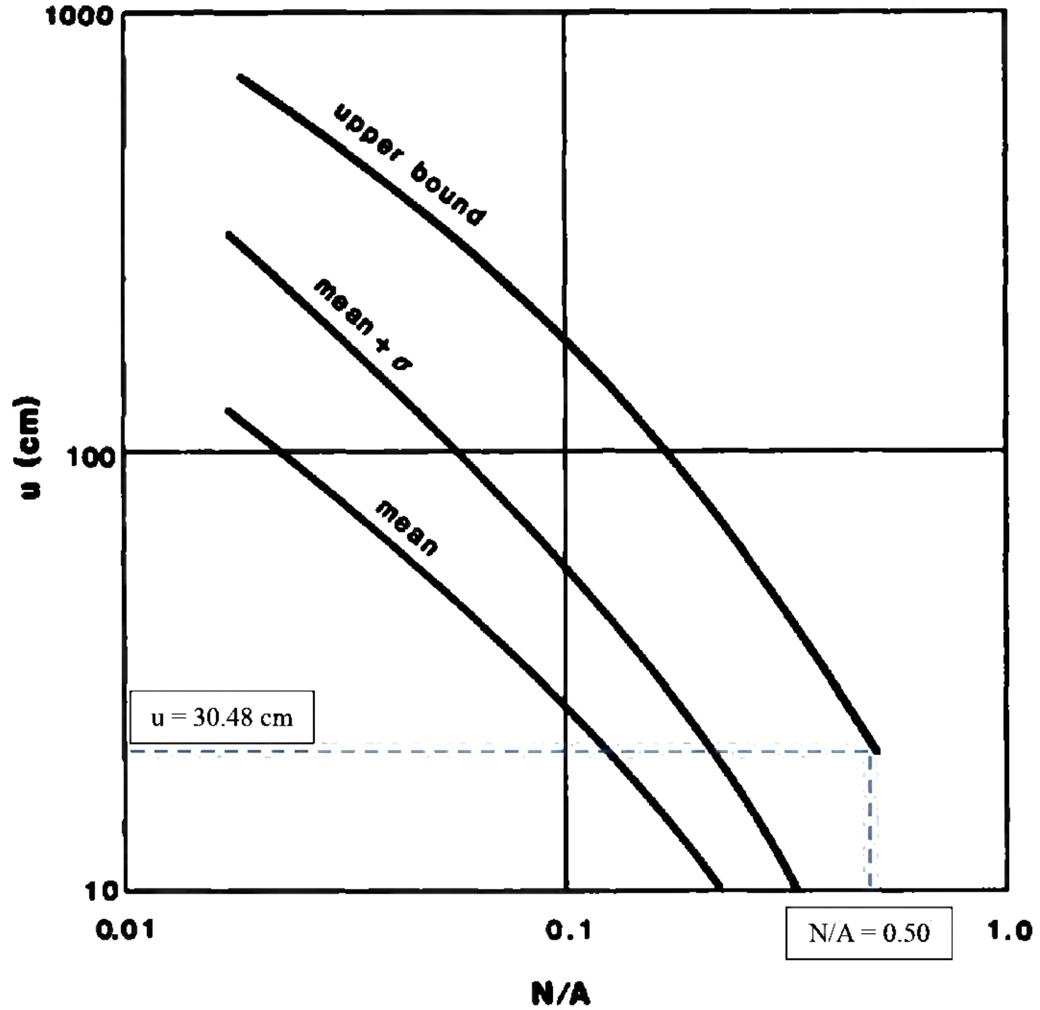

Figure 12. Allowable Displacement (u) vs. N/A (from Figure 7 of Hynes-Griffin and Franklin, 1984)

Notes:

1. An allowable deformation (u) of 12 in. (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 in. of displacement.

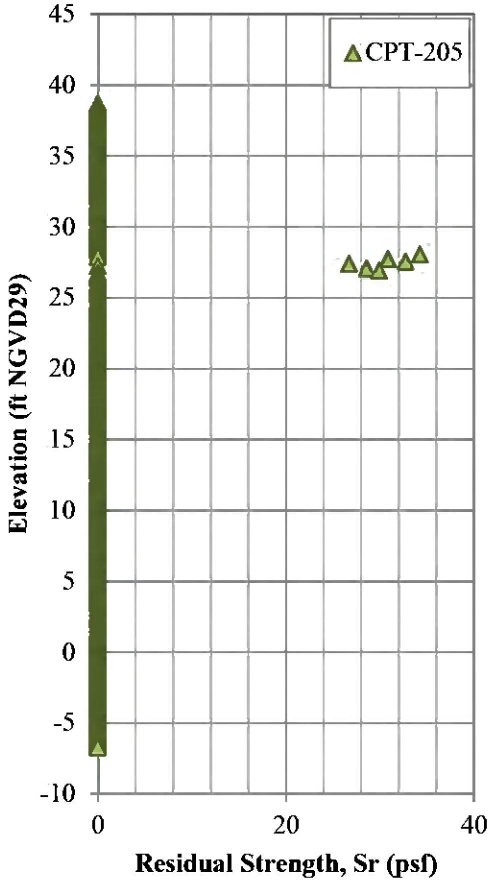

Figure 13. Computed Residual Shear Strength of Liquefiable Zone

Notes:

1. Liquefaction potential analysis is presented in Attachment 7 of this Stability Report.
2. Residual strengths of non-liquefiable soils are plotted above as zeros to show the extent of soil data within CPT-205. Long-term, drained shear strength parameters were applied for non-liquefiable soils. Liquefiable soils were computed in accordance with Idriss and Boulanger (2008).

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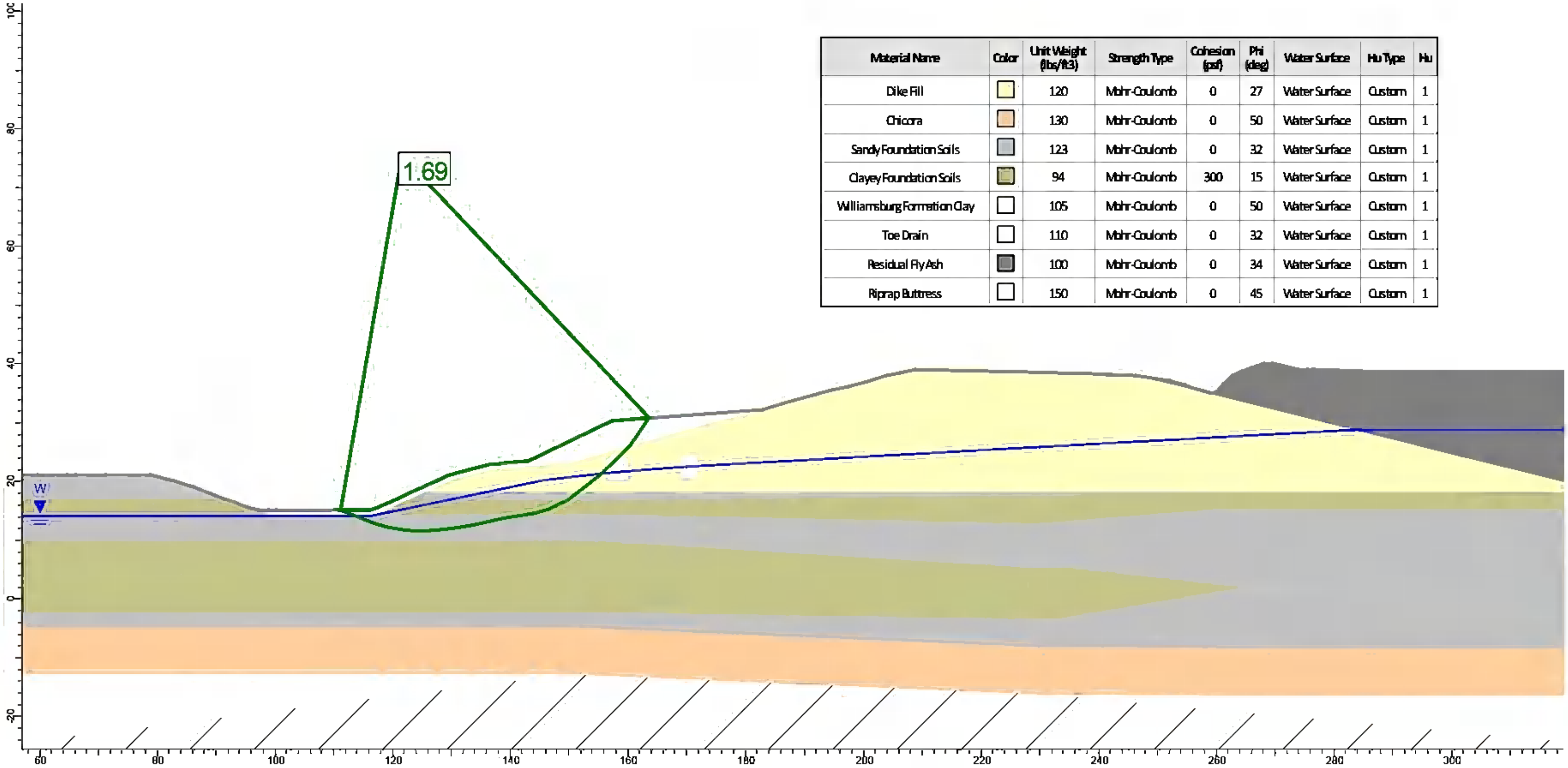

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	🟩	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	⬛	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	☐	150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 14. Critical Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool

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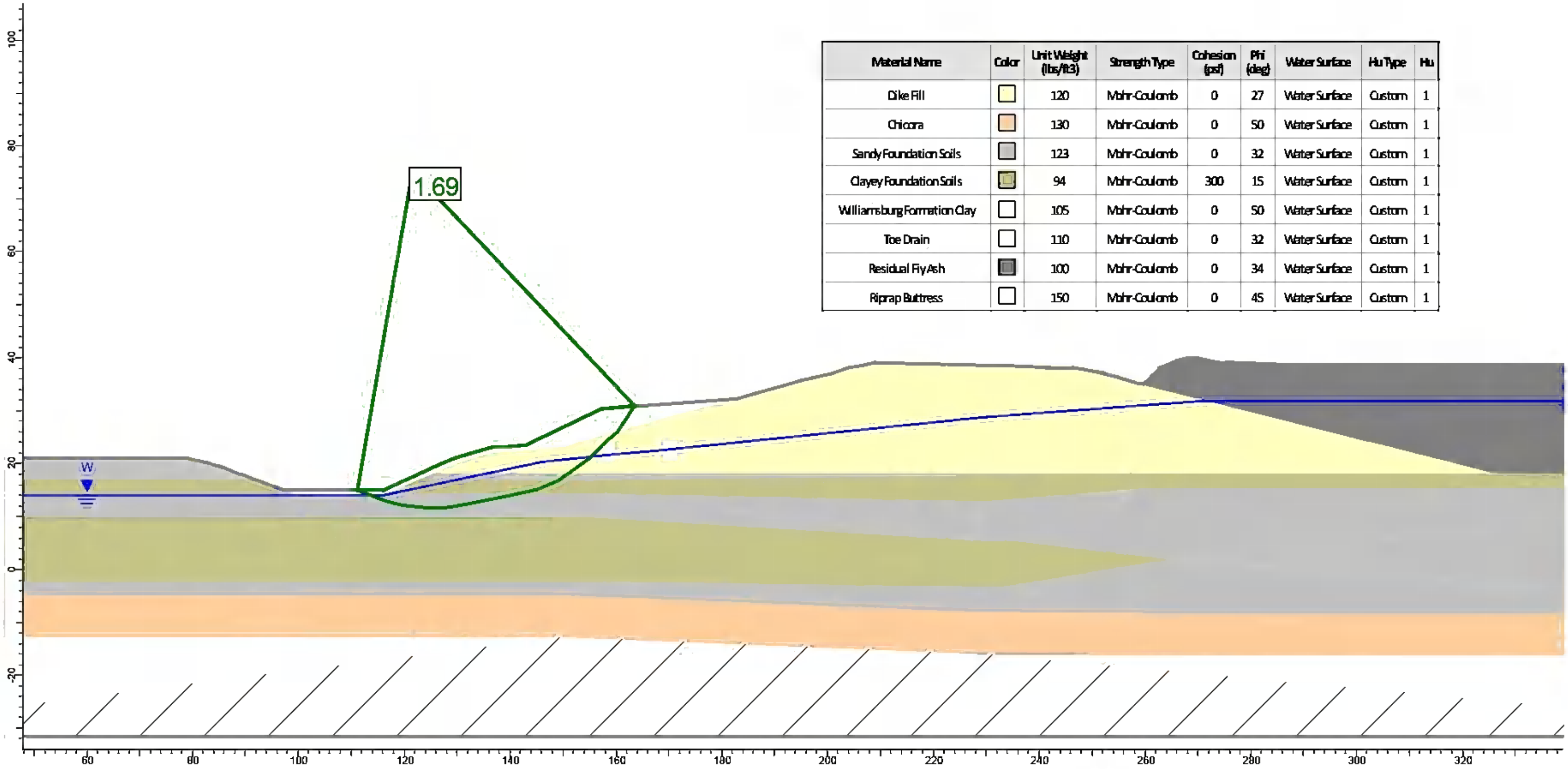
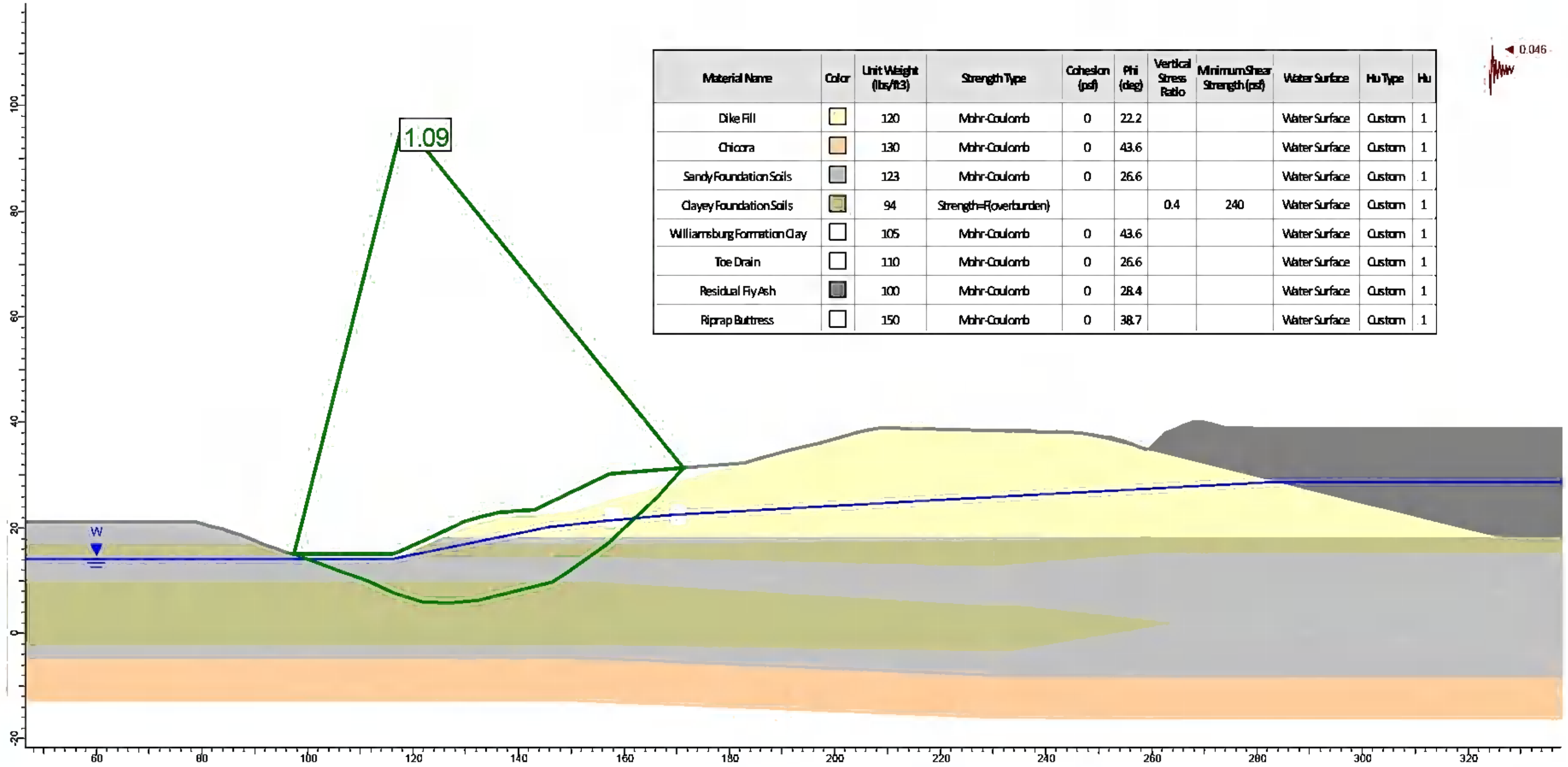
Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



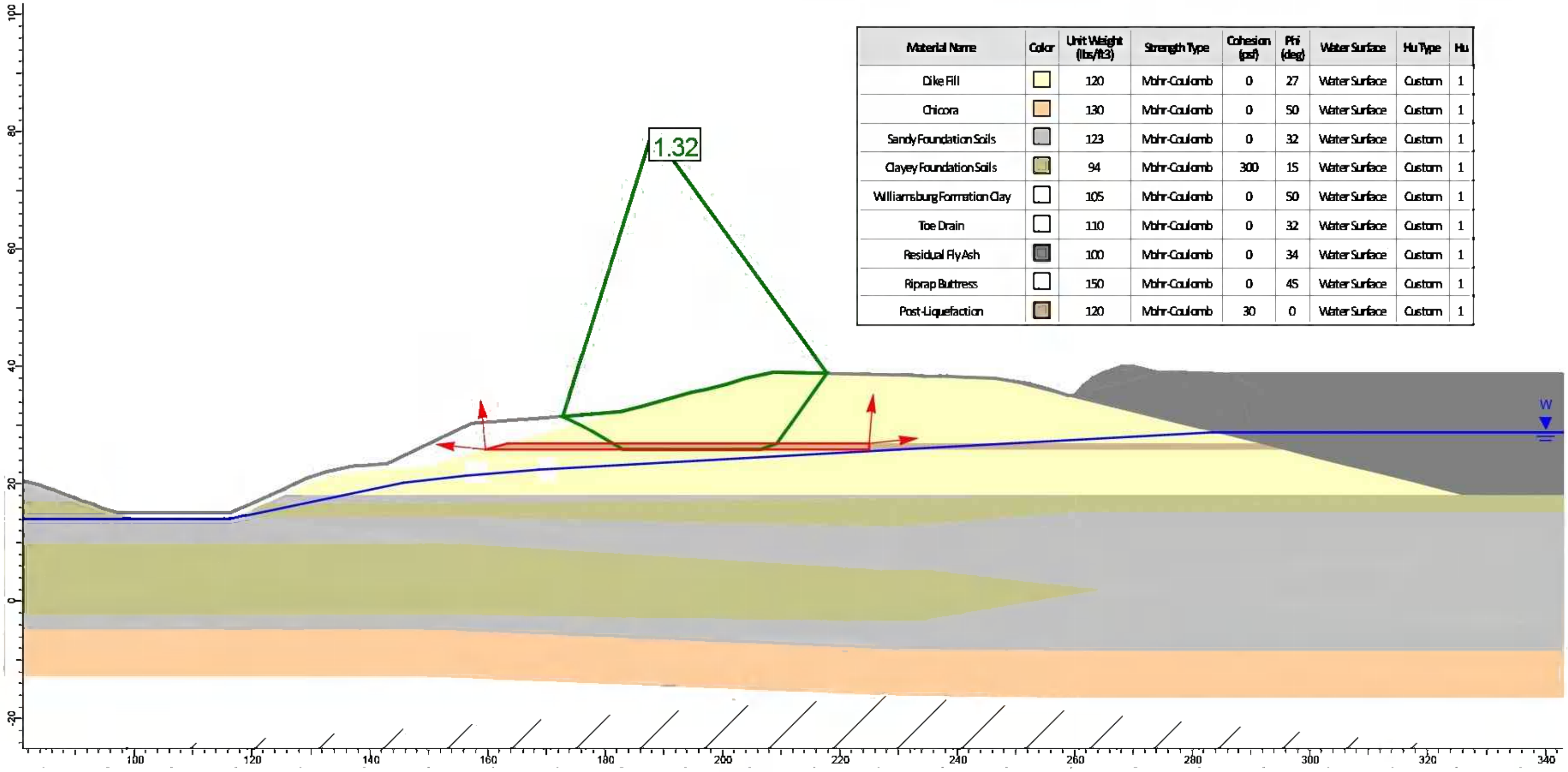
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	▨	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	☐	150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 15. Critical Factor of Safety for Cross Section A: Static Factor of Safety - Maximum Surcharge Pool



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	22.2			Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Clayey Foundation Soils	🟩	94	Strength=F(overburden)			0.4	240	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	28.4			Water Surface	Custom	1
Riprap Buttress	☐	150	Mohr-Coulomb	0	38.7			Water Surface	Custom	1

Figure 16. Critical Factor of Safety for Cross Section A: Seismic Factor of Safety – Maximum Normal Storage Pool

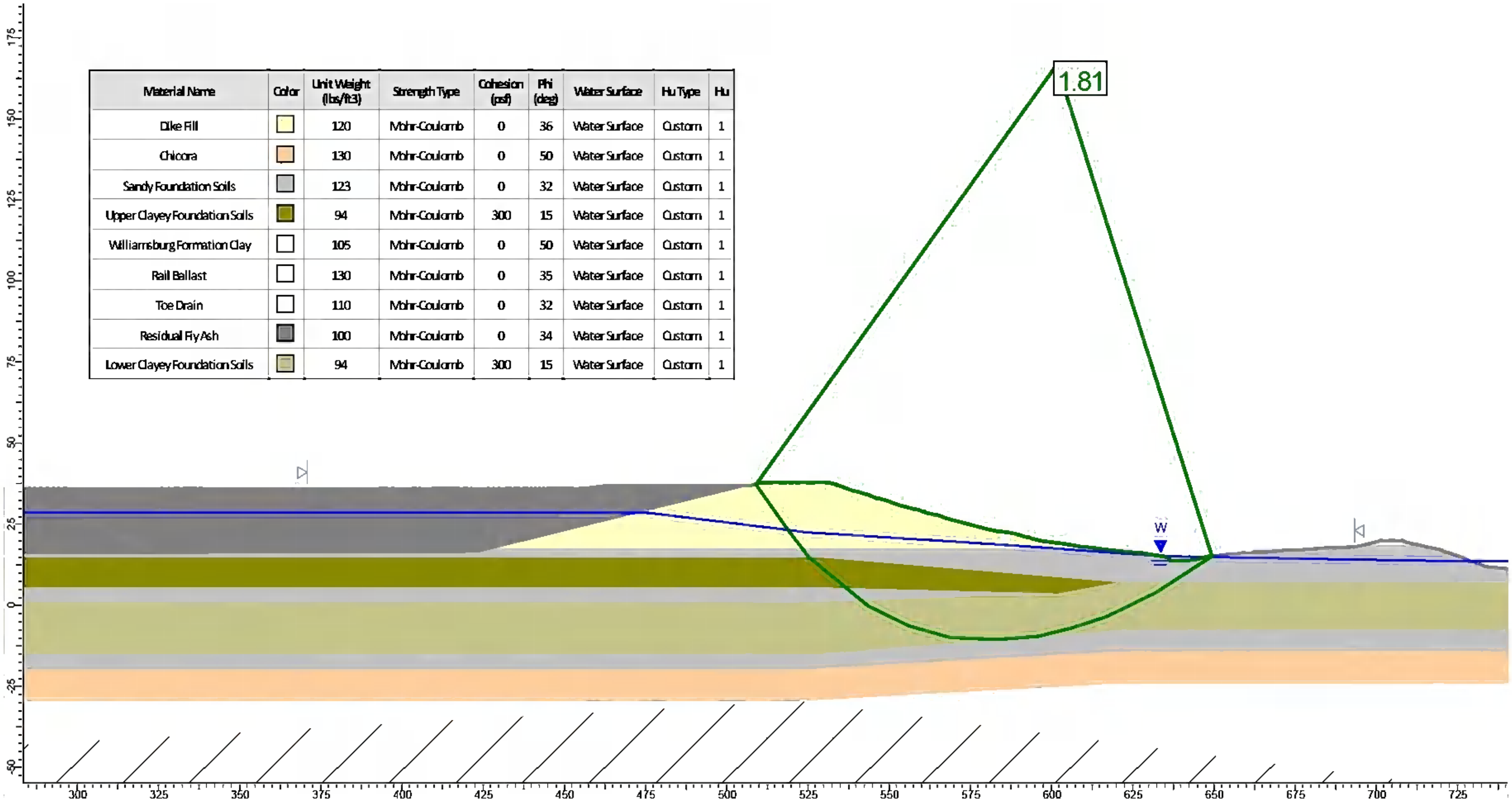


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils	🟩	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	□	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain	□	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	⬛	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress	□	150	Mohr-Coulomb	0	45	Water Surface	Custom	1
Post-Liquefaction	🟫	120	Mohr-Coulomb	30	0	Water Surface	Custom	1

Figure 17. Critical Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool – Liquefaction

Written by: **A. Brewster / A. Brown** Date: **10/12/2016** Reviewed by: **B. Gross** Date: **10/12/2016**

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

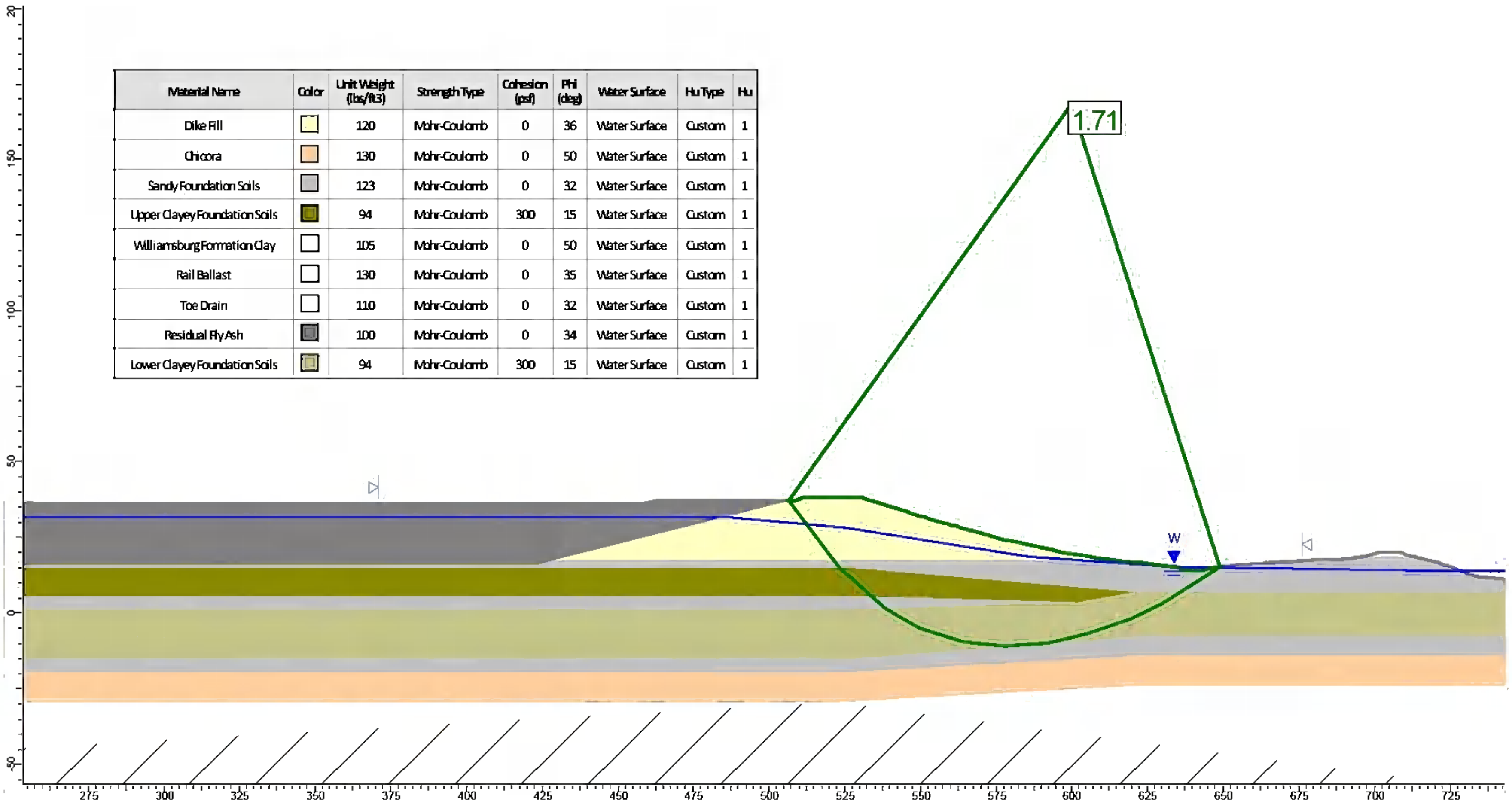


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	36	Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Upper Clayey Foundation Soils	🟫	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	☐	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Rail Ballast	☐	130	Mohr-Coulomb	0	35	Water Surface	Custom	1
Toe Drain	☐	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Lower Clayey Foundation Soils	▨	94	Mohr-Coulomb	300	15	Water Surface	Custom	1

Figure 18. Critical Factor of Safety for Cross Section B: Static Factor of Safety – Maximum Normal Storage Pool

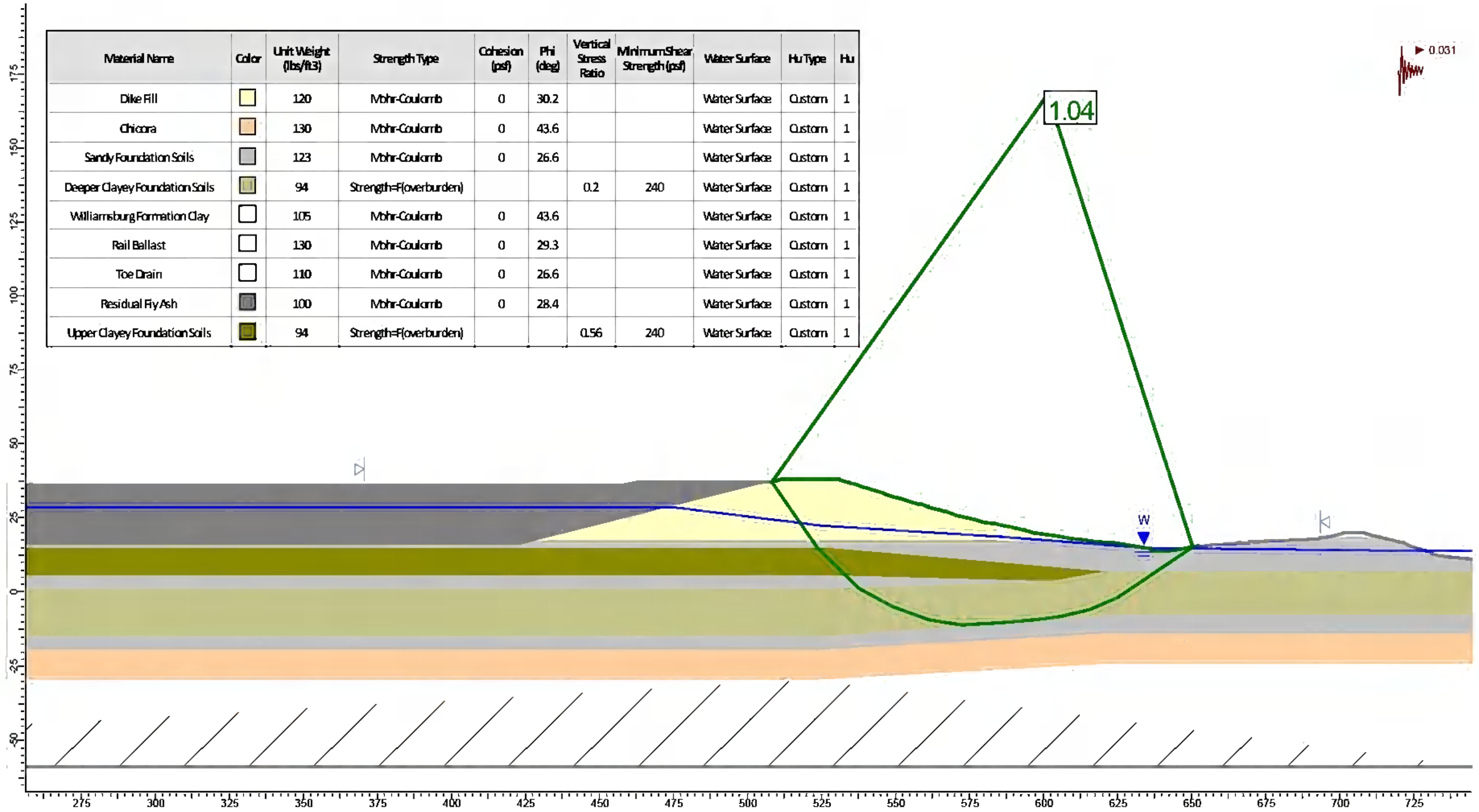
Written by: **A. Brewster / A. Brown** Date: **10/12/2016** Reviewed by: **B. Gross** Date: **10/12/2016**

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



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill	■	120	Mohr-Coulomb	0	36	Water Surface	Custom	1
Chicora	■	130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils	■	123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Upper Clayey Foundation Soils	■	94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay	□	105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Rail Ballast	□	130	Mohr-Coulomb	0	35	Water Surface	Custom	1
Toe Drain	□	110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Lower Clayey Foundation Soils	■	94	Mohr-Coulomb	300	15	Water Surface	Custom	1

Figure 19. Critical Factor of Safety for Cross Section B: Static Factor of Safety - Maximum Surcharge Pool



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill	🟨	120	Mohr-Coulomb	0	30.2			Water Surface	Custom	1
Chicora	🟧	130	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Sandy Foundation Soils	▨	123	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Deeper Clayey Foundation Soils	▨	94	Strength=F(overburden)			0.2	240	Water Surface	Custom	1
Williamsburg Formation Clay	□	105	Mohr-Coulomb	0	43.6			Water Surface	Custom	1
Rail Ballast	□	130	Mohr-Coulomb	0	29.3			Water Surface	Custom	1
Toe Drain	□	110	Mohr-Coulomb	0	26.6			Water Surface	Custom	1
Residual Fly Ash	■	100	Mohr-Coulomb	0	28.4			Water Surface	Custom	1
Upper Clayey Foundation Soils	■	94	Strength=F(overburden)			0.56	240	Water Surface	Custom	1

Figure 20. Critical Factor of Safety for Cross Section B: Seismic Factor of Safety – Maximum Normal Storage Pool