Prepared for

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2016 SURFACE IMPOUNDMENT PERIODIC SAFETY FACTOR ASSESSMENT REPORT

SLURRY POND

WINYAH GENERATING STATION, GEORGETOWN, SOUTH CAROLINA

Prepared by

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engineers | scientists | innovators

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CERTIFICATION STATEMENT

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

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10/13/2016

Date

EXECUTIVE SUMMARY

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

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR (Code of Federal Regulations) Parts 257 and 261, regulating on 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 Slurry Pond at WGS is classified as an "existing CCR surface impoundment" by the CCR Rule. The *2016 Surface Impoundment Periodic Safety Factor Assessment: Slurry Pond* (Safety Factor Assessment Report) presents the first periodic (i.e., initial) safety factor assessment in accordance with the CCR Rule for the Slurry Pond at WGS prepared by Geosyntec Consultants (Geosyntec) on behalf of Santee Cooper.

A hydrologic and hydraulic analysis (Attachment 1) of the Slurry Pond and appurtenances was conducted to demonstrate the inflow design flood (IDF) can be managed and conveyed safely (i.e., without overtopping the perimeter dikes) during and after the rainfall event. Since the surface impoundment has identified as a "High Hazard Potential" surface impoundment, the probable maximum flood (PMF) with rainfall duration of 72 hours was selected as the IDF. The surface water level within the Slurry Pond is maintained at an elevation of 19.6 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a Floating Pump Station which ultimately conveys free water to the Site's Cooling Pond. The peak water level during and after the IDF within the Slurry Pond was computed as 35.4 ft NGVD29, which is below the minimum dike crest of 36.0 ft NGVD29. Thus, the Slurry Pond will adequately manage inflows during and following the peak discharge from the PMF in accordance with §257.73(d)(1)(v) of the CCR Rule.

In support of the periodic safety factor assessment, Geosyntec developed and performed a site investigation and laboratory testing program to characterize the dike and subsurface soils for the Slurry Pond in 2013. Data from historical field investigations was also reviewed and incorporated into the evaluation of the condition of the Slurry Pond and selection of engineering properties for the dike fill and foundation soils. Boring logs, Cone Penetration Test (CPT) sounding data, and laboratory testing results have been provided in Attachments 2, 3, and 4 of this Safety Factor Assessment Report, respectively, 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 probability of exceedance of 2% in 50 years (i.e., 2,475 year return period) as defined in §257.53. Site response analyses (Attachment 6) were performed to evaluate the influence of the local subsurface conditions on the maximum horizontal acceleration and to compute the maximum cyclic shear stresses developed within the soil profile during the design earthquake.

The potential of dike fill to liquefy during the design earthquake was evaluated at each 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. The evaluation results did not show that the dike fill of the Slurry Pond perimeter dikes or the foundation soils directly underlying the perimeter dikes of the Slurry Pond were susceptible to liquefaction during the design earthquake. It is noted that the liquefaction potential of the foundation soils near the downstream dike toe (i.e., outside the perimeter dike footprint) of the Slurry Pond will be evaluated separately as a part of an evaluation of "Unstable Areas" in accordance with §257.64 at a later time.

A safety factor assessment (Attachment 8) was performed on selected cross sections of the Slurry 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 calculated "Maximum Normal Storage Pool" level (i.e., 19.6 ft NGVD29) and "Maximum Surcharge Pool" level (i.e., 35.4 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 the "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions are 1.50 and 1.40, respectively. Additionally, seismic slope stability with a minimum required safety factor of 1.00 was also evaluated for the Slurry Pond perimeter dikes. The safety factor assessment results indicated that the selected cross sections of the perimeter dikes met the minimum required safety factors provided in §257.73(e)(1) of the CCR Rule. It is noted that the safety factor considering post-liquefaction conditions of the dike fill was not evaluated in this Safety Factor Assessment Report, because the dike fill 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 the Slurry 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 (CCR): Slurry Pond 3&4 (Slurry Pond), West Ash Pond, Unit 2 Slurry Pond, Ash Pond A, Ash Pond B, and the South Ash Pond.

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR Parts 257 and 261, regulating on 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 safety factor criteria for existing CCR surface impoundments.

The Slurry Pond is situated west of the power block. It manages CCR in the form of flue gas desulfurization (FGD) residuals as well as process water resulted from power generating activities. The Slurry Pond is considered as an existing surface impoundment under the CCR Rule. The *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond* (Safety Factor Assessment Report) has been prepared by Geosyntec Consultants (Geosyntec) on behalf of Santee Cooper to demonstrate that the Slurry Pond meets the criteria for periodic safety factor assessment in accordance with §257.73(e) of the CCR Rule.

1.2 Project Site and Construction History

The Slurry Pond was commissioned in 1980. It has a surface area of approximately 106 acres and is located in the northwest corner of the Site. The Slurry Pond is an unlined surface impoundment and designated to receive FGD residuals that do not meet minimum specifications for beneficial use as wallboard-grade gypsum. It also receives process water resulted from the power generating activities and stormwater runoffs from the Limestone Slurry/Ball Mill area and Coal Pile (generally from the west half of the Coal Pile). The solids within the sluiced FGD residuals and stormwater runoffs are contained in the Slurry Pond by gravity settling.

The Slurry Pond was constructed by compacting excavated soils from the surface impoundment interior to form the perimeter dikes and the divider dike, which separates the Slurry Pond from the adjacent West Ash Pond (closed) to the southwest. During the initial construction, a finger dike was constructed into the center of the Slurry Pond primarily to allow solids to settle prior to recirculation of the wastewater, but also provided for access, maintenance, and observation of the pond interior. The Slurry Pond perimeter dikes are approximately 30 ft in height in the northern and eastern sections, 26 ft in height in the western section, and 15 ft in height in the southern section (Thomas and Hutton, 2012). The upstream and downstream slopes of the perimeter dikes range from 2 Horizontal to 1 Vertical (2H:1V) to 3H:1V. The dike crest is approximately 12- to 15-ft wide and typically at elevations 37.0 to 39.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) (Thomas and Hutton, 2012). The total storage capacity of the Slurry Pond is approximately 1,700 ac-ft, with 30% capacity remaining in the north portion of the surface impoundment (ARCADIS, 2012). Stormwater runoff from the downstream side slope is collected in a stormwater trench, conveyed to Pump Station No. 2, and then pumped into the Slurry Pond.

Previously, the impounded free water within the Slurry Pond was routed via rim ditches and a series of culverts to the West Ash Pond and subsequently pumped across an existing pipe bridge to the South Ash Pond. Currently, the free water is managed by the Floating Pump Station, which routes discharge from the Slurry Pond to the Discharge Canal. The surface of the West Ash Pond was closed, re-graded, and capped in 2015 to drain stormwater runoff by gravity to the Slurry Pond through two 36-in diameter corrugated High Density Polyethylene (HDPE) culverts and four 22-in diameter corrugated HDPE through the west side of the divider dike. The free water within the Slurry Pond has been lowered to an operating elevation of 19.6 ft NGVD29 by the Floating Pump Station as a part of the seismic risk mitigation project (Geosyntec, 2014).

1.3 Report Organization

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

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

- Geotechnical subsurface investigations performed by Geosyntec and others are presented in Section 3;
- Subsurface conditions, geology, and geotechnical properties are discussed in Section 4;
- Selection of the seismic hazard parameters for WGS and the site response analysis of the Slurry Pond perimeter dikes performed by Geosyntec are presented in Section 5;
- Results of liquefaction potential evaluation conducted by Geosyntec 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 structural stability and safety factor assessments are presented in Section 8.

2. HYDROLOGIC AND HYDRAULIC EVALUATION

2.1 Hydrologic and Hydraulic Analysis

The following section discusses the regulatory framework, the methodology and assumptions, and the results of the hydrologic and hydraulic (H&H) analysis for the Slurry 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."

§257.73(d)(1)(v)(B)(1) states that the spillways must manage the peak discharge from the *"Probable maximum flood (PMF) for a high hazard potential CCR surface impoundment."*. Additionally, §257.73(d)(1)(v)(A) indicates that *"All spillways must be either:*

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

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

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

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

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

The Slurry Pond does not have a traditional spillway for discharge of free water from the pond interior. The Slurry Pond has the Floating Pump Station to carry sustained flows and to manage the discharge from the surface impoundment. The Slurry Pond also contains a culvert that hydraulically connects and combines the storage capacity of the Slurry Pond with the West Ash Pond. The spillway referenced within this Safety Factor Assessment Report is the Floating Pump Station. The Inflow Design Flood (IDF) was selected as the PMF in accordance with the CCR Rule because the Slurry Pond has been assigned a "High Hazard Potential" classification (Geosyntec, 2016). H&H analysis were performed to demonstrate that the Slurry Pond spillway is able to adequately manage flow during and following the 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(e)(1)(ii).

2.1.2 Methodology and Assumptions

Details of the H&H analysis are provided in a calculation package titled *"Hydrologic and Hydraulic Analysis for Slurry Pond 3&4 and West Ash Pond"*, which is provided 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 Slurry Pond.

The Floating Pump Station manages and maintains the free water at an elevation of 19.6 ft NGVD29 during normal operating conditions by two (2) Tsurumi GSZ-4-45-4 submersible pumps. The capacity of these two pumps operating in parallel is 3,075 gallons per minute (gpm) at the maximum head, normal pool operating elevation when pumping directly to the Discharge Canal. The Floating Pump Station is held in the north corner of the Slurry Pond by guy wires and typically conveys free water to Pump Station No. 1 and ultimately to the Discharge Canal. The piping from the Floating Pump Station can be partitioned such that it bypasses Pump Station No. 1 and is conveyed directly to the Discharge Canal. The Slurry Pond is hydraulically connected to the 64-acre West Ash Pond, which drains surface water runoff to the Slurry Pond through two (2) 36-in diameter corrugated HDPE culverts (invert: 26.0 ft NGVD29), four (4) 22-in diameter corrugated HDPE culverts (invert: 33.7 ft NGVD29), and a 200-ft wide emergency spillway (invert: 36.25 ft NGVD29). The Slurry Pond also receives a base flow of 2,880,000 gallons per day (gpd) (4.46 ft^3/s) of process water when all four electric generating units at WGS are operating.

HydroCAD® (HydroCAD, 2011) software was utilized to apply the Soil Conservation Service (SCS) Technical Release 20 (TR-20) method (SCS, 1982) to compute stormwater volume and to model the performance of the hydraulic structures of the Slurry Pond during the PMF. The PMF was conservatively selected as the 72-hour (hr) duration precipitation event resulting in a rainfall depth of 53.0 inches (NOAA, 1978) and modeled within HydroCAD® using a SCS Type III rainfall distribution. The analysis was performed under the following assumptions, which were confirmed by WGS personnel:

- The Floating Pump Station was assumed to cease pumping due to temporary loss of power during the PMF;
- Stormwater runoff from the Coal Pile will be routed to the South Ash Pond instead of the Slurry Pond during the PMF;
- Pump Station No. 2 located just north of the Slurry Pond will lose power or be switched off during the PMF and does not route stormwater into the Slurry Pond;
- The process water flow will be routed into the Slurry Pond during the PMF; and
- The maximum normal operating pool within the Slurry Pond is maintained at 19.6 ft NGVD29.

2.1.3 Analysis Results

Under the conditions and assumptions described in Section 2.1.2, the maximum surface water level or "maximum surcharge pool" level during and following the PMF event was computed as 35.37 ft NGVD29 (rounded herein to 35.4 ft NGVD29) occurring 47.9 hours into the rainfall event.

3. GEOTECHNICAL SUBSURFACE INVESTIGATIONS

This section summarizes the historical and recent subsurface investigation programs. Two subsurface investigations (S&ME, 1978; PCRA 1999) were conducted historically within the vicinity of the Slurry Pond at WGS. More recently in the spring of 2013, Geosyntec conducted a supplementary, focused subsurface investigation program to obtain geotechnical data for global seismic stability analyses requested by the USEPA (Geosyntec, 2013). After completion of the spring 2013 subsurface investigation, Geosyntec returned to the Site in the fall of 2013 and Spring of 2016 and performed additional geotechnical subsurface investigations within the Slurry Pond. Figure 3 presents the locations of soil test borings, Cone Penetration Test (CPT) soundings, and test pit excavations performed during these subsurface investigation programs. In 2014, Geosyntec installed temporary piezometers in the Slurry Pond to monitor the phreatic surface within the perimeter dikes during drawdown of the free water within the surface impoundment.

The geotechnical data obtained during the historical and recent subsurface investigations, including soil test borings, CPT soundings, and laboratory tests, are included in Attachments 2, 3, and 4. The interpretation of the subsurface stratigraphy and materials properties is presented in Attachment 5. The following sections provide summaries of each of the subsurface investigations.

3.1 <u>Historical Investigations</u>

3.1.1 Soil and Materials Engineers (S&ME) Investigation

In 1977 and 1978, Soil and Materials Engineers, Inc. (S&ME) performed a general subsurface investigation in support of the design and construction of the Slurry Pond, the West Ash Pond, the South Ash Pond, and the Unit 2 Slurry Pond. The subsurface investigation included soil test borings with Standard Penetration Tests (SPTs) generally at 5-ft depth intervals and test pits excavated 10 to 15 ft below ground surface (bgs). Twenty borings and fifteen test pit excavations were performed within the footprint of the Slurry Pond and the West Ash Pond. These boring logs and test pit logs were presented in the subsurface investigation report prepared by S&ME (1978). The boring logs pertaining to the Slurry Pond are provided in Attachment 2-A of this Safety Factor Assessment Report. The S&ME report did not provide details of the subsurface investigation that are relevant for liquefaction potential evaluation and in-situ testing interpretation, such as the drilling method and SPT hammer calibration (i.e., energy ratio).

During the S&ME investigation, representative samples were collected from SPTs by means of a standard split spoon sampler or from test pit excavations as bulk samples. Additionally, thin-walled Shelby tube samples were collected for index and triaxial strength testing. Overall, the laboratory program for this subsurface investigation consisted of index testing (grain size distribution and Atterberg Limit tests), unit weight, compaction testing, consolidation testing, and shear strength testing. Recompacted samples were also tested to evaluate the design shear strength properties of the dike structure to be constructed.

3.1.2 Paul C. Rizzo and Associates (PCRA) Investigation

In 1999, Paul C. Rizzo and Associates, Inc. (PCRA) conducted a geotechnical and hydrogeologic subsurface investigation at WGS primarily through the perimeter dikes of the Slurry Pond and Unit 2 Slurry Pond. The intent of this subsurface investigation was to evaluate the subsurface and hydrogeologic conditions in the vicinity of these surface impoundments. Eighteen borings were advanced through the perimeter dike centerline of the Slurry Pond and the centerline of the divider dike bounding the Slurry Pond and the West Ash Pond. One temporary piezometer was installed near the downstream toe of the perimeter dike of the Slurry Pond. These soil test borings were reported to have been performed using the mud rotary wash drilling method and a side discharge bit.

Two drilling subcontractors were utilized during this subsurface investigation: Carolina Drilling and Mid-Atlantic Drilling, Inc. (MAD). Carolina Drilling, in general, continuously collected samples with SPT blow counts recorded until refusal was encountered at "limestone" (geologic interpretation of this stratum is discussed in Section 4.2.1). Once "limestone" was encountered, a casing was installed to allow rock coring of the "limestone" layer. Borings advanced by Carolina Drilling were generally terminated once the underlying "Black Mingo Formation" (geologic interpretation of this stratum is also discussed in Section 4.2.1) was encountered, but a few of these borings penetrated into the upper 5 to 10 ft of this stiff to very hard clay formation. MAD advanced borings without sampling (i.e., without SPT measurements) to locate the top of "limestone." These borings were generally terminated when the "Black Mingo Formation" stratum was reached. In a few cases, however, MAD cored the "limestone" layer and penetrated nearly 10 ft into the "Black Mingo Formation" (PCRA, 1999). The boring logs from this exploration that are pertinent to the material property interpretation and liquefaction potential analyses are presented in Attachment 2-B of this Safety Factor Assessment Report.

PCRA performed a limited geotechnical testing program consisting of index testing to characterize the hydraulic behavior of the soil; nine of which were located in the Slurry Pond.

3.2 Geosyntec Investigations

3.2.1 Spring 2013 Subsurface Investigation

From February 21st to March 6th, 2013, Geosyntec conducted a geotechnical subsurface investigation in the Slurry Pond and the West Ash Pond areas, which included eleven mud rotary soil test borings, two hollow stem auger borings, fifty-three CPT soundings, and three test pit excavations in the vicinity of the Slurry Pond. Soil Consultants, Inc. (SCI) of Charleston, SC was subcontracted to advance the soil test borings using the mud rotary method. Eight soil test borings were advanced through the Slurry Pond perimeter dike centerline and three soil test borings were advanced near the downstream toe of the perimeter dikes. These borings were terminated when "Chicora Member" (geologic interpretation of this stratum is discussed in Section 4.2.1) was encountered. Refusal was encountered when the paddle, side discharge drill bit encountered the partially cemented "Chicora Member" stratum and SPT blow counts exceeded 50 blows per foot. In one boring (GSB-4), the split spoon penetrated through the "Chicora Member" stratum and entered the underlying "Williamsburg Formation Clay" (geologic interpretation of this stratum is discussed in Section 4.2.1). The boring was advanced an additional 10 ft after encountering the "Chicora Member" before the boring was terminated. During these soil test borings, representative samples were collected by SPT with a standard split spoon sampler typically in 5-ft depth intervals. In addition to the eleven soil test borings, two hollow stem auger borings were also advanced to 25 to 30 ft bgs through the dike centerline to measure a 24-hr water level within the dike fill without the influence of bentonite slurry within the borehole. Soil samples were not collected during these hollow stem auger borings. The boring logs describing these borings are provided in Attachment 2-C.

The two hollow stem auger borings were used to measure the phreatic surface at the dike centerline. These boreholes were left open for two to three days prior to abandonment with cement-bentonite grout. The depths to water levels were measured and recorded after 1 hour and 24 hours after the completion of drilling. At the time of the 2013 subsurface investigation, the surface water level within the Slurry Pond had been maintained at approximately 34.3 ft NGVD29 for several years.

Twenty-three CPT soundings with pore pressure measurements were performed through the centerlines of the perimeter, divider, and finger dikes of the Slurry Pond. An additional twenty-five CPT soundings were performed at the toe of the perimeter dike of this surface impoundment. Four CPT soundings were advanced within the Slurry Pond, which were terminated when the tip resistance and friction sleeve signatures indicated a material transition from CCRs to the natural soils beneath the pond. The CPT sounding logs for the Slurry Pond area are presented within Attachment 3.

In-situ tests were performed in conjunction with selected CPT soundings. The shear wave velocity (V_S) of the subsurface materials was measured and recorded in 5-ft depth intervals at four locations along the dike centerline: two locations at the downstream toe of the perimeter dike of the Slurry Pond. Additionally, porewater pressure dissipation tests were performed at different depths for several CPT soundings along and within the dike centerline, downstream dike toe, and interior of the Slurry Pond. Attachments 3-B and 3-C provide the original shear wave velocity and porewater pressure dissipation test data, respectively.

Three test pits were excavated to a depth of 5-ft bgs within the Slurry Pond at locations that are approximately 60 ft from the finger dike. The excavation was made using a long stick excavator operated by a Santee Cooper construction crew. One test pit was excavated on the western side of the finger dike; the other two test pits were excavated on the eastern side of the finger dike. Bulk samples were collected in 5-gallon buckets and transported to a laboratory for testing.

3.2.2 Fall 2013 Subsurface Investigation

In October and November 2013, Geosyntec mobilized to WGS to collect geotechnical subsurface data through additional soil test borings and CPT soundings in support of evaluating preliminary and conceptual closure alternatives for each CCR surface impoundment at WGS. The subsurface investigation was focused in the vicinity of the South Ash Pond, Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B. However, five soil test borings advanced by mud rotary wash drilling method and six CPT soundings were advanced within the vicinity of the Slurry Pond. The purposes of these investigation locations were to: (i) characterize impounded CCRs; (ii) investigate materials underlying the "Chicora Member" stratum; and (iii) investigate the perimeter dikes near GSB-11 where the dike material was observed to have SPT blow counts of weight of hammer (i.e., zero blow counts) during the Spring 2013 investigation. One boring was advanced to a depth of 100 ft bgs to evaluate the material properties of the Williamsburg Formation Clay. In this soil test borings, SCI switched to a tri-cone rotary wash drill bit once the Chicora Member was encountered to reach the target depth. The remaining four soil test borings were advanced 20 to 25 ft bgs and were utilized to collect Shelby tube samples of CCRs. Three CPT soundings were advanced within the interior, and two V_S profiles were collected within the Slurry Pond. The remaining three CPT soundings were advanced on both sides of GSB-11 to identify if the observed weight of hammer material within the dike fill soils was located in an isolated area.

3.2.3 Spring 2016 Subsurface Investigation

In February and March 2016, Geosyntec mobilized to WGS to further investigate subsurface conditions supporting the design of potential CCR landfills within the footprints of the Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B. Additional soil test borings were also advanced in the western half of the South Ash Pond to further evaluate engineering properties of the soft clays encountered within the area during the Spring 2013 investigation. One additional soil boring was advanced by Geosyntec adjacent to the historical boring GSB-11, which was completed by Geosyntec in 2013 using mud rotary drilling techniques. Except when Shelby tubes were pushed to collect undisturbed soil samples, the soil test boring (GSB-11A) was advanced continuously with SPTs through the perimeter dike soil (i.e., upper 30 ft bgs) and at 5-ft intervals thereafter. The depth to water level was measured as 8.2 ft bgs prior to borehole abandonment with cement-bentonite grout. Since the measured groundwater depth was observed to be inconsistent with historical piezometer measurements, an engineer with Santee Cooper supervised the advancement of a Geoprobe® boring (without sampling) to a depth of 30 ft bgs and the installation of a temporary standpipe. After 96-hrs, the depth to water within the standpipe was measured as 24.4 ft bgs before the instrument was pulled and abandoned.

3.2.4 Calibration Borings

During the Spring 2013 subsurface investigation described in Section 3.2.1, Geosyntec defined two borings as "calibration borings" for two purposes: (i) to directly compare soil test boring data with CPT sounding data; and (ii) to compare Geosyntec boring data with either S&ME (1978) or PCRA (1999) boring data. One calibration boring was located along the dike centerline directly adjacent to a PCRA boring. The other calibration boring was located along the downstream toe of the dike directly adjacent to an S&ME boring. A CPT sounding was also located adjacent to each calibration boring. As mentioned above, the drilling method (i.e., mud rotary or hollow stem) and SPT hammer calibration were not discussed in the S&ME's report (S&ME, 1978). Likewise, the SPT-hammer calibration information was not provided in the PCRA (1999) report. Geosyntec utilized these calibration borings to make reasonable assumptions for material parameter selection (i.e., energy ratio calibration) to apply into the liquefaction potential evaluation for historical borings.

3.2.5 Laboratory Testing

For both subsurface investigations performed in 2013, Geosyntec subcontracted Excel Geotechnical Testing, Inc. (EGT) of Roswell, Georgia to conduct a geotechnical laboratory testing program on representative disturbed (i.e., bulk or split spoon) sample and undisturbed (i.e., Shelby tube) samples. During the Spring 2013 investigation, the testing program on subsurface soils included grain size distribution tests (some with hydrometer tests), Atterberg limits tests, natural water content tests, shear strength tests (2- to 3-point consolidated-undrained [CU] triaxial tests), and unit weight tests. Additionally, grain size distribution, Atterberg limits, and unit weight tests were performed on the CCR samples collected from the interior the Slurry Pond. Two CU triaxial tests were also performed on thin-walled Shelby tube samples of impounded fly ash collected from the West Ash Pond.

During the Fall 2013 subsurface investigation, the laboratory program consisted of natural water content tests, grain size distribution tests, Atterberg Limits, fines content, and specific gravity tests predominantly performed on CCR. Additionally, one-dimensional consolidation tests FGD residuals and Williamsburg Formation Clay. Three CU tests (2- to 3-point) were conducted FGD residuals. One hydraulic conductivity test was also performed on the Williamsburg Formation Clay as a part of this investigation.

During the 2016 subsurface investigation, a grain size distribution, an Atterberg limits, and a CU test were performed on a sample of dike fill soil collected 19 ft bgs from GSB-11A. Additionally, natural moisture content determinations were performed on each of the soil samples from GSB-11A.

Laboratory testing results from each subsurface investigation are provided in Attachment 4.

3.2.6 Piezometer Installation

On 21 and 22 June 2014, Geosyntec mobilized to WGS with Grant Drilling, Inc. to install four temporary piezometers in the vicinity of the Slurry Pond to measure the phreatic surface within CCRs and perimeter dike during the controlled drawdown of the free water within the surface impoundment. Two (2) 1-inch diameter PVC piezometers were installed within the northern perimeter dikes. These two temporary piezometers were screened between 30 and 35 ft bgs and terminated at 35 ft bgs. Meanwhile, a 1-inch diameter PVC piezometer was installed within FGD residuals to the north and south of the Slurry Pond finger dikes. These temporary piezometers were screened between 20 to 25 ft bgs and terminated at 25 ft.

4. SUBSURFACE CONDITIONS AND GEOTECHNICAL PROPERTIES

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

4.1 Regional Geology

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

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

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

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

4.2 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 more recent geotechnical investigations and from regional geologic data. The information indicates that the subsurface soils primarily consist of four geotechnical units, within the depth of interest for the analyses presented in this Safety Factor Assessment Report. A brief description on each unit is presented as follows:

- Dike Fill: Materials within the dike consist predominantly of loose to medium dense, brown to gray, silty/clayey fine to medium sands, and stiff, sandy clays to low to medium plasticity clays. The dike fill is located approximately between elevations 37 to 40 ft NGVD29 (dike crest) and 10 to 19 ft NGVD29 (dike toe).

- Foundation Soils: Foundation soils encountered during the borings onsite consist typically of brown to gray, clayey sands, silty sands, and poorly graded fine to medium sands with varying amounts of shells. Several clay lenses or thin layers of clay were occasionally encountered in more sandy foundation soils. The relative density of the foundation soils ranged from very loose to medium dense. The foundation soils were generally found to be 10 to 30 ft thick. In the majority of borings, the lower 5 to 10 ft of foundation soils consist heavily of shell fragments and shells. This layer was described in the field typically as loose to medium dense, clayey fine sand with interbedded shells.

- 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. Blow counts in this layer exceeded 50 blows over less than 6 in. of advancement with minimal sample recovery. Historical borings indicated that the thickness of this layer typically ranged from 1 to 10 ft, and was fairly thin in the eastern corner of the Slurry Pond. The PCRA report (1999) referred to this layer as "limestone" or "shell hash – Coquina – limestone". The S&ME report (2001) described this layer as "Coquina", a local name used to describe shell hash and partially cemented shells. Doar (2012) indicated this layer could be considered to be Shell Hash and utilized the term "Chicora". Due to limited samples recovered from Geosyntec's subsurface investigations, Geosyntec primarily relied on the review of the literature listed above and information from previous subsurface investigations and the regional geology for the geologic interpretation of this layer. Based on these sources, this layer is defined as "Chicora" or "Chicora Member of the Williamsburg Formation". The term Chicora Member is used to refer to this soil unit throughout Safety Factor Assessment Report.

- Williamsburg Formation Clay: The Williamsburg Formation Clay was encountered beneath the Chicora Member located in the upper reaches of the overall Williamsburg Formation and described as stiff to very hard, dark gray to black, medium to high plasticity clay or silt with sand. The Williamsburg Formation Clay has historically been referred to as "Black Mingo Clay" or the "Black Mingo Formation" at the Site. The term "Williamsburg Formation Clay" is used throughout this Safety Factor Assessment Report.

4.2.2 Water Levels

During the Spring 2013 subsurface investigation, depths to the phreatic surface were measured from each soil test boring located along the dike centerline. CPT Porewater pressure signatures recorded and CPT porewater pressure dissipation test results were also used to evaluate the depths to the phreatic surface at the time of the subsurface investigation. During the 2013 subsurface investigation, the surface water level within the Slurry Pond had been maintained at 34.3 ft NGVD29 for several years. The measured depth to phreatic surface within the perimeter dikes was observed to vary across the perimeter dike. Two generalized phreatic surface elevations through the dike centerlines were selected between 20 and 28 ft NGVD29 to interpret in-situ data for development of material properties. Further discussion of the interpretation of in-situ testing data is provided in Attachment 5.

The phreatic surface through the perimeter dikes of this Safety Factor Assessment Report was developed based on water levels collected from temporary piezometers installed through the Slurry Pond perimeter dike centerline in 2014 and depth to water measurements collected during the subsurface investigation in 2013. Water elevations measured from PPZ-31 and PPZ-32, which are located in the western and northwestern corners of the perimeter dike centerline, were found to range between 12.0 and 16.0 ft NGVD29; while the impoundment was maintained at its previous operating level (34.3 ft NGVD29). As free water was removed from the Slurry Pond, water level elevations measured in temporary piezometers PPZ-31 and PPZ-32 lowered over time. Based on the water level measurements with these piezometers during lowering of the free water, the phreatic surface through the perimeter dike centerline was selected be 12.6 ft NGVD29 for the "Maximum Normal Storage Pool" conditions and 16.0 ft NGVD29 for the "Maximum Surcharge Pool" conditions.

Since PPZ-31 and PPZ-32 are located in the north and west perimeter dikes of the Slurry Pond, phreatic surface for the eastern perimeter dikes was estimated from CPT porewater pressure measurements and from mud rotary and hollow stem auger boreholes. During the "Maximum Normal Storage Pool" conditions, measurements from PPZ-33 were selected to model the phreatic surface within the FGD residuals as 29.5 ft NGVD29; while dike centerline phreatic surface levels were used to select the phreatic surface through the dike fill material on a cross section by cross section or point by point basis.

4.2.3 GSB-11 Evaluation

During the Spring 2013 geotechnical subsurface investigation (Section 3.2.1), Geosyntec observed very soft, high plasticity clays and clayey sands within the perimeter dike fill materials exhibiting "weight-of-hammer" SPT measurements at GSB-11. Subsequently in fall 2013 (Section 3.2.2), CPT-116, CPT-116a, and CPT-117 were advanced within 100-ft of GSB-11 to evaluate the presence and lateral extent of this soft zone. Cone tip resistance (q_t), soil behavior index (I_c), effective friction angle (ϕ'), and undrained shear strength ratio (S_u/σ'_v) were evaluated by methods described in Attachment 5 and compared with other CPTs advanced through perimeter dikes. A weak zone was not observed in CPT-116, CPT-116a, and CPT-117, and the correlated engineering properties were similar to those across the entirety of the perimeter dike with laboratory triaxial strength testing. Based on this information, the "weight of hammer" material observed in GSB-11 was considered to be an isolated area.

As described in Section 3.2.3, Geosyntec remobilized to WGS to investigate the Slurry Pond perimeter dikes adjacent to GSB-11 with a single soil test boring (GSB-11A) and to inspect the condition of the downstream perimeter dikes. The soil test boring encountered Dike Fill material with measured N-values between 2 and 20 blows per foot with a majority of the soil intervals exceeding 6 blows per foot. A CU test on the dike fill material resulted in a $\phi' = 31.7$ degrees and $c' = 0$ psf. Meanwhile, the natural moisture content test results on dike fill soils at GSB-11A ranged generally between 17 percent and 27 percent and were consistent with moisture content test results from other areas of the Slurry Pond perimeter dikes. Elevated moisture content measurements were not identified within the dike fill soils. The water level measured within the temporary standpipe was consistent with adjacent historical piezometers suggesting a phreatic surface below the base of the dike fill material.

4.3 Coal Combustion Residuals (CCR)

As noted in Sections 3.2.1 and 3.2.2, several borings and CPTs have been advanced within the interior of the Slurry Pond. Four soil test borings, three test pit excavations, and seven CPT soundings were performed in the FGD residual materials contained within the Slurry Pond. The surface material was light brown, slightly sandy silt to silty sand. The material closest to the surface appeared to be more oxidized than the residual materials encountered at depth. Deeper slurry residuals were found to be black, saturated, and highly silty.

4.4 Material Parameters

Representative parameters of subsurface materials were selected based on in-situ and laboratory testing results, as discussed in Attachment 5. Additionally, correlations based on in-situ testing methods were applied to supplement laboratory testing, particularly with respect to shear strength testing results. Shear strength parameters were selected from these results, which correspond to the current range of overburden stresses experienced in the vicinity of the perimeter dikes. A summary of the material parameters selected for slope stability analyses are presented in Table 1.

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

5. SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

This section presents the results of seismic hazard evaluation and site response analysis of the Slurry 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 and summarized herein.

5.1 Seismic Hazard Evaluation

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

5.1.1 Seismic Hazard Level

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

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

§257.53 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."

In accordance with the CCR Rule, the analysis presented in this Safety Factor Assessment Report was based on establishing seismic design parameters (e.g., 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 a 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 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 analyses of the Site. Under these assumptions, the subsurface stratigraphy is modeled as a one-dimensional column of soil layers for the analyses. Three representative profiles were developed for the Slurry 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 cyclic shear strains, and maximum cyclic 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 evaluation of liquefaction potential for the Slurry Pond perimeter dike fill and foundation soils directly underlying the 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.

6.1 Regulatory Framework

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

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

The purpose of this section is to discuss the methodology, analysis, and results applied to evaluate if the Slurry Pond dike fill soils are susceptible to liquefaction.

6.2 Methodology

Liquefaction potential analysis was performed based on the Simplified Procedure recommended by Seed and Idriss (1971) and the subsequent update by Idriss and Boulanger (2008). This approach is based on comparing in-situ test results with case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. The analyses presented herein were conducted for both the SPT borings and the CPT soundings performed during the historical (i.e., PCRA 1999 investigation) and the 2013 Geosyntec subsurface investigations, presented in Section 3. The criteria recommended by Bray and Sancio (2006) were applied to evaluate the susceptibility of fine-grained soils to cyclic softening. Nearly 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 perimeter dikes to the downstream dike toe at the time of liquefaction potential analysis was developed based on water levels collected from piezometers installed in June 2014 (Section 4.2.2) and water levels measured from borehole and CPT porewater pressure signatures and porewater pressure dissipation tests. The phreatic surface through the center of the dike was selected to be 16.0 ft

NGVD29 during the design earthquake event for the extent of the perimeter dikes where piezometer data is available and considered representative (i.e., the north and west perimeter dikes). This phreatic surface corresponds to steady-state conditions considering that the maximum surcharge pool (35.4 ft NGVD29) of the Slurry Pond is maintained for a period of time. This assumption was considered conservative as the Floating Pump Station will remove free water within the Slurry Pond to the Maximum Normal Storage Pool (19.6 ft NGVD29).

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 Era). As described in the SCDOT (2010), liquefaction resistance, as modeled by the Cyclic Resistance Ratio (CRR), may be adjusted to account for particle cementation in older soils based on time from deposition (i.e., geologic age) and time from last occurrence of liquefaction (i.e., geotechnical age). As described in Attachment 7, an age correction factor (K_{dr}) of 1.3 was applied for the Pleistocene age soils at the WGS site (typically foundation soils). The transition of dike fill into foundation soils was developed based on the elevations of historical borings conducted prior to construction of the Slurry Pond (i.e., S&ME, 1978).

6.3 <u>Evaluation Results</u>

The factor of safety against liquefaction (FS_{Liq}) was computed at each interval where in-situ data was collected for each soil test boring (2-ft or 5-ft intervals) and each CPT sounding (0.16-ft intervals) advanced through the Slurry Pond perimeter dike crest. FS_{Liq} values computed for embankment or dike soils were found to exceed 1.0 for the conditions described within this Safety Factor Assessment Report. Analysis results for each boring and CPT sounding analyzed are provided as figures within Attachment 7 of this Safety Factor Assessment Report. Based on these analyses, the dike fill or foundation soils directly underlying the Slurry Pond perimeter dikes were not found to be susceptible to liquefaction during the design earthquake, and thus the liquefaction safety factor of the perimeter dike is not required to be evaluated during the periodic safety factor assessment. It is noted that the post-liquefaction conditions of the foundations soils outside the footprint of the Slurry 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.

7. SAFETY FACTOR ASSESSMENT

This section presents the periodic safety factor evaluation for the Slurry Pond perimeter dikes. This evaluation is presented in detail in Attachment 8 and summarized herein.

7.1 Regulatory Framework

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

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

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

7.2 Analysis Models

Subsurface cross sections were developed through the perimeter dikes of the Slurry Pond based on the information obtained from several sources: (i) recent topographic surveys (Thomas and Hutton, 2012); (ii) design grading for the West Ash Pond cover drainage plan; (iii) previous engineering reports and drawings for WGS (Lockwood-Greene, 1978; S&ME, 1978); (iv) subsurface stratigraphy developed based on historical and recent subsurface investigations (Section 4); and v) water level measurements (Section 4.2.2). Five representative cross sections (Cross Sections A through E) were selected (Figure 5) based on geometric and subsurface conditions. Figures 6 through 10 present the locations and geometry of each selected cross section.

7.3 Methodology

7.3.1 Static Slope Stability

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

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

7.3.2 Seismic Slope Stability

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

7.4 <u>Static Safety Factor – Maximum Normal Storage Pool</u>

§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 Slurry Pond is maintained at 19.6 ft NGVD29 and steady state conditions have been established within the perimeter dikes.

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 Slurry Pond is maintained at 35.4 ft NGVD29 and steady state conditions have been established within the perimeter dikes. The maximum surcharge pool elevation of 35.4 ft NGVD29 was computed as the peak water level within the Slurry Pond during and following the PMF rainfall event (Section 2).

7.6 Seismic Safety Factor – Maximum Normal Storage Pool

§257.73(e)(1)(iii) requires that the seismic factor of safety meets or exceeds 1.00 for the maximum normal storage pool conditions within the surface impoundment. The seismic safety factor was evaluated for Cross Sections A through E by applying a seismic horizontal force coefficient of 0.04 to Cross Sections A, B, and C and 0.05 for Cross Sections D and E within SLIDE®. As described in Section 7.4, the Slurry Pond free water level is maintained at 19.6 ft NGVD29 and steady state conditions are expected to have been established within the perimeter dikes. During the evaluation of the Seismic Safety Factor, the undrained shear strength of cohesive materials was reduced by 20 percent to account for the influence of cyclic degradation (Hynes-Griffin and Franklin, 1984).

7.7 Liquefaction Safety Factor – Maximum Normal Storage Pool

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

7.8 Summary of Results

The calculated minimum factors of safety for each analysis case and each of these Cross Sections A through E are summarized in Table 3. Cross Section D was calculated with the minimum factor of safety for the static factor of safety cases; while, Cross Section E was calculated with the minimum factor of safety for the seismic factor of safety case.

The results corresponding to the lowest calculated factor of safety for the three evaluated scenarios are provided in Figures 11 through 13. These results indicate that the perimeter dikes of the Slurry Pond and WGS meet the periodic safety factor assessment criteria required by §257.73(e)(1) of the CCR Rule. Further details of the safety factor assessment for the Slurry Pond can be found in Attachment 8.

8. SUMMARY AND GENERAL CONCLUSIONS

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

- The hydrologic and hydraulic performance of the Slurry Pond during the PMF 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 investigation, and laboratory testing program was carried out to evaluate the construction history, characterize the dike and subsurface soils, and understand the existing conditions of the Slurry 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 probability of exceedance of 2 percent in 50 years as required by the CCR Rule and represents a peak ground motion corresponding to "geologically realistic" conditions. Site response analyses were performed to compute the maximum cyclic shear stresses and maximum horizontal equivalent accelerations, which were applied to evaluate the liquefaction potential and seismic safety factors of the Slurry Pond perimeter dikes.
- The evaluation of liquefaction potential indicated that the dike fill soil and foundation soils directly underlying the Slurry Pond perimeter dikes were not liquefiable and the evaluation of the liquefaction safety factor was not required during the periodic safety factor evaluation. Further evaluation of liquefaction within foundation soils near the downstream perimeter dike toe (i.e., outside the perimeter dike footprint) will be presented at a later time in a subsequent evaluation of "Unstable Areas" in accordance §257.64 for the Slurry Pond.
- Based on the safety factor assessment of five representative cross sections of the Slurry Pond perimeter dikes, the Slurry Pond meets the required safety factors presented in §257.73(e)(1).

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

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TABLES

Table 1. Summary of Selected Geotechnical Material Properties

Material	Total Unit Weight (pcf)[2]	Drained Parameters[4]		Undrained Parameters [4]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill Material	125	33[3]	100	0.65	100
Foundation Materials (Clayey)	100	28	50[4]	0.35 to 0.40 [5]	100[7]
Foundation Materials (Sandy)	115	32	0	-	-
Riprap Buttress	150	45	0	-	-
Chicora Member	130	50	0	-	-
Williamsburg Formation Clay	105	50	0	-	-
FGD Residuals	95	40	0	0.5	0

Notes:

1. pcf = pounds per cubic feet; ϕ' = effective friction angle; c' = cohesion intercept; S_u/σ_v' = undrained shear strength ratio; and $S_{u,\,min}$ = minimum undrained shear strength.
2. Total unit weights presented herein were utilized in the safety factor assessment (Attachment 8). For the liquefaction potential evaluation (Attachment 7), total unit weights were generalized based on the I_c.
3. Dike Fill within Cross Section B was modeled with a ϕ' = 31.7° based on a consolidated undrained (CU) triaxial strength test from GSB-11A. Cross Section B was analyzed to demonstrate adequate safety factors for the perimeter dikes near GSB-11, where soil borings indicated weight of hammer material within the dike fill (Geosyntec, 2013).
4. It was assumed that an effective cohesion intercept up to c' = 50 psf is appropriate for this stratum.
5. Undrained strength parameters for clayey foundation soils were applied for seismic slope stability case only.
6. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the shear strengths of Dike Fill Material, Foundation Soils (Clayey), Foundation Soils (Sandy), and FGD Residuals were reduced during pseudo-static analyses by 20% to account for cyclic degradation during an earthquake. Foundation Materials (Sandy) were modeled with excess porewater pressures (R_u = 0.5) in accordance with recommendations made by Marcuson et al (1990).
7. The minimum undrained shear strength ($S_{u,min}$) was applied for Cross Sections D and E.

Table 2. Summary of Maximum Cyclic Shear Stresses

Profile 1		Profile 2		Profile 3	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	38.9	2.5	37.1	2.5	45.4
7.5	88.7	7.5	88.6	7.5	90.0
12.5	143.8	12.5	131.2	12.5	127.1
17.5	197.1	17.5	178.0	17.5	169.0
22.5	250.7	22.5	226.9	22.5	206.1
27.5	298.8	27.5	263.3	27.5	242.6
32	327.1	32	287.8	32	279.9
36	343.6	36	300.9	36	298.5
40	354.0	40	307.2	40	307.4
44	352.7	44	304.0	44	304.1
48	363.4	48	313.4	48	313.4
52.5	442.2	52.5	375.2	52.5	385.9
60	547.9	60	492.3	60	488.1

Table 3. Summary of Calculated Safety Factors

Factor of Safety Case	**Target FS**	**Cross Section A**	**Cross Section B**	**Cross Section C**	**Cross Section D**	**Cross Section E**
Static FS- Maximum Normal Storage Pool	1.50	1.95	1.95	1.61	*1.51*	1.60
Static FS- Maximum Surcharge Pool	1.40	1.79	1.85	1.61	*1.41*	*1.41*
Seismic FS- Maximum Normal Storage Pool	1.00	1.24	1.24	1.10	1.05	*1.03*
Liquefaction FS[2]	1.20	N/A	N/A	N/A	N/A	N/A

Notes:

1. The liquefaction safety factor was not evaluated since embankment soils were not found to be liquefiable.
2. The lowest computed safety factor for each analysis case was italicized.

FIGURES

SITE LOCATION MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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Figure 1a

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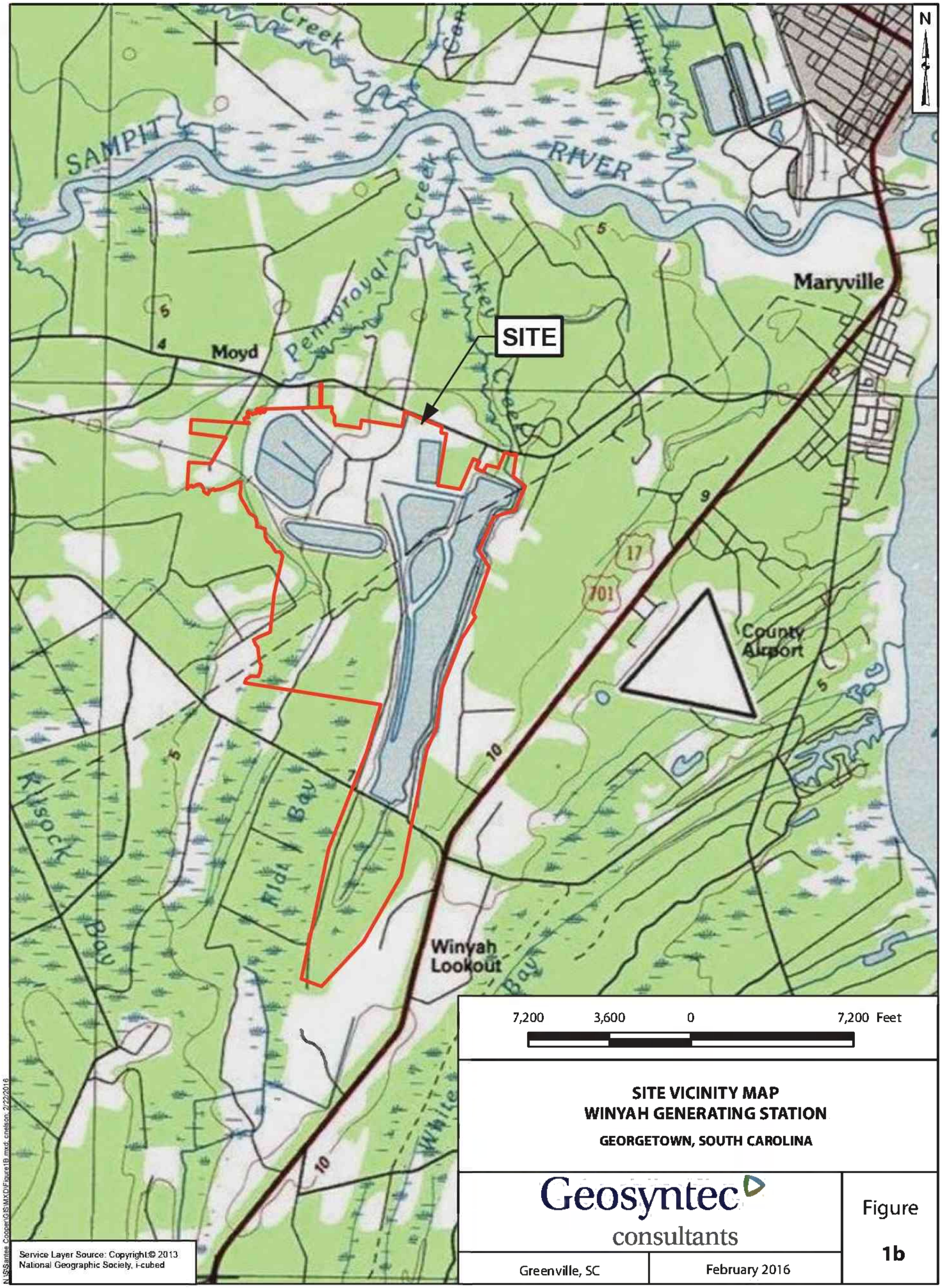

SITE VICINITY MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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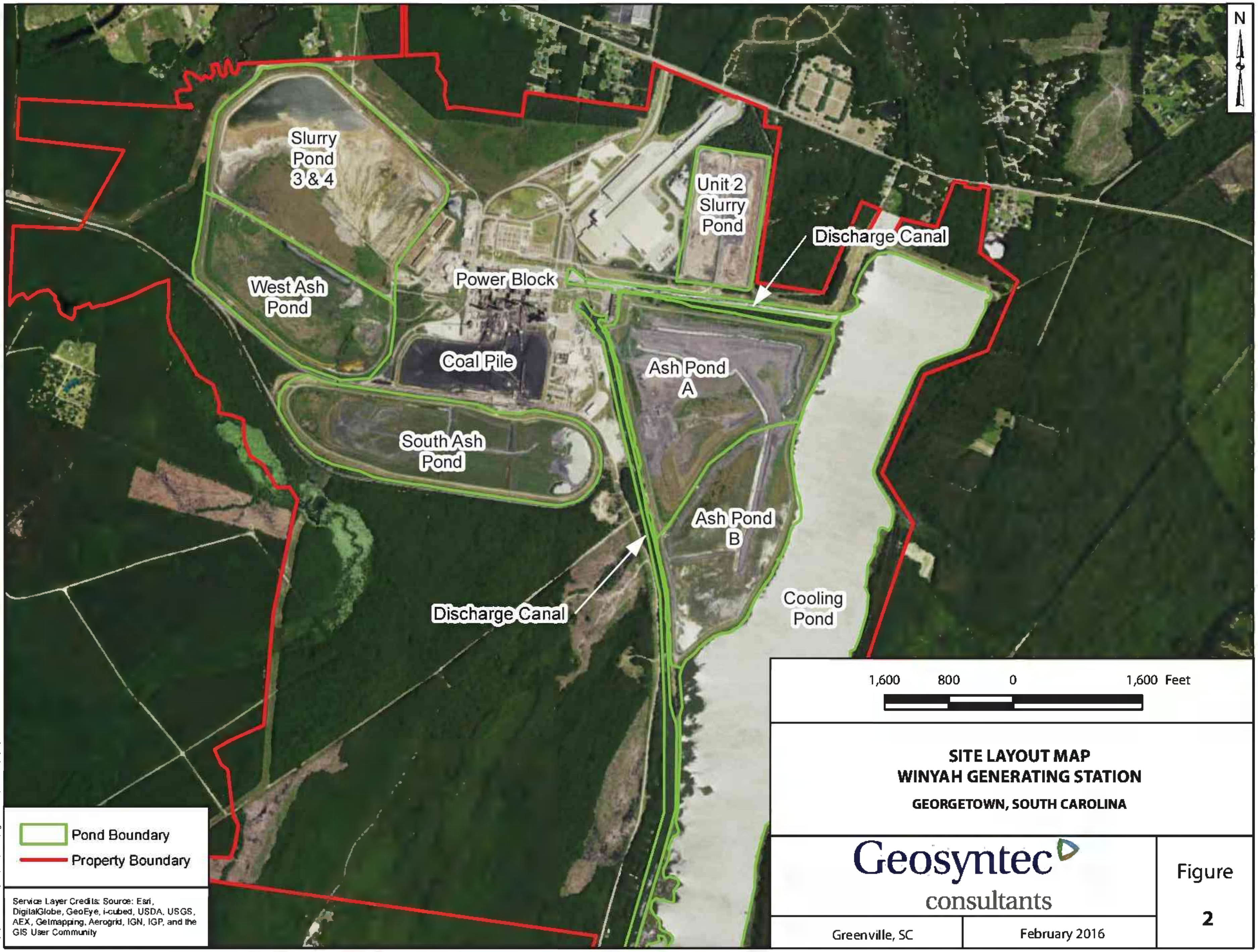
Figure **1b**



SITE LAYOUT MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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Figure 2

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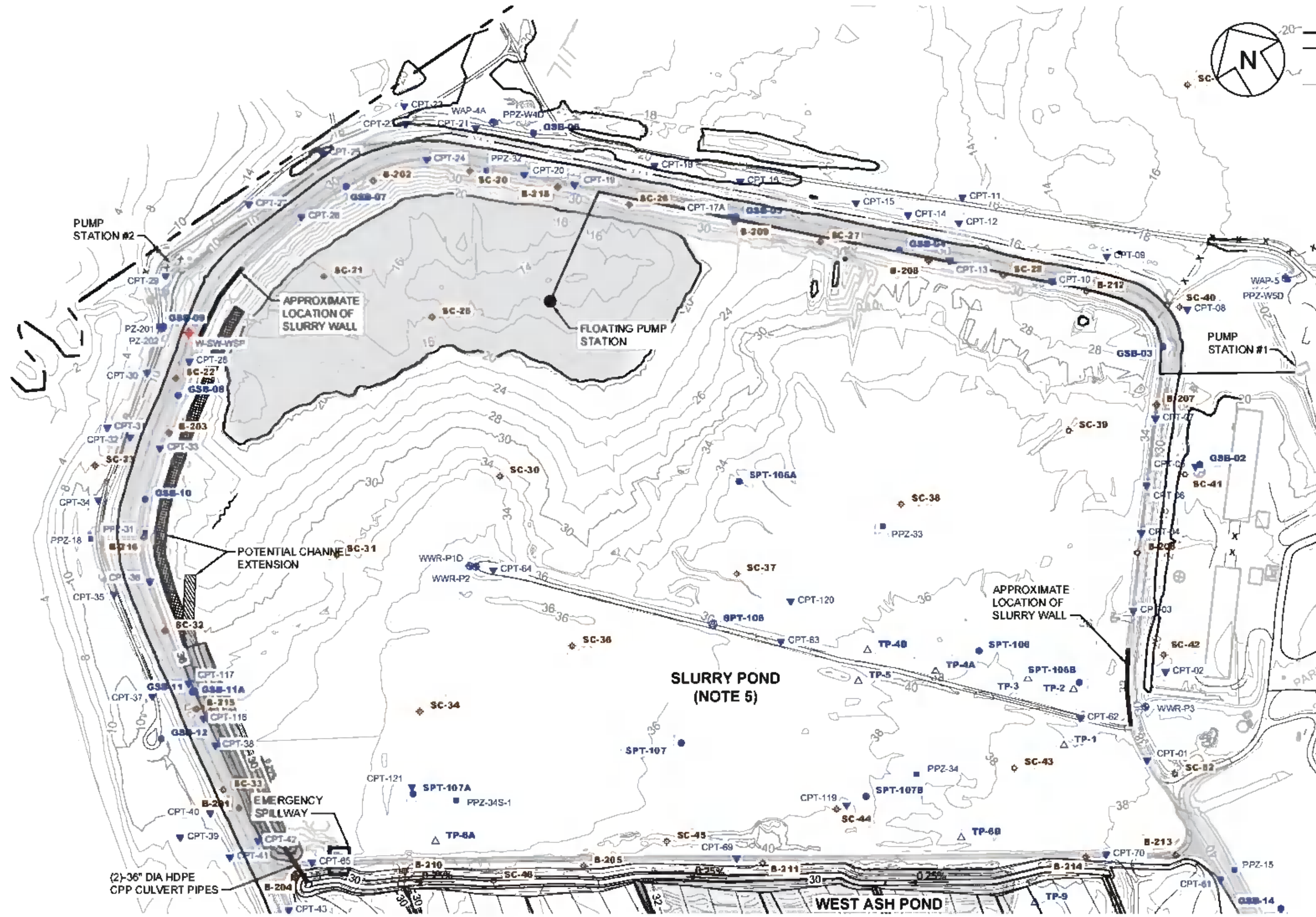

LEGEND

30	DESIGN MAJOR GRADE CONTOUR
10	EXISTING MAJOR GRADE CONTOUR
W-SW-WSP	EXISTING STAFF GAUGE
B-201 SC-20	BORING BY OTHERS
CPT-01	GEOSYNTEC CONE PENETRATION TEST
GSB-02	GEOSYNTEC SOIL BORING
WAP-4A, WWR-P1D	MONITORING WELL
PZ-201, PPZ-15, PPZ-W4D	PIEZOMETER
TP-1	TEST PIT

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. TEMPORARY PZ-31 AND PZ-32 WERE INSTALLED IN JUNE 2014 TO MEASURE THE PHREATIC SURFACE WITHIN THE SLURRY POND PERIMETER DIKES. PZ-31 AND PZ-32 WERE DESTROYED IN OCTOBER 2015 AND JUNE 2013, RESPECTIVELY.
3. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
4. 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.
5. THE FREE WATER LEVEL IN THE SLURRY POND IS MAINTAINED AT AN ELEVATION OF 19.6 FT NATIONAL GEODETIC VERTICAL DATUM OF 1928 (NGVD29) BY A FLOATING PUMP STATION. THE MAXIMUM SURCHARGE POOL WITHIN THE SLURRY POND WAS COMPUTED BASED ON THE INFLOW DESIGN FLOOD (IDF) AS 35.3 FT NGVD29.
6. PZ-201 AND PZ-202 WERE INSTALLED BY PAUL C. RIZZO AND ASSOCIATES (PCRA) IN 1999 AND HAVE SINCE BEEN ABANDONED.

0 300' 600'
SCALE IN FEET

WGS - SLURRY POND
BORING LOCATION MAP

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FIGURE 3

PROJECT NO: GSC5242 | SEPTEMBER 2016

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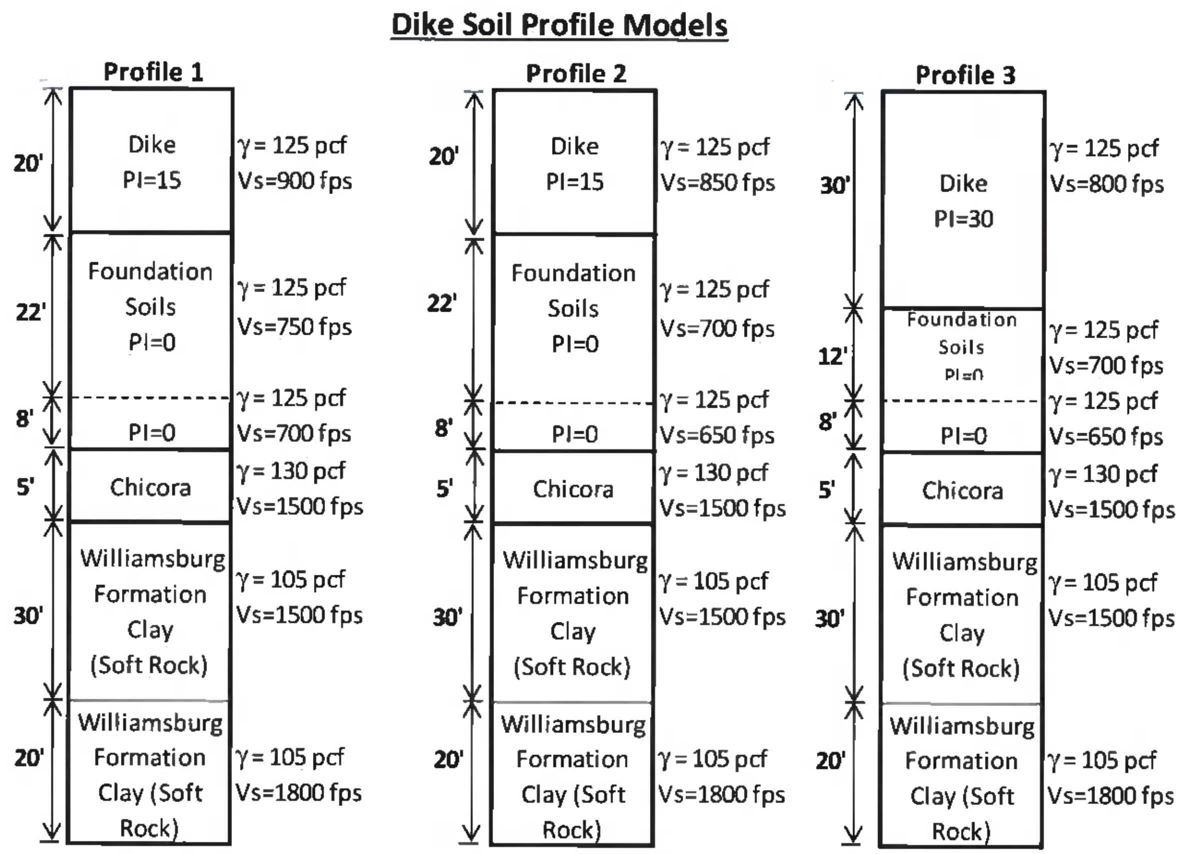
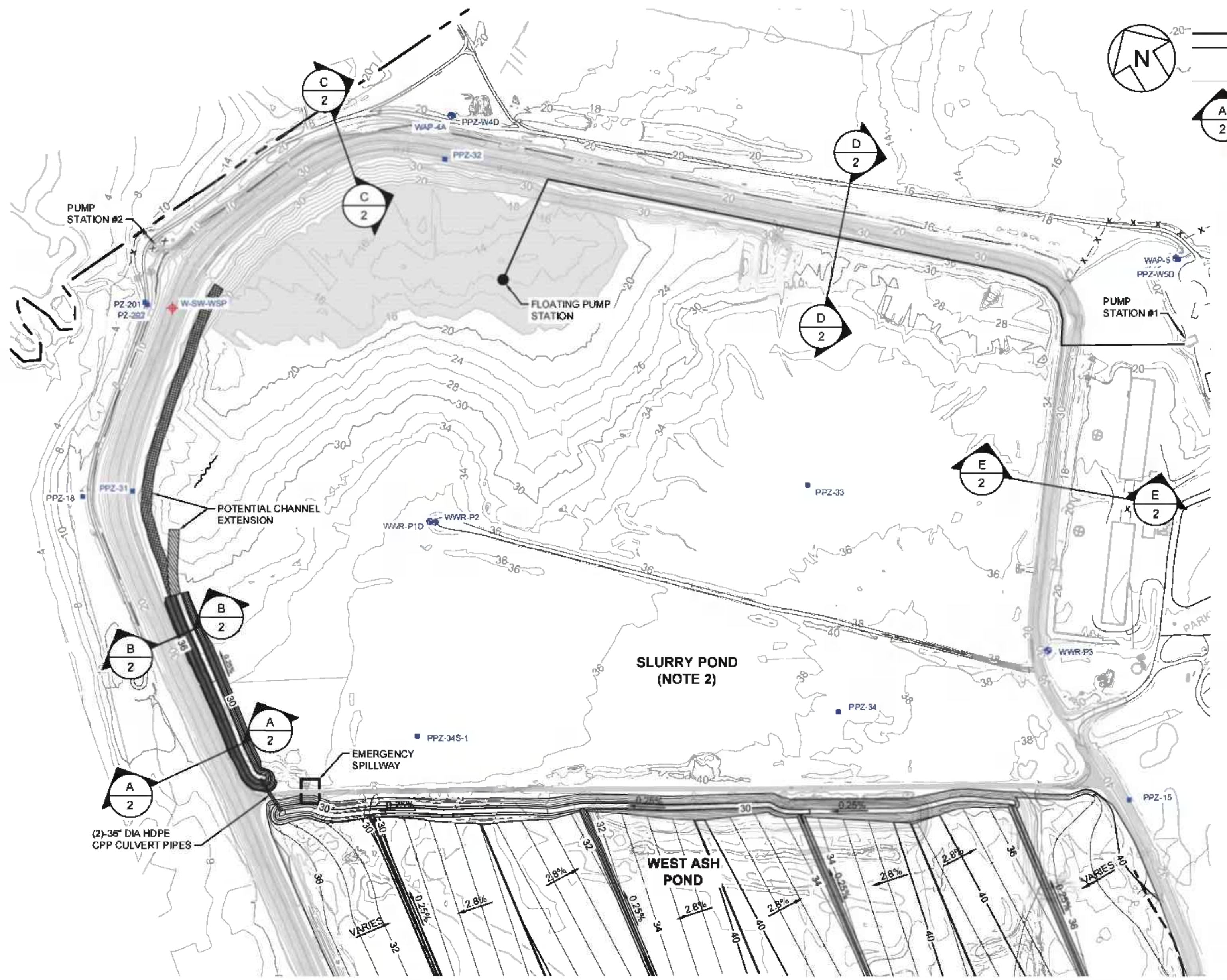


Figure 4. Representative Subsurface Profiles for Site Response Analysis

Note:

1. Development of the representative site response profiles are provided in Appendix 6 of Attachment 6 to this Stability Report.



NOTES:

1. TEMPORARY PZ-31 AND PZ-32 WERE INSTALLED IN JUNE 2014 TO MEASURE THE PHREATIC SURFACE WITHIN THE SLURRY POND PERIMETER DIKES. PZ-31 AND PZ-32 WERE DESTROYED IN OCTOBER 2015 AND JUNE 2013, RESPECTIVELY.
2. THE FREE WATER LEVEL IN THE SLURRY POND IS MAINTAINED AT AN ELEVATION OF 19.6 FT NATIONAL GEODETIC VERTICAL DATUM OF 1928 (NGVD29) BY A FLOATING PUMP STATION. THE MAXIMUM SURCHARGE POOL WITHIN THE SLURRY POND WAS COMPUTED BASED ON THE INFLOW DESIGN FLOOD (IDF) AS 35.3 FT NGVD29.
3. PZ-201 AND PZ-202 WERE INSTALLED BY PAUL C. RIZZO AND ASSOCIATES (PCRA) IN 1999 AND HAVE SINCE BEEN ABANDONED.

WINYAH GENERATING STATION
CROSS SECTION LOCATION MAP

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FIGURE 5

PROJECT NO: GSC5242 | SEPTEMBER 2016

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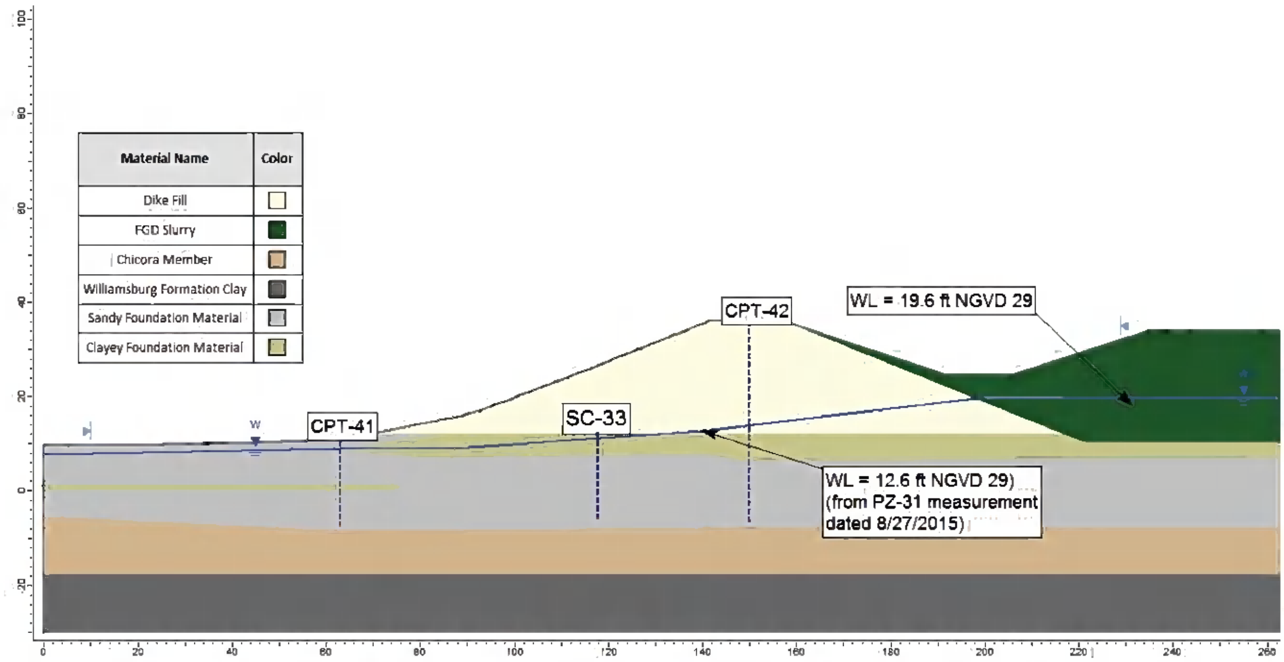

Figure 6. Cross Section A Geometry during Maximum Normal Storage Pool Conditions

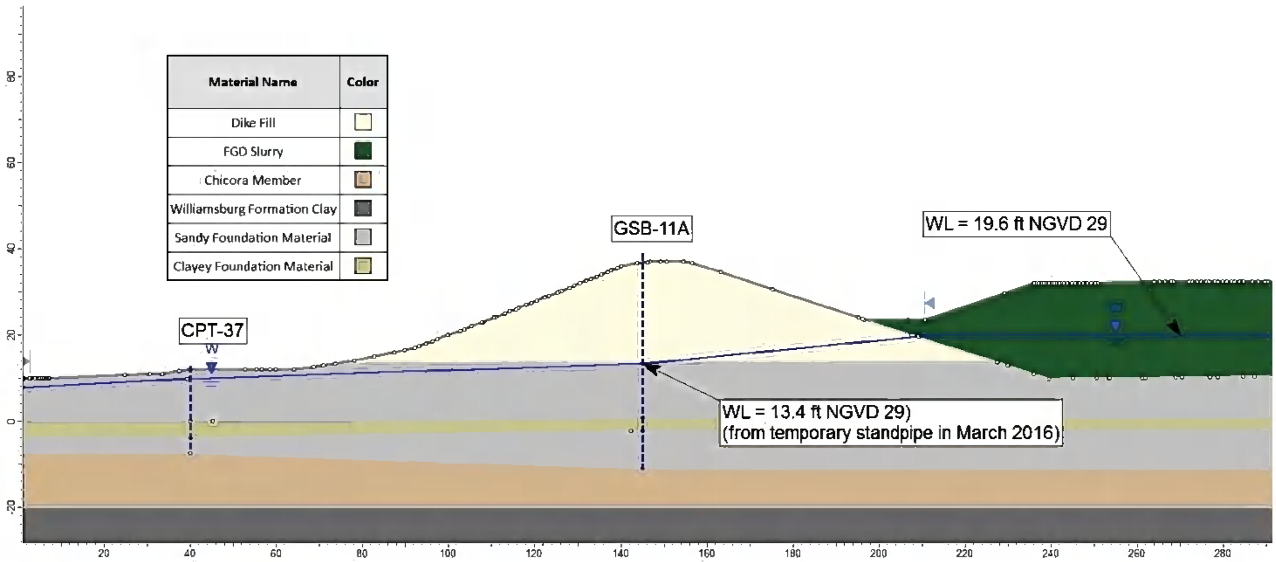

Figure 7. Cross Section B Geometry during Maximum Normal Storage Pool Conditions

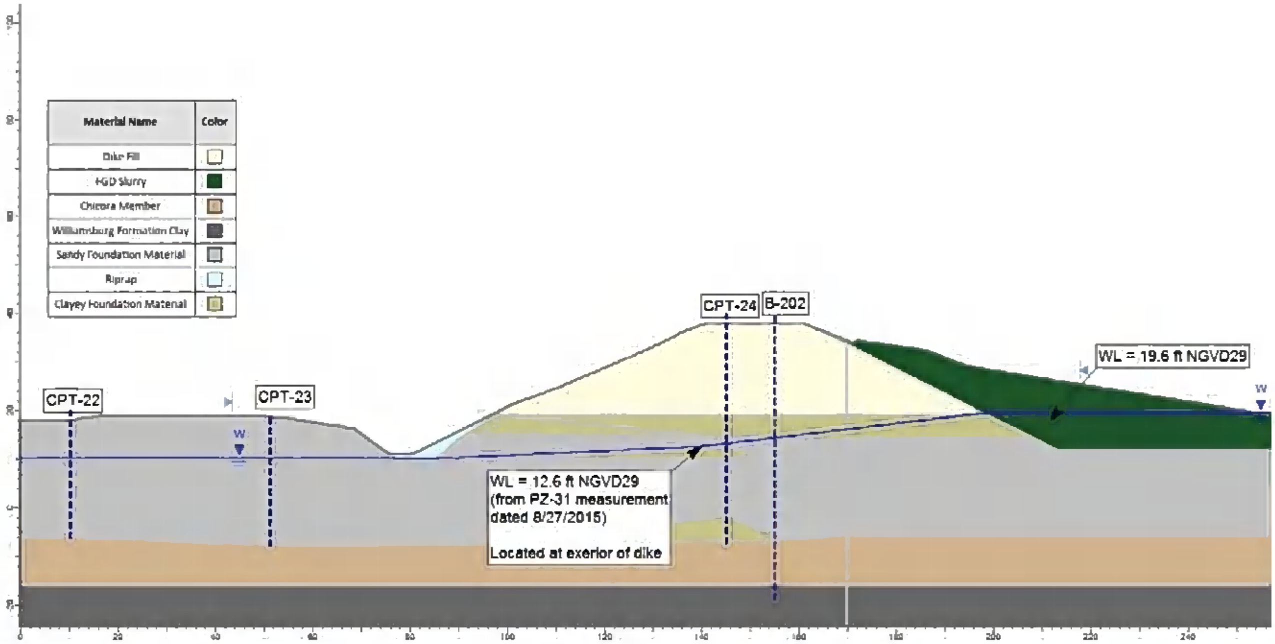

Figure 8. Cross Section C Geometry during Maximum Normal Storage Pool Conditions

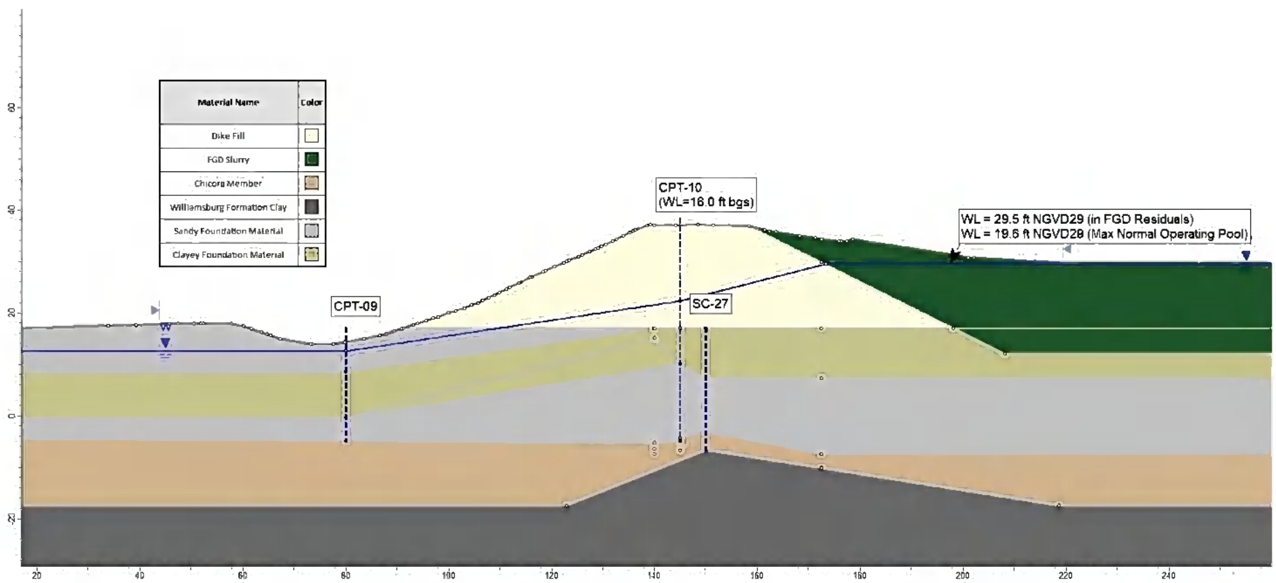

Figure 9. Cross Section D Geometry during Maximum Normal Storage Pool Conditions

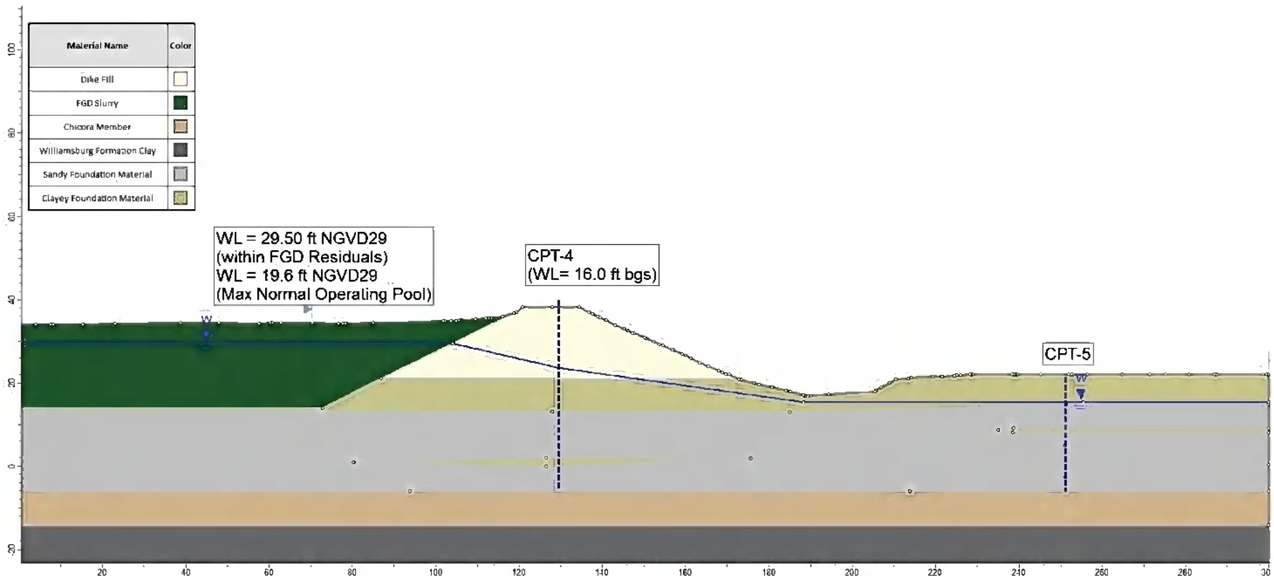

Figure 10. Cross Section E Geometry during Maximum Normal Storage Pool Conditions

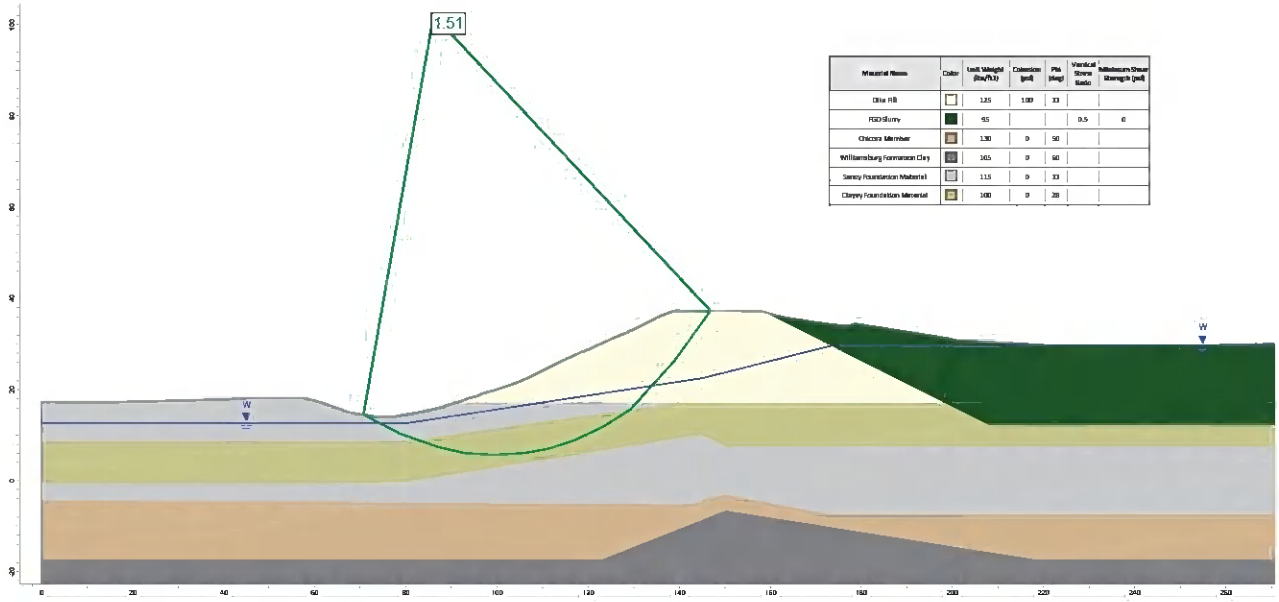

Figure 11. Critical FS for Cross Section D: Static FS - Maximum Normal Storage Pool

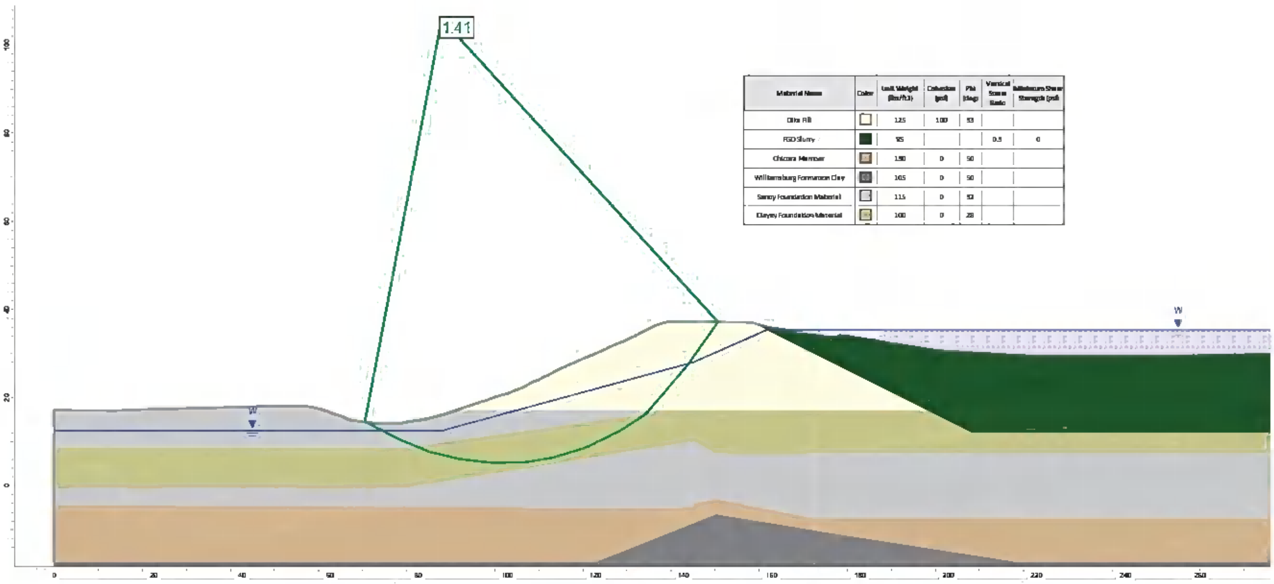

Figure 12. Critical FS for Cross Section D: Static FS - Maximum Surcharge Pool

Notes:

1. The phreatic surface through the Cross Section D under Maximum Surcharge Pool was assumed to be approximately 10 ft below the dike crest, which is consistent with the 24-hr water level measurement from a hollow stem auger boring nearby (HSA-1). During this boring, the water level within the Slurry Pond had been maintained at an elevation of 34.25 ft NGVD29 for several years and was considered similar to the steady state conditions during maximum surcharge pool conditions (35.4 ft NGVD29).

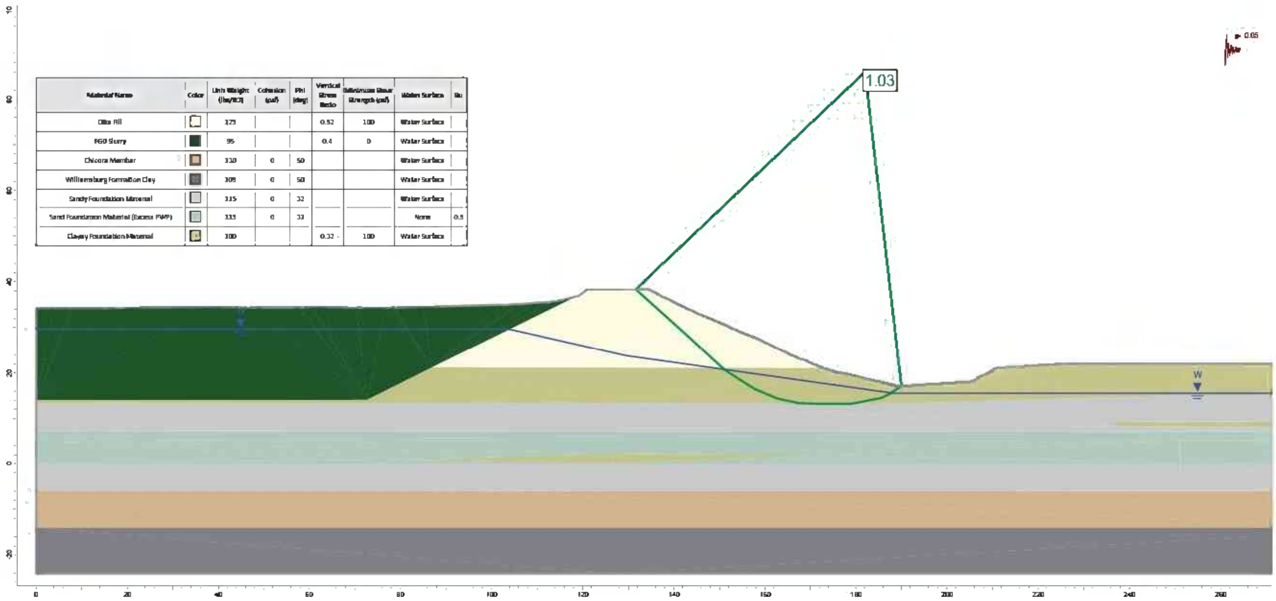

Figure 13. Critical FS for Cross Section E: Seismic FS - Maximum Normal Storage Pool

ATTACHMENT 1

Hydrologic and Hydraulic (H&H) Analysis

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COMPUTATION COVER SHEET

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

Title of Computations: **Hydrologic and Hydraulic Analysis: Slurry Pond 3 and 4**

Computations by:	Signature	[signature]	2/10/16
	Printed Name	Sarah Herr	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (senior reviewer)	Signature	Brianna L. Wallace	10/7/16
	Printed Name	Brianna Wallace	Date
	Title	Senior Engineer	
Computations, Assumptions, and Procedures Checked by: (peer reviewer)	Signature	[signature]	10/7/16
	Printed Name	Hari Parthasarathy	Date
	Title	Senior Staff Engineer	
Computations backchecked by: (originator)	Signature	[signature]	10/7/16
	Printed Name	Sarah Herr	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature	Brianna L. Wallace	10/7/16
	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**

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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

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 Slurry Pond 3 and 4 (Slurry 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 Slurry Pond is regulated by the CCR Rule as an existing CCR surface impoundment. Under the CCR Rule, a high hazard ranking classification is associated with the Probable Maximum Flood (PMF) precipitation event. Since the Slurry Pond is a high hazard surface impoundment, the PMF storm frequency is analyzed herein.

The Slurry Pond, encompassing approximately 107 acres (ac), is situated near the northwest corner of the Site, directly adjacent to Pennyroyal Creek. (Note that 107 ac is the area contained within the dike crest boundary. The area of the limits of CCR is slightly less at approximately 106 ac.) The Slurry Pond is bounded by perimeter dikes which are approximately 30 feet (ft) in height to the north, 26 ft in height to the west, and 18 ft in height to the east (Thomas and Hutton, 2012). The Slurry Pond is separated from the West Ash Pond by a divider dike which forms the southern boundary of the Slurry Pond. Under the CCR Rule, the West Ash Pond is an inactive landfill. A Site Map depicting the Slurry Pond boundary and hydraulic features associated with the Slurry Pond is provided in **Figure 1**.

The Slurry Pond receives Flue Gas Desulfurization (FGD) residuals that do not meet specifications for use as wallboard grade gypsum or which contain gypsum fines discharged from the Dewater Plant. The Slurry Pond also receives process water and stormwater from the Limestone Slurry/Ball Mill area and stormwater from Detention Pond No. 2. A Floating Pump Station, installed in the Slurry Pond in 2015, normally conveys water from the Slurry Pond directly to the Discharge Canal. Piping is valved such that the Floating Pump Station may convey water to the Pump Station No. 1 sump located immediately east of the Slurry Pond. Pump Station No. 1 conveys water to the Discharge Canal. The Discharge Canal is connected to and flows into the Cooling Pond.

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

Stormwater is collected in Detention Ponds No. 1 and No. 2 (shown as catchments 1S, 2S, and 3S and 4S, respectively, in **Appendix A**) located along the outside perimeter of the Slurry Pond. These Detention Ponds were designed to manage the 25 yr, 24 hour (hr) storm event (Santee Cooper, 2004). Pump Station No. 1 (labeled in **Figure 1**) receives water from Detention Pond No. 1, as well as Cooling Tower blowdown, and discharges to the Discharge Canal. Pump Station No. 2 receives water from Detention Pond No. 2 and discharges to the Slurry Pond. Detention Pond No. 2 is equipped with a spillway to Pennyroyal Creek which may only be activated during storm events greater than the 25 yr, 24 hr storm.

The West Ash Pond gravity drains to the Slurry Pond through two (2) 36 inch (in.) diameter culverts. There are also four (4) 22 in. diameter culverts with higher invert elevations and an emergency spillway that hydraulically connects the Slurry Pond and West Ash Pond. The West Ash Pond is not subject to the CCR Rule, because it has not impounded water since before the effective date of the CCR Rule.

METHODOLOGY

Stormwater runoff volumes and associated discharges to the Slurry 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 West Ash Pond and Slurry Pond.

Rainfall

The 72 hr duration precipitation event was used in this analysis. It is the maximum duration precipitation event for which the National Oceanic and Atmospheric Agency (NOAA) provides isopluvial maps for PMF events. The rainfall depth corresponding to the 72 hr duration precipitation event for the PMF frequency return period for the Site is 53.0 in. (NOAA, 1978). The design storm hyetograph was developed using SCS Type III rainfall distribution and was directly input to the *HydroCAD* model.

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

Drainage Areas and Curve Numbers

The contributing watershed areas for the West Ash Pond and Slurry Pond are 64.4 ac and 107.2 ac, respectively. These areas were delineated using the dike crests to correspond to the ponds' direct drainage areas. Each pond was assigned a curve number (CN) based on guidance provided in Technical Release 55 (TR-55) (SCS, 1986) representing the type of ground cover in that area. The West Ash Pond closure plan includes placement of a cover, consisting of seamed geomembrane panels, to limit infiltration into the underlying CCR material. As a result, the West Ash Pond ground cover was assumed to be impervious (CN = 98). Alternatively, Slurry Pond ground cover was assumed to be 90% FGD material and 10% water (Weighted CN = 87) (Santee Cooper, 2013). The contributing watershed areas and CNs are summarized in **Table 1** and were directly input to 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 West Ash Pond is characterized entirely as open channel flow (shown in **Figure 2**). Open channel flow travel times were calculated as:

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

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

L = flow length (ft); and

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

Open channel flow velocities were calculated using Manning's equation. The average velocities were computed assuming bank-full elevation as:

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

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

n = Manning's roughness coefficient;

R = hydraulic radius (ft); and

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

A Manning's roughness coefficient of 0.013 was used to represent open channel flow across the geomembrane cover for the West Ash Pond. The open channels were designed with trapezoidal configurations per Phase I of the West Ash Pond closure plan (Geosyntec, 2015). Channel dimensions are summarized in **Table 2**. The hydraulic radii were computed as:

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

where: R = hydraulic radius (ft);

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

P_w = wetted perimeter (ft).

The cross sectional flow areas were calculated by:

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

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

B = bottom width of the channel (ft);

D = depth of the channel (ft); and

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

The wetted perimeters were calculated by:

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

where: P_w = wetted perimeter (ft);

B = bottom width of the channel (ft);

D = depth of the channel (ft); and

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

The parameters used to describe open channel flow in the West Ash Pond are presented in **Table 3**. The computed times of concentration for the West Ash Pond channels are summarized in **Table 4**.

The flow path from the most remote point within the Slurry Pond is characterized by sheet flow, shallow concentrated flow, and channel flow (shown in **Figure 3**). *HydroCAD* applied the Overton and Meadows formulation to calculate travel time for sheet flow for distances less than 300 ft (NRCS, 2010):

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

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) (ft/ft).

A Manning's roughness coefficient of 0.020 was used to represent sheet flow in the Slurry 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

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frequency storm event is 4.38 in. (NOAA, 2006). The parameters used to model sheet flow within the Slurry Pond are shown in **Table 3**.

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

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

where: T_t = travel time (s);

L = flow length (ft); and

V = average velocity (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 Slurry Pond. The parameters used to describe shallow concentrated flow within the Slurry Pond are presented in **Table 3**.

Open channel flow within the Slurry Pond is characterized using the method previously described for open channel flow within the West Ash Pond. The open channel flow velocities were calculated using Manning's equation. A Manning's roughness coefficient of 0.020 was selected to represent open channel flow across the FGD residuals present within the Slurry Pond. The open channels were designed with trapezoidal configurations per Phase I of the West Ash Pond closure plan (Geosyntec, 2015). Channel dimensions are summarized in **Table 2**. The parameters used to describe open

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channel flow within the Slurry Pond are presented in **Table 3**. The resulting times of concentration for sheet flow, shallow concentrated flow, and open channel flow are presented in **Table 4**.

Inflows

In the *HydroCAD* model, stormwater inflow generated from the West Ash Pond and Slurry Pond is modeled as Sub-Catchments 1S and 2S, respectively. Stormwater inflow generated from Sub-Catchment 1S is routed into Pond 3P (the West Ash Pond), while stormwater inflow generated from Sub-Catchment 2S flows into Pond 4P (the Slurry Pond). The *HydroCAD* model routing diagram is provided in **Appendix B**.

In addition to stormwater inflow, FGD process water is discharged to the Slurry Pond. This base flow is represented by Node 5L in the *HydroCAD* model and contributes to the inflow to Pond 4P, the Slurry Pond. The base process water inflow when all four units are operational is modeled as 2,880,000 gallons per day (gpd) (4.46 cubic feet per second [cfs]) (Santee Cooper, 2013).

The Slurry Pond normally receives flow from Pump Station No. 2. This pump station must be shut down during storm events greater than the 25 yr, 24 hr event to ensure no overtopping of the Slurry Pond. Additionally, piping exists to route flow from the Coal Pile Runoff Pump Station to the Slurry Pond. However, Coal Pile runoff is currently routed to the South Ash Pond. This analysis assumes no inflows from the Coal Pile Runoff Pump Station.

Storage Capacities

In *HydroCAD*, Ponds 3P and 4P model the available storage volumes within the West Ash Pond and Slurry Pond, respectively.

The available stormwater storage volume of the West Ash Pond between elevations 26 ft and 37 ft National Geodetic Vertical Datum of 1929 (NGVD 29) (approximate crest) was calculated by developing an area-volume curve based on the closure design grading plan (Geosyntec, 2015). The lowest available contour within the West Ash Pond is 26 ft NGVD 29. The minimum crest elevation of the West Ash Pond perimeter dikes is 37 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 West Ash Pond between these elevations is 98.9 acre-feet (ac-ft). The area-volume data are presented in **Table 5**.

Similarly, the available stormwater storage volume in the Slurry Pond between elevations 12 ft (pond bottom per bathymetric survey) and 35.67 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 available contour within the Slurry Pond is 12 ft NGVD 29. The minimum crest elevation of the Slurry Pond perimeter dikes is 35.67 ft NGVD 29. The surface area of each contour was measured and tabulated at each elevation. Next, 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 Slurry Pond between these elevations is 510.2 ac-ft. Per the weekly inspection reports, the Slurry Pond is maintained at a normal operational pool elevation of 19.6 ft NGVD 29 by the Floating Pump Station. As a result, the starting elevation of Pond 4P is set to 19.6 ft NGVD 29. The available cumulative storage volume between 19.6 ft and 35.67 ft NGVD 29 is 469.9 ac-ft. The area-volume data are presented in **Table 5**.

<u>Outlet Structures</u>

The outlet structures between the West Ash Pond to the Slurry Pond include two (2) 36 in. diameter smooth interior, corrugated exterior, high density polyethylene (HDPE) pipe culverts with upstream inverts at 25.96 and 25.90 ft NGVD 29 (Thomas and Hutton, 2016). An existing 200 ft wide spillway with an invert elevation of 36.25 ft NGVD 29 is provided in the divider dike (Santee Cooper, 2013). These outlet structures allow water to drain from the West Ash Pond to Slurry Pond. Additionally, four (4) 22 in. diameter HDPE pipe culverts from historical operations (upstream invert elevations at 33.40 ft, 33.25 ft, 33.42 ft, and 33.32 ft NGVD 29) also hydraulically connect the Slurry Pond to the West Ash Pond (Santee Cooper, 2013; Thomas and Hutton, 2016).

The Slurry Pond is equipped with two (2) Tsurumi GSZ-4-45-4 submersible pumps, housed in the Floating Pump Station located over the deepest area of the Slurry Pond. The capacity of these pumps operating in parallel is 3,100 gallons per minute (gpm) at the maximum head, normal pool operating elevation of 19.6 ft NGVD 29 when pumping directly to the Discharge Canal (Geosyntec, 2014). Normally, the Floating Pump Station discharges directly to the Discharge Canal. However, for this analysis, the Floating Pump Station is assumed to be nonoperational due to temporary loss of power during a large storm event.

Since Pump Station No. 1 is designed to manage the 25 yr, 24 hr storm event, it will be inundated during the 100 yr, 24 hr and greater storm events. If the Detention Pond No. 2 spillway is activated (indicating a storm event greater than the 25 yr, 24 hr storm event), Santee Cooper will check the valves on the piping to ensure they are adjusted such that the Floating Pump Station will pump directly to the Discharge Canal. To allow Pump Station No. 1 to be completely dedicated to pumping stormwater flowing during a large storm event, Santee Cooper will pump Units 3 and 4 Cooling Tower blowdown directly to the Discharge Canal to allow Pump Station No. 1 to be fully committed to managing stormwater flows. Santee Cooper will also shut down Pump Station No. 2 that discharges to the Slurry Pond during storm events greater than the 25 yr, 24 hr storm event.

RESULTS

The resulting peak water surface elevations and storage volumes for the PMF storm event are shown in **Table 6**. During this storm event, the West Ash Pond and Slurry Pond will effectively operate as a single pond as the intermediate dike culverts and spillway allow flow between both storage areas. This hydrologic and hydraulic analysis demonstrates that the West Ash Pond and Slurry Pond contain the 72 hr duration precipitation event for the PMF frequency return period given the following assumptions and operating conditions:

- The normal maximum operating pool elevation for the Slurry Pond is 19.6 ft NGVD 29. This allows an additional 6.4 ft of storage capacity for operational flexibility during storm events. The maximum water elevation to maintain seismic stability is 26 ft NGVD 29.
- The Floating Pump Station is not operational due to temporary loss of power during a large storm event.
- Pump Station No. 2, located just north of the Slurry Pond, shall be shut off during storm events greater than the 25 yr, 24 hr storm. Detention Pond No. 2 and Pump

Station No. 2 are designed to manage the 25 yr storm. The Site is permitted to discharge over the Detention Pond No. 2 emergency spillway to Pennyroyal Creek during larger storm events.

- The Coal Pile Runoff Pump Station shall not pump stormwater to the Slurry Pond.
- Activation of the Detention Pond No. 2 spillway shall serve as a trigger for the plant to shut down Pump Station No. 2 to eliminate this stormwater flow from entering the Slurry Pond.

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TABLES

Table 1 – Watershed Areas and Curve Numbers

Drainage Basin	Area (ac)	Weighted Curve Number (--)
West Ash Pond	64.4	98
Slurry Pond	107.2	87

Table 2 – Open Channel Dimensions (Geosyntec, 2015)

Flow Path	Channel Configuration	Side Slope Ratio (H:V) (ft:ft)	Bottom Width of the Channel (ft)	Depth of the Channel (ft)
West Ash Pond				
Channel A	Trapezoidal	3:1	4	3
Channel B	Trapezoidal	3:1	4	3
Channel C	Trapezoidal	3:1	14	3
Slurry Pond				
Channel A	Trapezoidal	3:1	14	4
Channel B	Trapezoidal	3:1	14	4

Table 3 – Input Parameters Describing Open Channel Flow, Sheet Flow, and Shallow Concentrated Flow

Flow Path	Open Channel Flow					Sheet Flow				Shallow Concentrated Flow		
	Cross Sectional Area (sq ft)	*Wetted Perimeter (ft)*	*Channel Slope (ft/ft)*	*Manning's Roughness Coefficient (--)*	*Flow Length (ft)*	*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)*
West Ash Pond												
Channel A	39	23.0	0.0025	0.013	302	--	--	--	--	--	--	--
Channel B	39	23.0	0.0025	0.013	1,022	--	--	--	--	--	--	--
Channel C	69	33.0	0.0025	0.013	2,222	--	--	--	--	--	--	--
Slurry Pond												
Sheet	--	--	--	--	--	0.0005	0.020	100	4.38	--	--	--
Shallow Concentrated	--	--	--	--	--	--	--	--	--	2,290	0.0009	16.1
Channel A	104	39.3	0.0025	0.020	656	--	--	--	--	--	--	--
Channel B	104	39.3	0.0169	0.020	674	--	--	--	--	--	--	--

Table 4 – Times of Concentration

Flow Path	Time of Concentration (minutes [min])
West Ash Pond	
Channel A	0.6
Channel B	2.1
Channel C	4.0
Slurry Pond	
Sheet	7.3
Shallow Concentrated	79.0
Channel A	1.5
Channel B	0.6

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

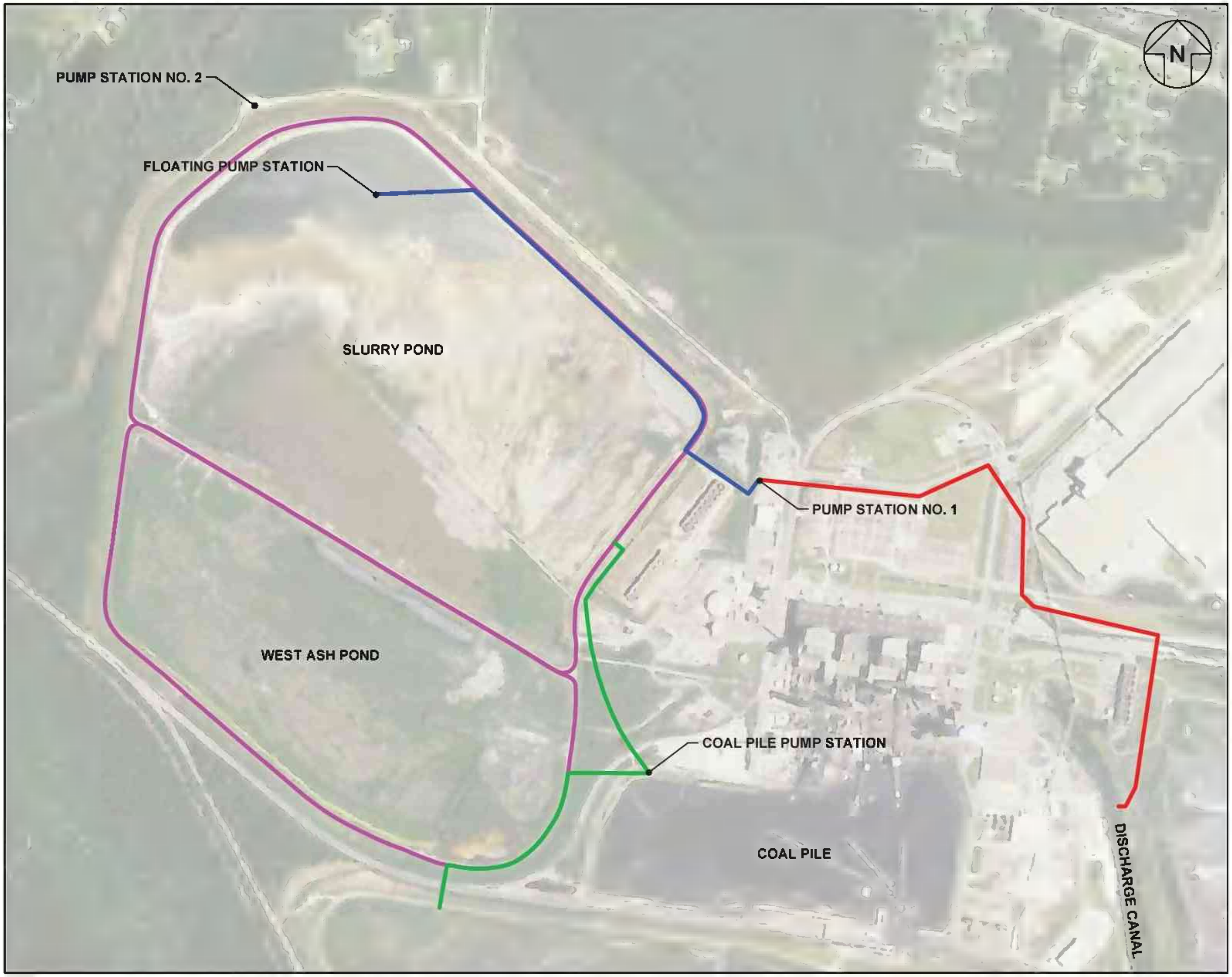
Slurry Pond				**West Ash Pond**			
Elevation (NGVD 29) (ft)	*Area (ac)*	*Volume (ac-ft)*	*Cumulative Volume (ac-ft)*	*Elevation (NGVD 29) (ft)*	*Area (ac)*	*Volume (ac-ft)*	*Cumulative Volume (ac-ft)*
35.67	77.2	103.9	510.2	37	36.6	32.0	98.9
34	47.2	83.6	406.3	36	27.4	23.4	66.9
32	36.4	67.5	322.7	35	19.4	16.4	43.5
30	31.0	57.5	255.3	34	13.4	11.1	27.1
28	26.5	49.0	197.7	33	8.9	7.1	16.0
26	22.5	41.3	148.7	32	5.4	4.2	8.9
24	18.8	34.3	107.5	31	3.0	2.3	4.7
22	15.5	28.0	73.2	30	1.6	1.3	2.4
20	12.6	22.2	45.2	29	0.9	0.7	1.1
18	9.6	14.7	23.0	28	0.5	0.3	0.4
16	5.0	6.7	8.3	27	0.2	0.1	0.1
14	1.6	1.7	1.7	26	0.0	0.0	0.0
12	0.0	0.0	0.0	--	--	--	--

Note: The *HydroCAD* model uses the normal operating pool elevation of 19.6 ft NGVD 29 as the starting water surface elevation for the Slurry Pond.

Table 6 – Peak Elevations and Volumes

Storm Event	**West Ash Pond**			**Slurry Pond**		
	Elevation (NGVD 29) (ft)	*Volume (ac-ft)*	*Time (hr)*	*Elevation (NGVD 29) (ft)*	*Volume (ac-ft)*	*Time (hr)*
PMF, 72 Hr	36.67	87.215	37.21	35.37	488.118	47.66

FIGURES



LEGEND

- POND BOUNDARY
- APPROXIMATE PIPE ALIGNMENT FROM COAL PILE PUMP STATION
- PIPE ALIGNMENT FROM FLOATING PUMP STATION TO PUMP STATION NO. 1
- PIPE ALIGNMENT FROM PUMP STATION NO. 1 TO DISCHARGE CANAL

0 500' 1000'
SCALE IN FEET

WINYAH GENERATING STATION
SITE MAP

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FIGURE 1

PROJECT NO: GSC5242 | OCTOBER 2016

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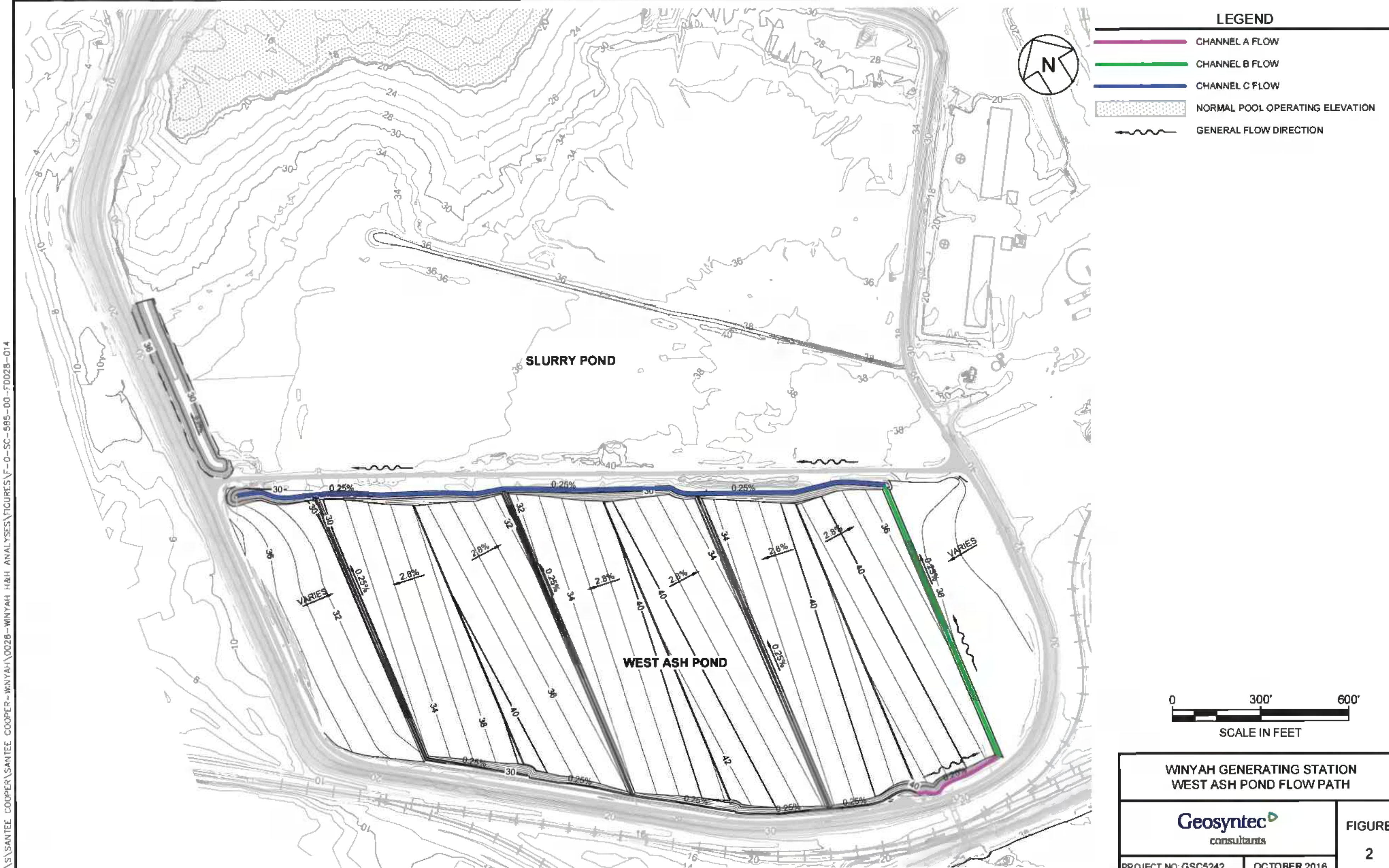

WINYAH GENERATING STATION
WEST ASH POND FLOW PATH

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FIGURE 2

PROJECT NO: GSC5242 | OCTOBER 2016

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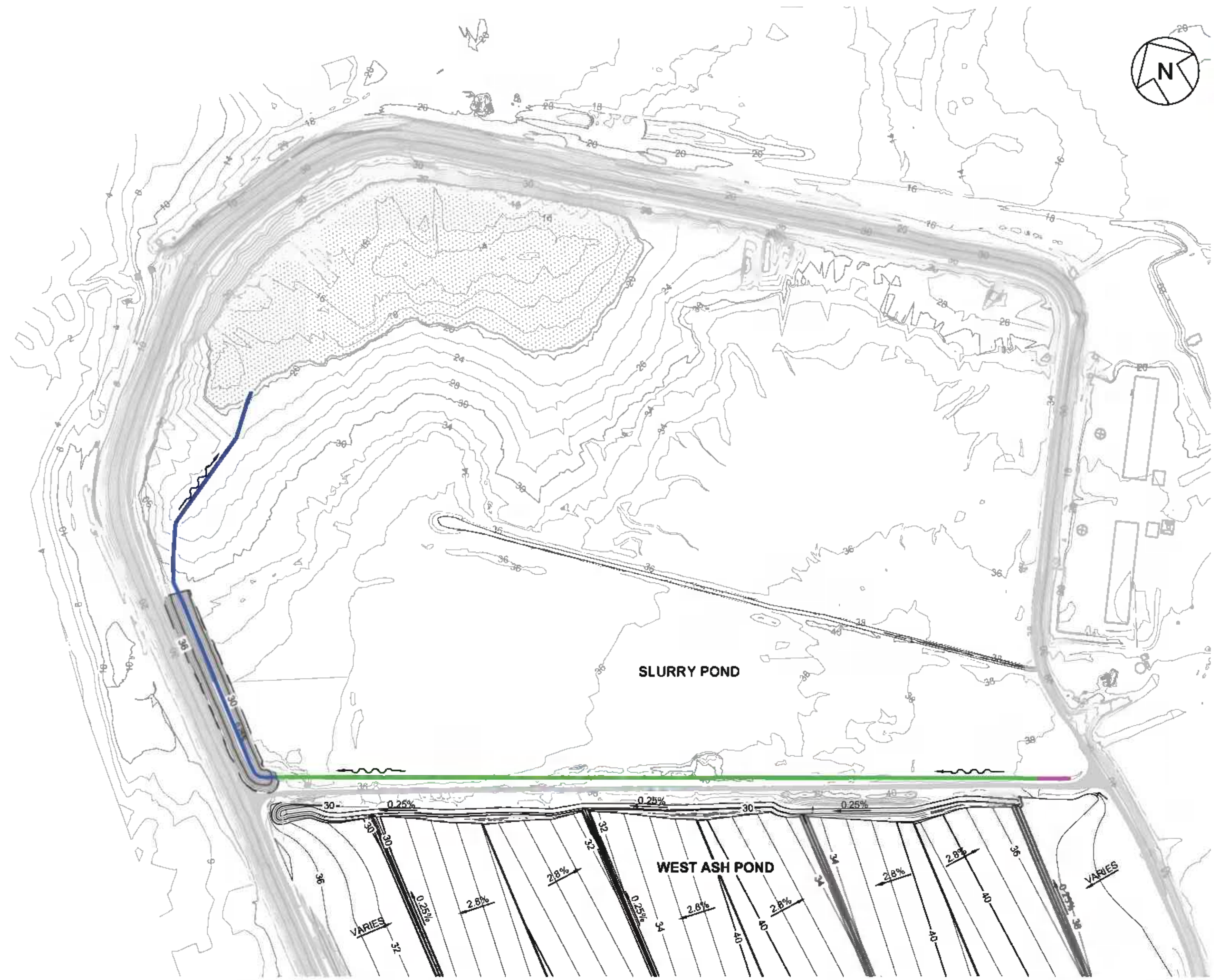

WINYAH GENERATING STATION
SLURRY POND FLOW PATH

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FIGURE 3

PROJECT NO: GSC5242 | OCTOBER 2016

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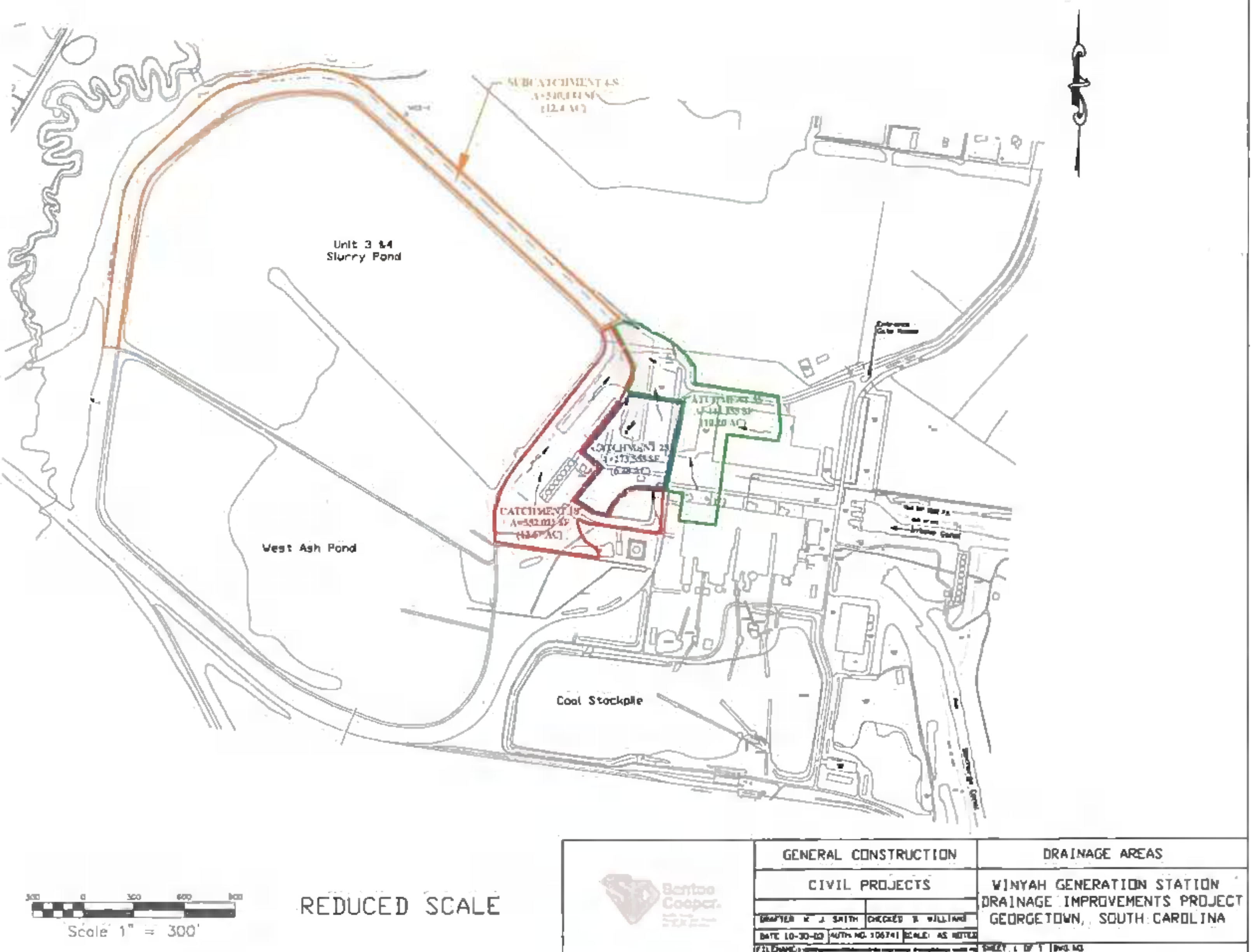
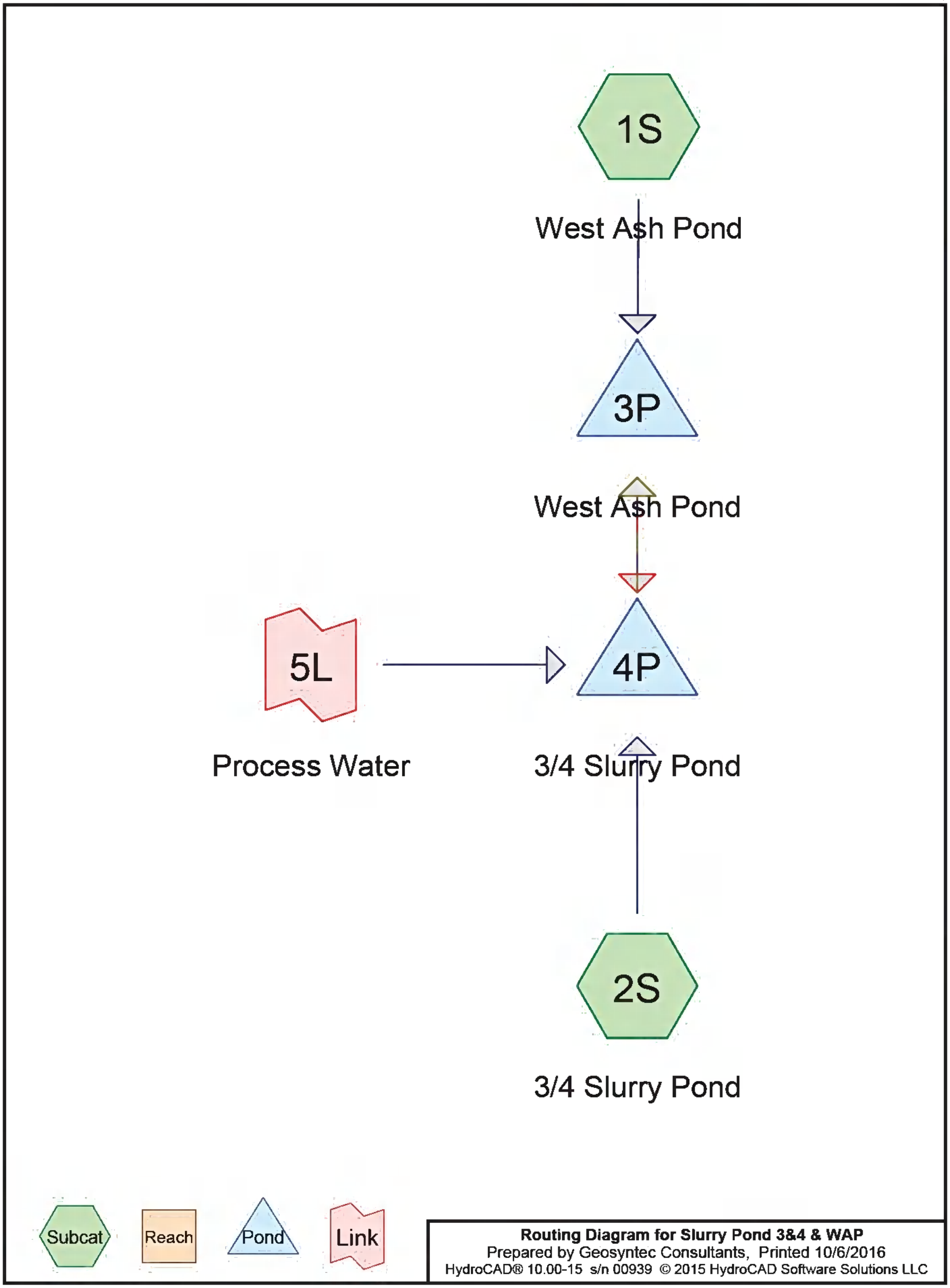
APPENDICES

APPENDIX A



GENERAL CONSTRUCTION	DRAINAGE AREAS
CIVIL PROJECTS	WINYAH GENERATION STATION DRAINAGE IMPROVEMENTS PROJECT GEORGETOWN, SOUTH CAROLINA

APPENDIX B



Routing Diagram for Slurry Pond 3&4 & WAP
Prepared by Geosyntec Consultants, Printed 10/6/2016
HydroCAD® 10.00-15 s/n 00939 © 2015 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
107.205	87	90% FGD + 10% Water (2S)
64.385	98	Exposed Geomembrane (1S)

Time span=0.00-600.00 hrs, dt=0.010 hrs, 60001 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Sim-Route method - Pond routing by Sim-Route method

Subcatchment1S: West Ash Pond Runoff Area=64.385 ac 100.00% Impervious Runoff Depth=52.76"
Flow Length=3,546' Slope=0.0025 '/' Tc=6.7 min CN=98 Runoff=1,282.84 cfs 283.057 af

Subcatchment2S: 3/4 Slurry Pond Runoff Area=107.205 ac 0.00% Impervious Runoff Depth=51.25"
Flow Length=3,720' Tc=88.4 min CN=87 Runoff=1,074.34 cfs 457.838 af

Pond 3P: West Ash Pond Peak Elev=36.67' Storage=87.215 af Inflow=1,282.84 cfs 283.057 af
mary=100.39 cfs 206.020 af Secondary=42.17 cfs 56.538 af Tertiary=140.83 cfs 20.496 af Outflow=270.20 cfs 283.053 af

Pond 4P: 3/4 Slurry Pond Peak Elev=35.37' Storage=488.118 af Inflow=1,209.40 cfs 923.126 af
Primary=119.39 cfs 810.254 af Secondary=0.00 cfs 0.000 af Tertiary=0.00 cfs 0.000 af Outflow=119.39 cfs 810.254 af

Link 5L: Process Water Manual Hydrograph Inflow=4.46 cfs 202.731 af
Primary=4.46 cfs 202.731 af

Summary for Subcatchment 1S: West Ash Pond

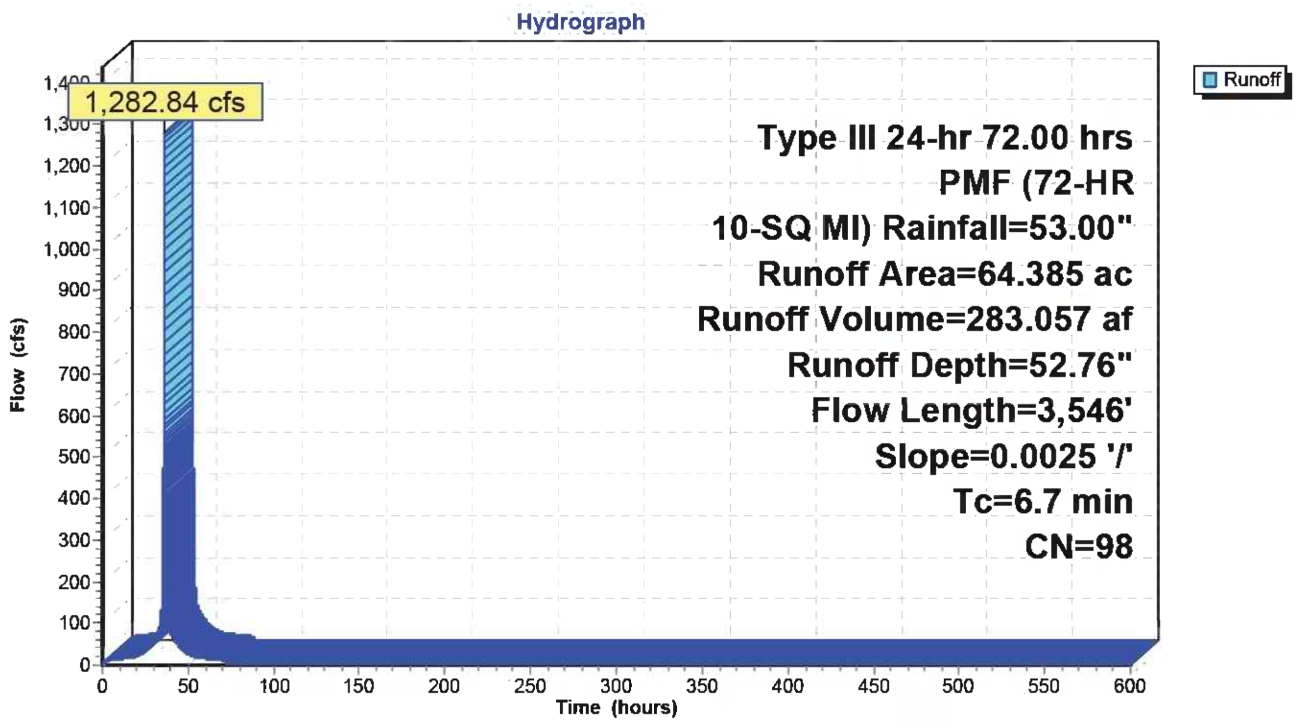
Runoff = 1,282.84 cfs @ 36.10 hrs, Volume= 283.057 af, Depth=52.76"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-600.00 hrs, dt= 0.010 hrs
Type III 24-hr 72.00 hrs PMF (72-HR, 10-SQ MI) Rainfall=53.00"

Area (ac)	CN	Description
64.385	98	Ex osed Geomembrane
64.385		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.6	302	0.0025	8.13	316.96	**Channel Flow, Channel A Flow** Area= 39.0 sf Perim= 23.0' r= 1.70' n= 0.013
2.1	1,022	0.0025	8.13	316.96	**Channel Flow, Channel B Flow** Area= 39.0 sf Perim= 23.0' r= 1.70' n= 0.013
4.0	2,222	0.0025	9.35	644.84	**Channel Flow, Channel C Flow** Area= 69.0 sf Perim= 33.0' r= 2.09' n= 0.013
6.7	3,546	Total			

Subcatchment 1S: West Ash Pond



Summary for Subcatchment 2S: 3/4 Slurry Pond

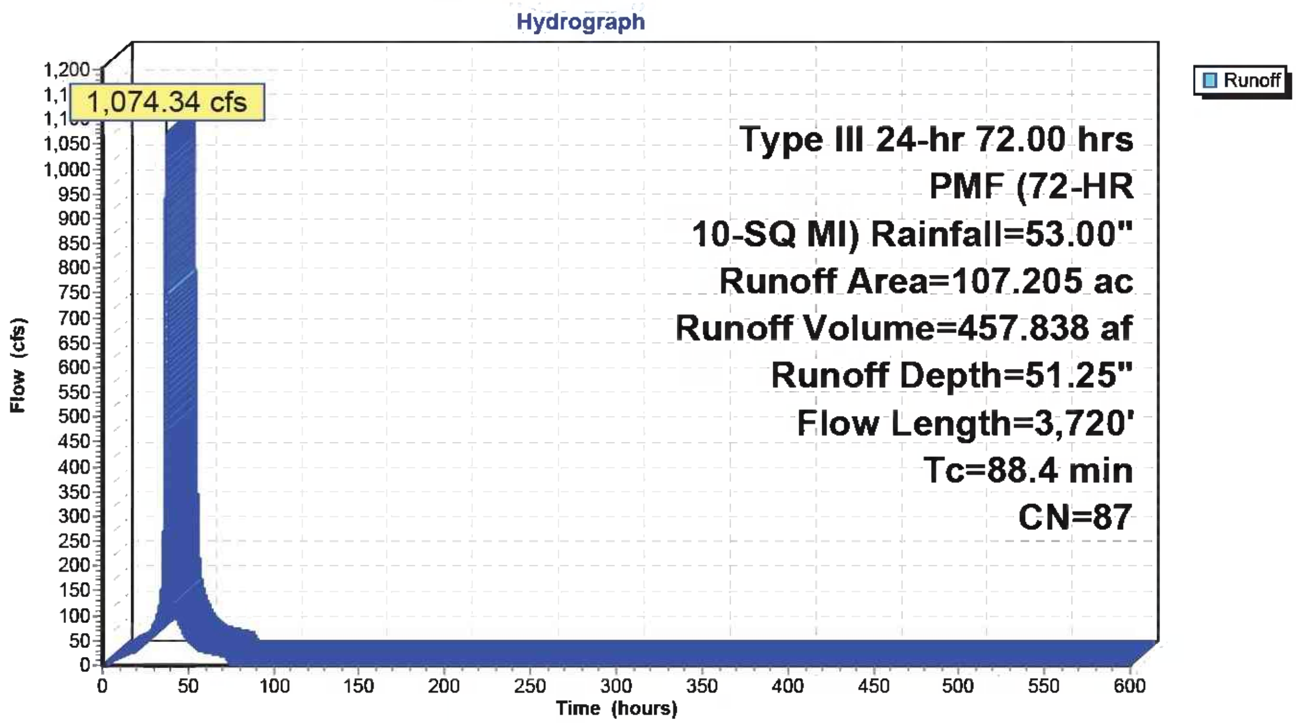
Runoff = 1,074.34 cfs @ 37.13 hrs, Volume= 457.838 af, Depth=51.25"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-600.00 hrs, dt= 0.010 hrs
Type III 24-hr 72.00 hrs PMF (72-HR, 10-SQ MI) Rainfall=53.00"

	Area (ac)	CN	Description
*	107.205	87	90% FGD + 10% Water
	107.205		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
7.3	100	0.0005	0.23		**Sheet Flow, Sheet Flow** n= 0.020 P2= 4.38"
79.0	2,290	0.0009	0.48		**Shallow Concentrated Flow, Shallow Concentrated Flow** Unpaved Kv= 16.1 fps
1.5	656	0.0025	7.11	739.18	**Channel Flow, Channel A Flow** Area= 104.0 sf Perim= 39.3' r= 2.65' n= 0.020
0.6	674	0.0169	18.48	1,921.88	**Channel Flow, Channel B Flow** Area= 104.0 sf Perim= 39.3' r= 2.65' n= 0.020
88.4	3,720	Total			

Subcatchment 2S: 3/4 Slurry Pond



Summary for Pond 3P: West Ash Pond

Inflow =	1,282.84 cfs @	36.10 hrs,	Volume=	283.057 af
Outflow =	270.20 cfs @	37.16 hrs,	Volume=	283.053 af, Atten= 79%, Lag= 63.7 min
Primary =	100.39 cfs @	36.34 hrs,	Volume=	206.020 af
Secondary =	42.17 cfs @	37.21 hrs,	Volume=	56.538 af
Tertiary =	140.83 cfs @	37.21 hrs,	Volume=	20.496 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.010 hrs
Peak Elev= 36.67' @ 37.21 hrs Surf.Area= 33.519 ac Storage= 87.215 af

Plug-Flow detention time= 498.3 min calculated for 283.048 af (100% of inflow)
Center-of-Mass det. time= 498.7 min (2,673.6 - 2,175.0)

Volume	Invert	Avail.Storage	Storage Description
#1	26.00'	98.929 af	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
26.00	0.004	0.000	0.000
27.00	0.182	0.093	0.093
28.00	0.481	0.331	0.425
29.00	0.932	0.706	1.131
30.00	1.599	1.265	2.397
31.00	2.978	2.288	4.685
32.00	5.395	4.186	8.872
33.00	8.851	7.123	15.995
34.00	13.446	11.149	27.143
35.00	19.354	16.400	43.543
36.00	27.418	23.386	66.929
37.00	36.582	32.000	98.929

Device	Routing	Invert	Outlet Devices
#1	Primary	25.96'	**30.0" Round Culvert 1** L= 99.1' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 25.96' / 24.87' S= 0.0110 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 4.91 sf
#2	Primary	25.90'	**30.0" Round Culvert 2** L= 98.7' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 25.90' / 24.64' S= 0.0128 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 4.91 sf
#3	Secondary	33.40'	**17.8" Round Existing Culvert 1** L= 50.7' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 32.87' / 33.40' S= -0.0105 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#4	Secondary	33.25'	**17.8" Round Existing Culvert 2** L= 51.4' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 32.77' / 33.25' S= -0.0093 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#5	Secondary	33.42'	**17.8" Round Existing Culvert 3** L= 50.5' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 32.81' / 33.42' S= -0.0121 '/' Cc= 0.900

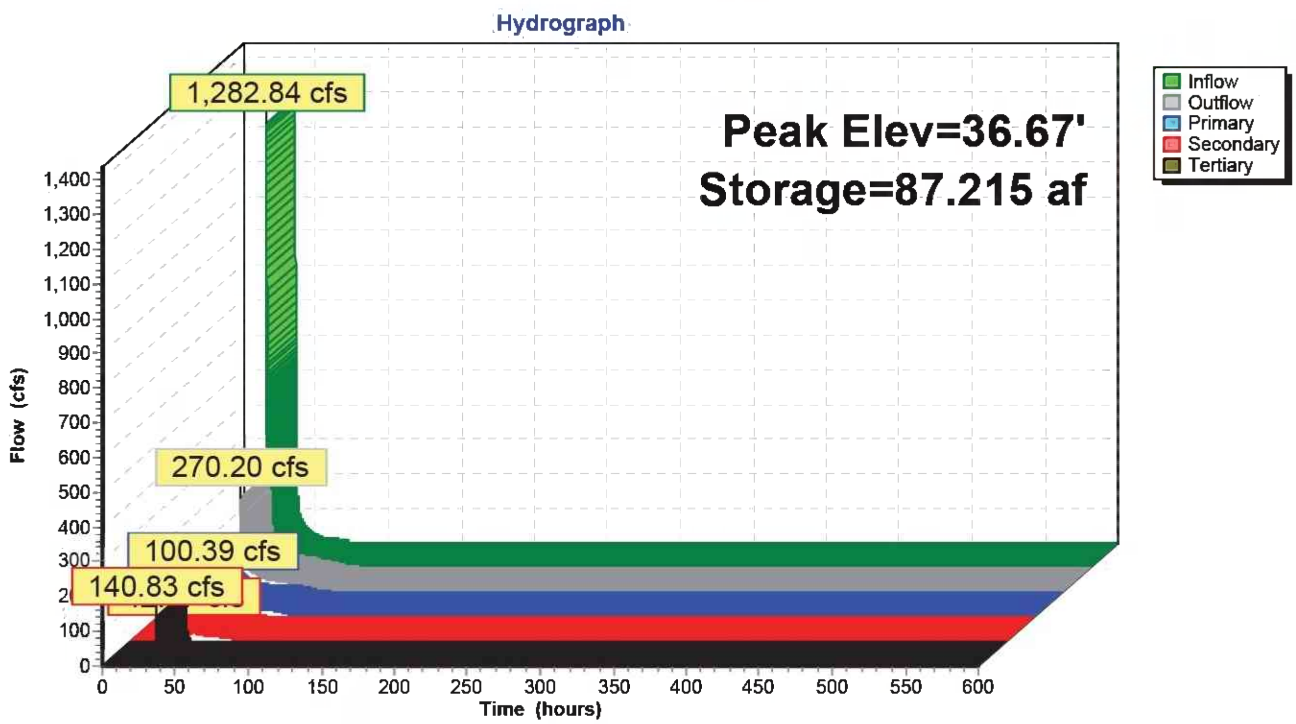
			n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#6	Secondary	33.32'	**17.8" Round Existing Culvert 4**
			L= 50.8' CPP, projecting, no headwall, Ke= 0.900
			Inlet / Outlet Invert= 32.75' / 33.32' S= -0.0112 '/' Cc= 0.900
			n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#7	Tertiary	36.25'	**200.0' long x 12.0' breadth Broad-Crested Rectangular Weir**
			Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60
			Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64
#8	Tertiary	36.75'	**30.0' long x 12.0' breadth Broad-Crested Rectangular Weir**
			Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60
			Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64

Primary OutFlow Max=100.24 cfs @ 36.34 hrs HW=36.20' TW=30.42' (Dynamic Tailwater)
1=Culvert 1 (Inlet Controls 50.12 cfs @ 10.21 fps)
2=Culvert 2 (Inlet Controls 50.12 cfs @ 10.21 fps)

Secondary OutFlow Max=42.17 cfs @ 37.21 hrs HW=36.67' TW=32.39' (Dynamic Tailwater)
3=Existing Culvert 1 (Inlet Controls 10.44 cfs @ 6.04 fps)
4=Existing Culvert 2 (Inlet Controls 10.74 cfs @ 6.22 fps)
5=Existing Culvert 3 (Inlet Controls 10.40 cfs @ 6.02 fps)
6=Existing Culvert 4 (Inlet Controls 10.60 cfs @ 6.13 fps)

Tertiary OutFlow Max=140.82 cfs @ 37.21 hrs HW=36.67' (Free Discharge)
7=Broad-Crested Rectangular Weir (Weir Controls 140.82 cfs @ 1.69 fps)
8=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

Pond 3P: West Ash Pond



Summary for Pond 4P: 3/4 Slurry Pond

Inflow	=	1,209.40 cfs @	37.13 hrs,	Volume=	923.126 af
Outflow	=	119.39 cfs @	47.66 hrs,	Volume=	810.254 af, Atten= 90%, Lag= 631.6 min
Primary	=	119.39 cfs @	47.66 hrs,	Volume=	810.254 af
Secondary	=	0.00 cfs @	0.00 hrs,	Volume=	0.000 af
Tertiary	=	0.00 cfs @	0.00 hrs,	Volume=	0.000 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.010 hrs
Starting Elev= 19.60' Surf.Area= 11.970 ac Storage= 40.293 af
Peak Elev= 35.37' @ 47.66 hrs Surf.Area= 71.884 ac Storage= 488.118 af (447.824 af above start)

Plug-Flow detention time= 5,395.1 min calculated for 769.960 af (83% of inflow)
Center-of-Mass det. time= 2,120.8 min (7,651.1 - 5,530.3)

Volume	Invert	Avail.Storage	Storage Description
#1	12.00'	510.207 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
12.00	0.029	0.000	0.000
14.00	1.643	1.672	1.672
16.00	5.029	6.672	8.344
18.00	9.636	14.665	23.009
20.00	12.553	22.189	45.198
22.00	15.458	28.011	73.209
24.00	18.807	34.265	107.474
26.00	22.466	41.273	148.747
28.00	26.497	48.963	197.710
30.00	31.046	57.543	255.253
32.00	36.416	67.462	322.715
34.00	47.197	83.613	406.328
35.67	77.209	103.879	510.207

Device	Routing	Invert	Outlet Devices
#1	Primary	25.96'	**30.0" Round Culvert 1** L= 99.1' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 24.87' / 25.96' S= -0.0110 '/' Cc= 0.900 n= 0.013, Flow Area= 4.91 sf
#2	Primary	25.90'	**30.0" Round Culvert 2** L= 98.7' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 24.64' / 25.90' S= -0.0128 '/' Cc= 0.900 n= 0.013, Flow Area= 4.91 sf
#3	Secondary	33.40'	**17.8" Round Existing Culvert 1** L= 50.7' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 33.40' / 32.87' S= 0.0105 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#4	Secondary	33.25'	**17.8" Round Existing Culvert 2** L= 51.4' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 33.25' / 32.77' S= 0.0093 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#5	Secondary	33.42'	**17.8" Round Existing Culvert 3**

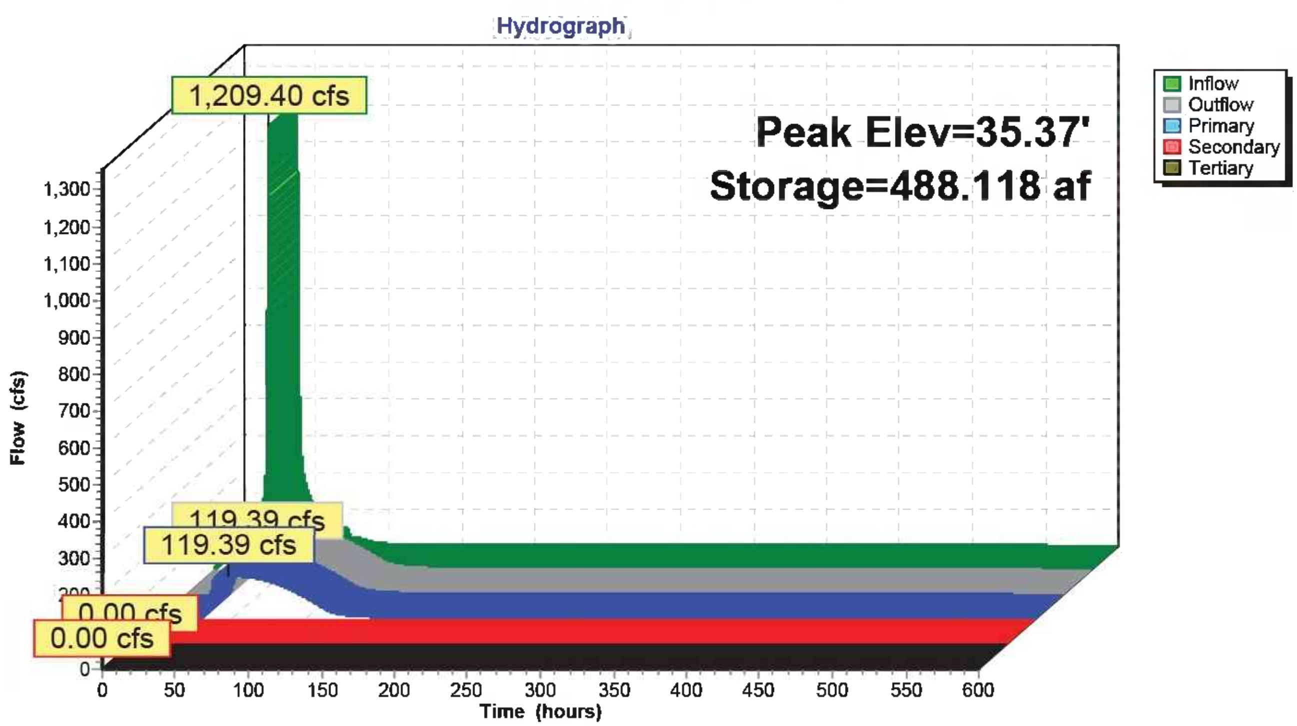
			L= 50.5' CPP, projecting, no headwall, Ke= 0.900
			Inlet / Outlet Invert= 33.42' / 32.81' S= 0.0121 '/' Cc= 0.900
			n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#6	Secondary	33.32'	**17.8" Round Existing Culvert 4**
			L= 50.8' CPP, projecting, no headwall, Ke= 0.900
			Inlet / Outlet Invert= 33.32' / 32.75' S= 0.0112 '/' Cc= 0.900
			n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.73 sf
#7	Tertiary	36.25'	**200.0' long x 12.0' breadth Broad-Crested Rectangular Weir**
			Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60
			Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64
#8	Tertiary	36.75'	**30.0' long x 12.0' breadth Broad-Crested Rectangular Weir**
			Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60
			Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64

Primary OutFlow Max=119.39 cfs @ 47.66 hrs HW=35.37' (Free Discharge)
1=Culvert 1 (Inlet Controls 59.59 cfs @ 12.14 fps)
2=Culvert 2 (Inlet Controls 59.81 cfs @ 12.18 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=19.60' TW=26.00' (Dynamic Tailwater)
3=Existing Culvert 1 (Controls 0.00 cfs)
4=Existing Culvert 2 (Controls 0.00 cfs)
5=Existing Culvert 3 (Controls 0.00 cfs)
6=Existing Culvert 4 (Controls 0.00 cfs)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=19.60' TW=26.00' (Dynamic Tailwater)
7=Broad-Crested Rectangular Weir (Controls 0.00 cfs)
8=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

Pond 4P: 3/4 Slurry Pond



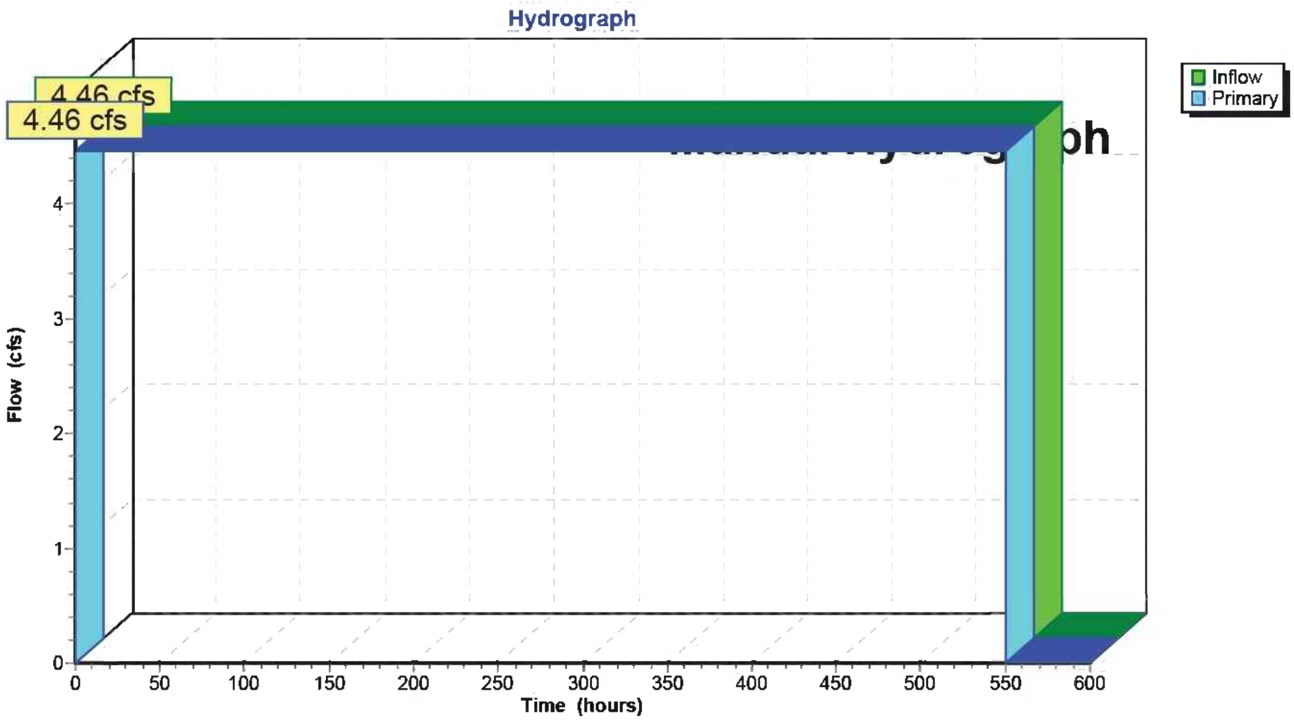
Summary for Link 5L: Process Water

Inflow	=	4.46 cfs @	0.00 hrs,	Volume=	202.731 af	
Primary	=	4.46 cfs @	0.01 hrs,	Volume=	202.731 af,	Atten= 0%, Lag= 0.6 min

Primary outflow = Inflow, Time Span= 0.00-600.00 hrs, dt= 0.010 hrs

12 Point manual hydrograph, To= 0.00 hrs, dt= 50.00 hrs, cfs =
4.46 4.46 4.46 4.46 4.46 4.46 4.46 4.46 4.46 4.46
4.46 4.46

Link 5L: Process Water



ATTACHMENT 2

Boring Logs

ATTACHMENT 2-A

Geosyntec Boring Logs

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104 South Main Street
Suite 115
Greenville, SC 29601

BORING LOG

BOREHOLE ID: *GSB-2*

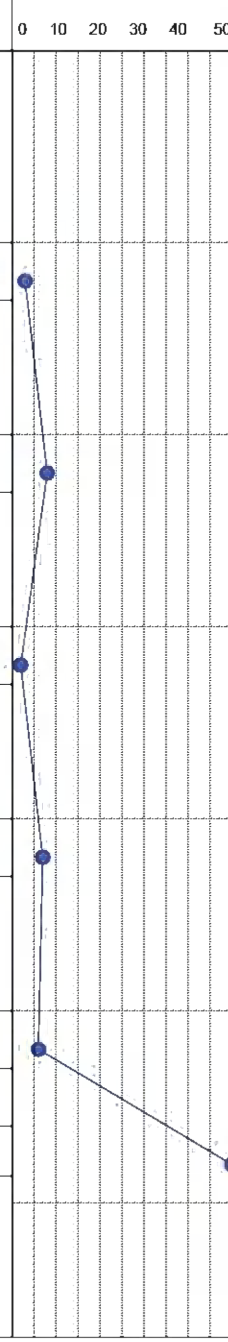
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/4/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *549429.980*
EASTING: *2500568.066*
GROUND ELEVATION: *21.93 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
20	0						Boring located at toe of dike adjacent to cooling towers
15	-5	Very loose, gray, clayey SAND (SC) with some small shell fragments		1-2-1		3"	MC=32.9%; Gravel=19.1%; Sand=45.0%; Silt=7.9%; Clay=28.0%; Fines=35.9% LL=42; PL=23; PI=19
10	-10	Loose, gray to brown, clayey fine SAND (SC)		2-4-4		10"	MC=32.0%; FC=18.9% 1/2 of sample distinctly more clayey than other 1/2 of sample
5	-15	Very loose, brown to gray, clayey silty SAND (SC) with some shells		3-1-1		15"	Shells observed at bottom of split spoon
0	-20	Loose, gray, clayey to fine SAND (SC) with shells at top of sample		2-3-4		18"	
-5	-25	Firm, dark gray to gray, sandy CLAY (CL) with shells		2-3-3		18"	MC=34.2%; FC=22.3%
		Very dense, black, clayey SAND (SC) with shells Refusal at 29.3 ft bgs		5-21-50		3"	59 blows/3" (Chicora)
-10	-30						



All depths referenced to ground surface.
Total Depth: 29.3 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-3*

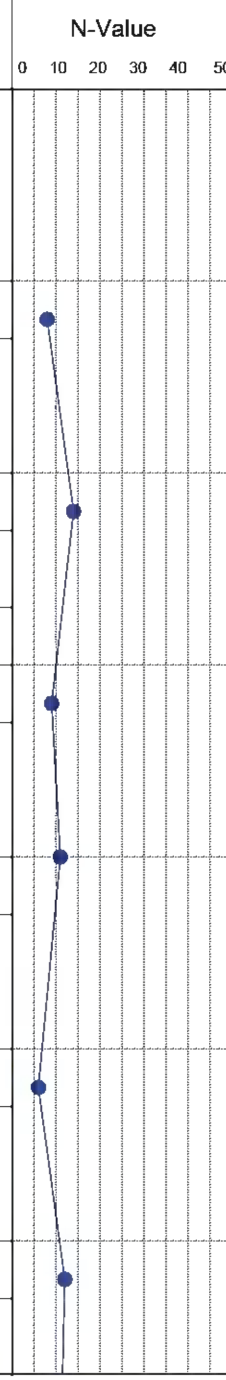
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/25/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *549760.850*
EASTING: *2500650.369*
GROUND ELEVATION: *38.39 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				Boring located at center of dike.
35	-5	First 12", Loose, brown SAND with some silt (SM). Next 6", Firm, brown to gray, sandy CLAY (CL)	2-3-5	12"	
30	-10	Stiff, dark brown to black, sandy CLAY (CL)	2-7-7	12"	MC=21.6%; Gravel=8.2%; Sand=41.3%; Silt=12.5%; Clay=38.0%; Fines=50.5% LL=29; PL=18; PI=11
			ST-1	24"	MC=19.8%; Gravel=0.0%; Sand=59.8%; Silt=12.4%; Clay=27.8%; Fines=40.2% LL=36; PL=18; PI=18; S.G=2.67
25	-15	Stiff, brown to gray, sandy CLAY (CL)	2-4-5	18"	
20	-20	First 12", Stiff, dark brown, sandy CLAY (CL). Next 6", Stiff, light brown, sandy CLAY (CL) with some wood chips	2-4-7	18"	MC=24.3%; LL=50; PL=23; PI=27
15	-25	First 6", Firm, dark brown, sandy to silty CLAY (CL).	1-3-3	18"	
10	-30	First 6", sluff. Next 6", Soft, gray CLAY (CL). Next 6", Medium dense, gray, poorly graded, medium grained SAND (SP)	2-3-9	18"	MC=58.0%; Gravel=0.2%; Sand=41.0%; Fines=58.8%
5					



All depths referenced to ground surface.
Total Depth: 44.3 ft bgs

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BORING LOG

Borehole ID: *GSB-3*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	-35	Medium dense, gray, medium grained SAND (SP), some interbedded clay lenses		3-6-5		12"	MC=35.4%;Fines=28.0%; Slight H2S odor
0							
	-40	Loose, gray, clayey SAND (SC) with interbedded shells throughout		2-3-4		18"	MC=33.9%; Gravel=6.8%; Sand=66.3%; Fines=26.9%
-5		Very dense, gray, clayey SAND (SC) with interbedded shells throughout		50/3"		3"	Hard drilling at 44 ft bgs. 50 blows/3" (Chicora)
	-45	Refusal at 44.3 ft bgs					
-10	-50						
-15	-55						
-20	-60						
-25	-65						
-30	-70						
-35	-75						

All depths referenced to ground surface.

Total Depth: 44.3 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-4*

GENERAL INFORMATION

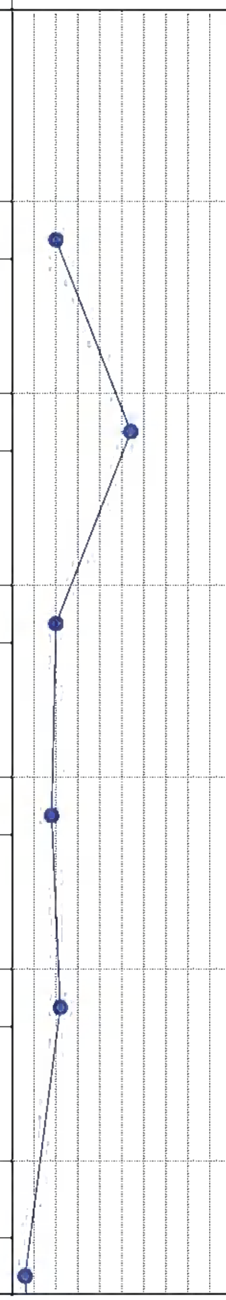
PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/25/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *550370.128*
EASTING: *2500167.341*
GROUND ELEVATION: *38.66 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				Boring located at center of dike.
35	-5	Medium dense, gray to dark brown, medium grained clayey SAND (SC).	3-4-6	18"	
30	-10	Dense, gray, medium grained clayey SAND (SC)	13-13-14	12"	MC=19.9%; Gravel=0.0%; Sand=81.8%; Silt=4.8%; Clay=13.4%; Fines=18.2%; LL=35; PL=19; PI=16
25	-15	Medium dense, brown to dark brown, clayey SAND (SC)	3-5-5	18"	
20	-20	Loose, dark brown, clayey SAND (SC)	3-4-5	18"	MC=20.1%; LL=42; PL=22; PI=20
15	-25	Medium dense, tan to brown, fine grained, clayey SAND (SC)	4-5-6	18"	
10	-30		ST-1	24"	MC=29.1%; Gravel=0.0%; Sand=80.1%; Fines=19.9%
		Very loose, gray, fine grained clayey SAND (SC)	1-1-2	24"	

N-Value



All depths referenced to ground surface.

Total Depth: 56.5 ft bgs

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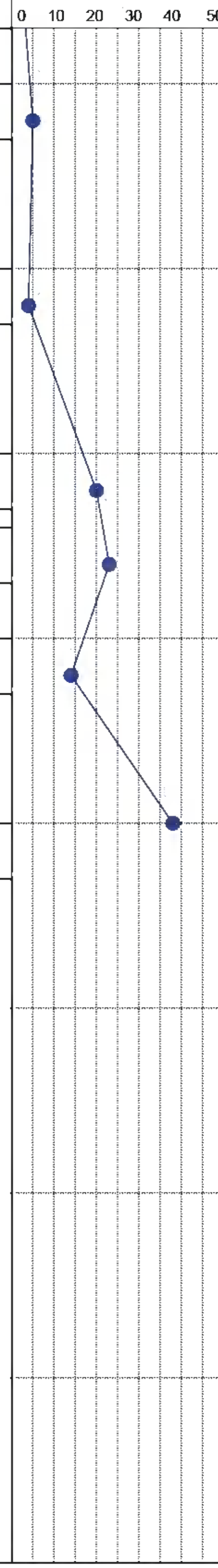
BORING LOG

Borehole ID: *GSB-4*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
5 to 0	-35 to -40	First 12", Loose, gray, clayey SAND (SC). Next 6", Firm, gray CLAY (CH)	2-2-3	18"	MC=47.2; Fines=39.0%; Shell lense at 36.5 ft bgs.
0 to -5	-40 to -43	Very loose, gray, clayey SAND (SC) with interbedded shells	2-1-3	18"	MC=35.3%; Fines=19.6%
-5	-44	Medium dense, gray, silty SAND (SM) with interbedded shell and rock fragments			
	-45	First 6", Medium dense cemented shells with some interbedded clay. Next 6", Very stiff, dark gray CLAY (CH) (Williamsburg Formation)	11-12-8	18"	
	-47		38-12-11	12"	MC=38.4%; Fines=79.0% Driller stated there was a hard cemented layer ~2" thick (Chicora).
-10	-50	Stiff, dark gray, dry CLAY (CH) (Williamsburg Formation)	3-6-8	18"	
-15	-55	Hard, black, dry SILT (MH) (Williamsburg Formation) Terminated at 56.5 ft bgs.	14-16-22	18"	MC=41.9%; Gravel=0.0%; Sand=17.2%; Silt=42.2%; Clay=40.6%; Fines=82.8%; LL=70; PL=46; PI=24

N-Value



All depths referenced to ground surface.

Total Depth: 56.5 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-5*

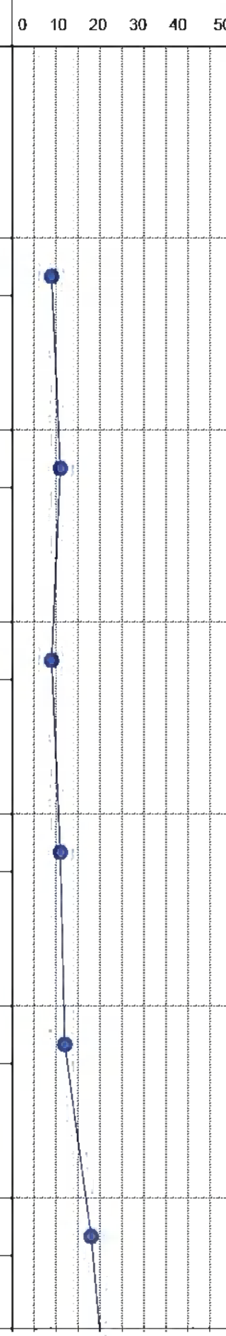
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/26/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *550684.521*
EASTING: *2499829.144*
GROUND ELEVATION: *38.05 NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0						Boring located at center of dike.
35							
	-5	Loose, brown to gray, clayey SAND (SC)		2-4-5		12"	
30							
	-10	Medium dense, brown to gray, silty to clayey SAND (SC).		4-5-6		12"	
25							
	-15	Loose, brown to dark brown, clayey SAND (SC) with trace wood chips		2-3-6		18"	MC=20.0%; Gravel=0.0%; Sand=63.9; Fines=36.1%; LL=33; PL=19; PI=14
20							
	-20	Medium dense, brown to black, clayey SAND (SC)		4-5-6		12"	
15							
	-25	Medium dense, gray with some brown, clayey SAND (SC)		4-5-6		18"	
10							
	-30	Medium dense, tan to orange, fine to medium grained, poorly graded, slightly silty SAND (SP)		6-8-10		18"	MC=28.5%; Fines=9.5%
5							



All depths referenced to ground surface.

Total Depth: 43.4 ft bgs

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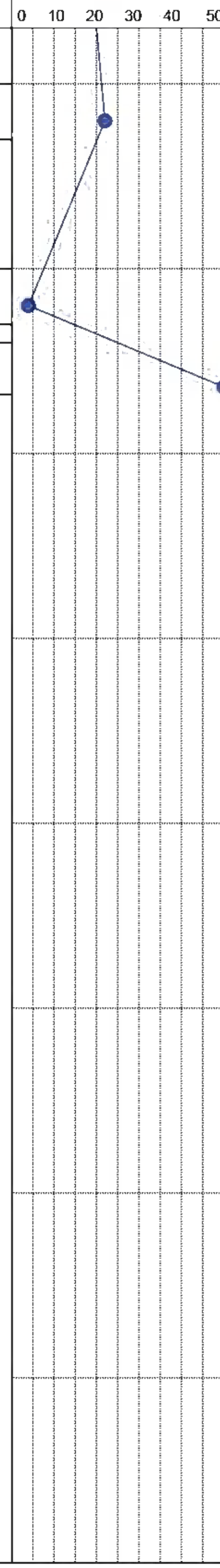
BORING LOG

Borehole ID: *GSB-5*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	-35	Medium dense, tan to gray, fine to medium grained, poorly graded, slightly silty SAND (SP)		8-10-12		6"	MC=22.1%; Fines=8.8%
0							
	-40	Very loose, gray, clayey SAND (SC) with large wood chips at 40.0 ft bgs (1" thick)		1-2-2		18"	MC=24.5%; Gravel=1.6%; Sand=76.5; Fines=21.9%
-5		Very dense, gray, clayey SAND (SC) with partially cemented shells Refusal at 43.4 ft bgs		50/5"		5"	Driller encountered hard layer at 43.0 ft bgs (Chicora)
	-45						
-10	-50						
-15	-55						
-20	-60						
-25	-65						
-30	-70						
-35	-75						



All depths referenced to ground surface.

Total Depth: 43.4 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-6*

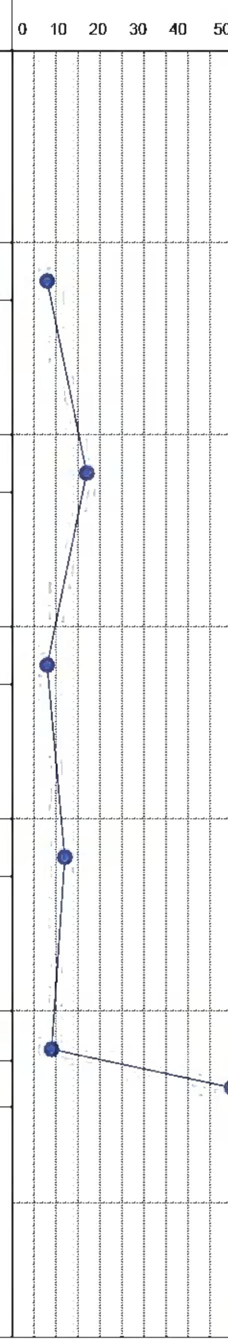
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/4/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *551174.405*
EASTING: *2499470.729*
GROUND ELEVATION: *20.68 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
20	0						Boring located at toe of dike.
15	-5	Loose, brown, medium to fine, slightly silty SAND (SM)		2-4-4		12"	
10	-10	Medium dense, brown, medium to fine clayey SAND (SC)		5-7-10		15"	MC=27.1%; Gravel=1.2%; Sand=77.1%; Fines=21.7%
5	-15	Loose, gray to tan, medium to fine SAND (SP)		4-3-5		12"	MC=23.2%; Fines=7.9%
0	-20	Medium dense, medium to fine, clayey SAND (SC) with shells		3-6-6		18"	MC=30.8%; Fines=16.3%
-5	-25	Loose, gray, slightly clayey SAND (SC) with shells		3-3-6		18"	
		Refusal at 27.5 ft bgs		50/4"			No Recovery. 50 blows/4" (Chicora)
-10	-30						



All depths referenced to ground surface.

Total Depth: 27.5 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-7*

GENERAL INFORMATION

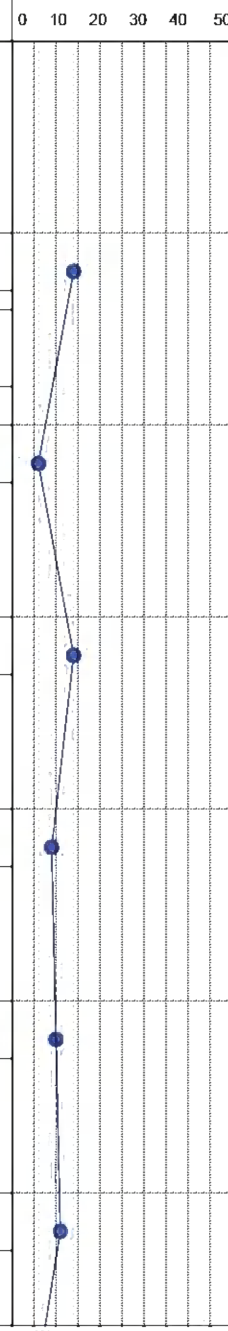
PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/26/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *551318.990*
EASTING: *2498950.556*
GROUND ELEVATION: *38.16 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				Boring located at center of dike.
35					
	-5	Medium dense, light gray, clean SAND (SP)	6-8-6	18"	
30			ST-1	24"	MC=21.5%; Gravel=0.0%; Sand=66.9%; Silt=8.5%; Clay=24.6%; Fines=33.1%; LL=33; PL=21; PI=12; S.G.=2.69
	-10	Medium stiff, brown to gray, clayey SAND (SC) with trace wood chips and rock fragments	2-2-4	18"	
25					
	-15	First 6", Medium dense, brown to gray, clayey SAND (SC)	2-8-6	18"	MC=19.1%; Gravel=0.0%; Sand=62.5%; Fines=37.5%; LL=34; PL=17; PI=17
20					
	-20	Loose, brown to dark gray, clayey SAND (SC)	2-4-5	18"	
15					
	-25	Medium dense, gray with some orange and red, clayey SAND (SC)	3-3-7	18"	MC=25.8%; LL=37; PL=19; PI=18
10					
	-30	Medium dense, gray to tan, fine to medium grained SAND (SC)	4-4-7	12"	MC=23.4%; Gravel=0.0%; Sand=87.5%; Fines=12.5%
5					

N-Value



All depths referenced to ground surface.

Total Depth: 51.5 ft bgs

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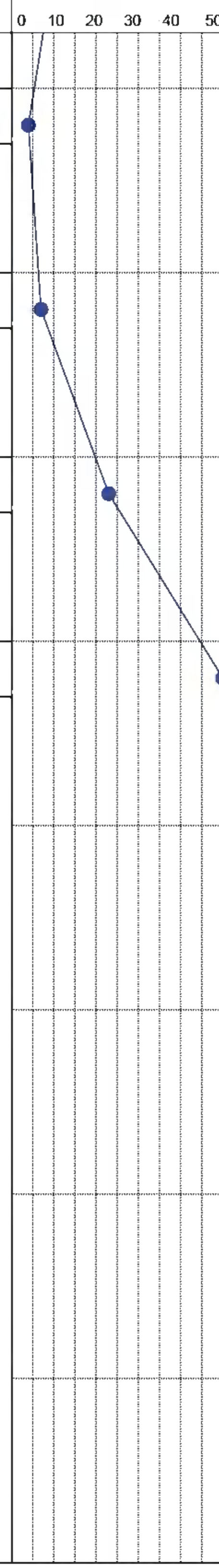
BORING LOG

Borehole ID: *GSB-7*
Project No: *GSC5242*

Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
-35	Very loose, greenish gray, clayey SAND (SC)	2-1-3	12"	MC=28.2%; Gravel=0.7%; Sand=56.4%; Silt=13.8%; Clay=29.1%; Fines=42.9%
-40	Loose, gray, clayey SAND (SC) with interbedded shells throughout	3-3-4	18"	
-45	Medium dense, gray, silty SAND (SM) with interbedded shells throughout	5-17-6	12"	MC=16.6%; Fines=17.5%
-50	Very dense, gray, clayey SAND (SC) with interbedded shells Refusal at 51.5 ft bgs	5-6-50/3"	15"	50 blows/3" (Chicora)

Elev. (ft NGVD29) scale: 0, -5, -10, -15, -20, -25, -30, -35
Depth (ft) scale: -35, -40, -45, -50, -55, -60, -65, -70, -75

N-Value



All depths referenced to ground surface.

Total Depth: 51.5 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-8*

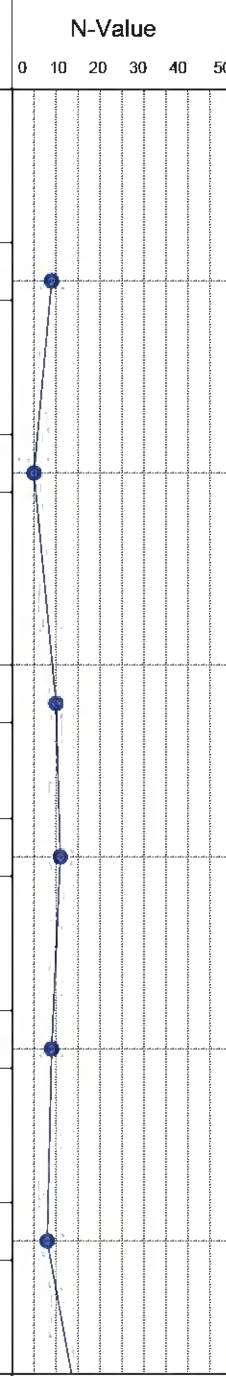
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/27/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *551066.479*
EASTING: *2498252.288*
GROUND ELEVATION: *39.19 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value	Recovery	Comments
	0						Boring located at center of dike.
35	-5	Loose, brown and gray, clayey SAND (SC)		2-4-5		18"	
30	-10	Loose, brown to gray, clayey SAND (SC) with trace wood chips		2-2-3		18"	MC=23.2%; Gravel=0.0%; Sand=54.3%; Fines=45.6%; LL=45; PL=16; PI=29
25	-15	First 4", Medium dense, brown and gray, clayey SAND (SC) with trace wood chips.		3-4-6		24" 12"	MC=23.0%; Fines=39.6%
20	-20	Medium dense, brown to gray, clayey SAND (SC)		5-6-5		12"	
15	-25	Loose, brown to gray, clayey SAND (SC) w/ 3" seam of black silt at 26 ft bgs		3-4-5		12"	MC=17.4%; Gravel=0.0%; Sand=61.7%; Silt=8.2%; Clay=30.1% Fines=38.3%
10	-30	Medium stiff, brown to gray, sandy CLAY (CL)		3-3-5		18"	



All depths referenced to ground surface.
Total Depth: 48.0 ft bgs

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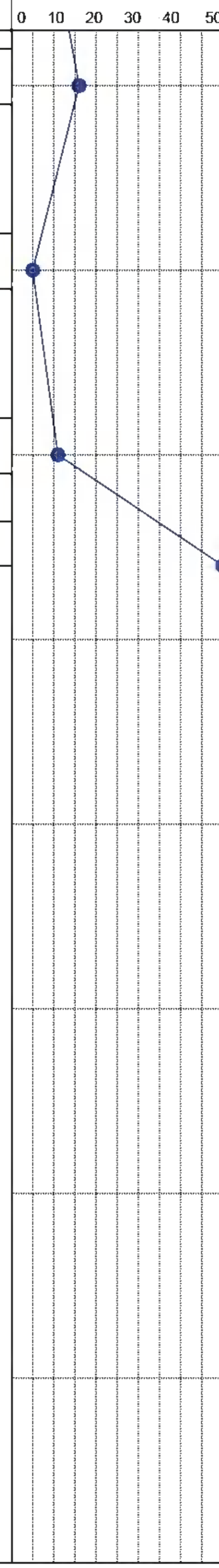
BORING LOG

Borehole ID: *GSB-8*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
5	-35	Medium dense, gray, fine to medium grained, silty SAND (SM)	5-7-9	18"	MC=23.9%; Fines=12.8%
				18"	MC=13.6%; Gravel=0.0%; Sand=94.5%; Fines=5.5%
					MC=30.6%; Gravel=0.5%; Sand=67.6%; Silt=8.2%; Clay=23.7% Fines=31.9%; S.G.=2.64
0	-40	Loose, gray, slightly clayey SAND (SC) w/ interbedded shells throughout	2-2-3	18"	MC=32.0%; Fines=17.4%
-5	-45	Medium dense, gray, slightly clayey SAND (SC) w/ shells	5-5-6	18"	MC=23.0%; Fines=15.1%
		Refusal at 48.0 ft bgs	50/0"	0"	Hard drilling at 48.0 ft bgs. 50 blows/0" (Chicora)
-10	-50				
-15	-55				
-20	-60				
-25	-65				
-30	-70				
-35	-75				

N-Value



All depths referenced to ground surface.

Total Depth: 48.0 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-9*

GENERAL INFORMATION

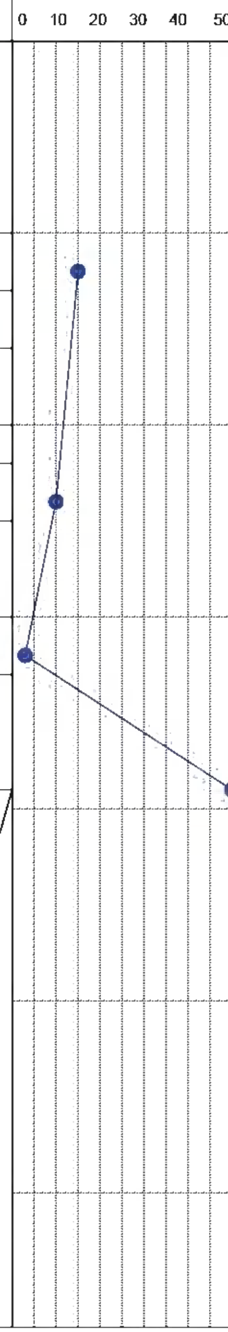
PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/4/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *551250.737*
EASTING: *2498317.159*
GROUND ELEVATION: *11.04 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
10	0				Recovery located at toe of dike
	-5	Medium dense, brown to light gray SAND(SP) with some fine gravel	2-9-6	6"	Attempted Shelby tube at 4 ft bgs. No advancement or recovery, tube bent.
			ST-1	24"	MC=21.9%; Gravel=6.2%; Sand=80.5%; Fines=13.53
0	-10	Medium dense, tan to gray, slightly silty SAND (SM)	2-3-7	16"	
-5	-15	Very loose, gray, silty SAND (SM) w/ interbedded shells	2-1-2	18"	MC=29.8%; Fines=20.8%
		Very dense, gray, silty SAND (SM) w/ interbedded partially cemented shells			
-10	-20	Refusal at 19.5 ft bgs	50/5"	3"	50 blows/5" (Chicora)
-15	-25				
-20	-30				

N-Value



All depths referenced to ground surface.

Total Depth: 19.5 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-10*

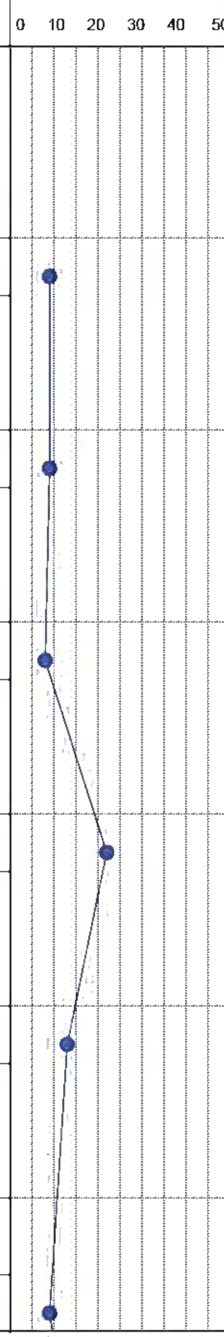
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *2/27/2013*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *550868.544*
EASTING: *2498024.243*
GROUND ELEVATION: *38.86 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	0				Boring located at center of dike.
35	-5	Loose, gray, clayey SAND (SC)	3-3-6	12"	
30	-10	Stiff, gray to brown, sandy CLAY (CL) with trace wood chips	4-4-5	6"	
25	-15	Loose, gray to brown, clayey SAND (SC) with trace wood chips	2-3-5	18"	MC=23.4%; Gravel=0.0%; Sand=49.7%; Fines=50.3%; LL=47; PL=20; PI=27
20	-20	Medium dense, brown to gray, clayey SAND (SC)	3-7-15	12"	MC=21.7%; Fines=41.4%
15	-25	Medium dense, gray, clayey SAND (SC)	6-6-7	18"	
10	-30		ST-1	24"	
		Loose, light gray, fine to medium grained clayey SAND (SC)	2-2-7	18"	MC=16.2%; Fines=26.5%



All depths referenced to ground surface.
Total Depth: 45.8 ft bgs

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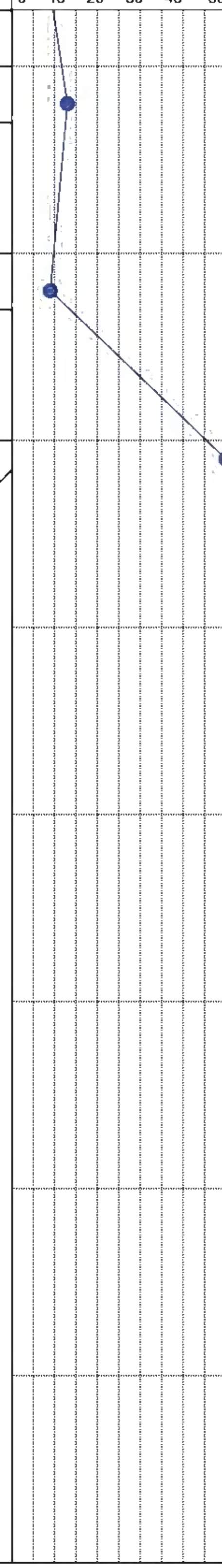
BORING LOG

Borehole ID: *GSB-10*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
5	-35	Medium dense, light gray, fine to medium grained, clayey SAND (SC)	7-6-7	18"	MC=24.4%; Fines=16.3%
0	-40	Loose, gray, clayey SAND (SC) with interbedded shells	2-2-7	18"	
-5	-45	Dense, gray, clayey SAND (SC) with rock and shell fragments Refusal at 45.8 ft bgs	50/3"	3"	50 blows/3"; MC=20.4%; Fines=16.0% (Chicora)
-10	-50				
-15	-55				
-20	-60				
-25	-65				
-30	-70				
-35	-75				

N-Value



All depths referenced to ground surface.

Total Depth: 45.8 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-11*

GENERAL INFORMATION	TECHNICAL INFORMATION
PROJECT NAME: *Winyah Generating Station*	**DRILLING METHOD:** *Rotary Wash*
PROJECT NO: *GSC5242*	**RIG TYPE:** *CME 550X*
SITE LOCATION: *Georgetown, South Carolina*	**BOREHOLE DIA:** *4"*
BORING DATE: *2/27/2013*	**SAMPLING METHOD:** *SPT w/ split spoon*
GEOSYNTEC REPRESENTATIVE: *D. Hanley*	**NORTHING:** *550344.335*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*	**EASTING:** *2497857.101*
DRILLER NAME: *M. Grimball*	**GROUND ELEVATION:** *37.78 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
35	0						Boring located at center of dike.
30	-5	Very loose, brown and gray, clayey SAND (SC)		WOH-WOH-WOH		6"	
25	-10	Very loose, brown and gray, clayey SAND (SC)		WOH-WOH-WOH		6"	MC=34.3%; Gravel=1.4%; Sand=50.0%; Silt=8.8%; Clay=39.8%; Fines=48.6%; LL=47; PL=17; PI=30
20	-15	Very loose, brown and gray, clayey SAND (SC)		WOH-WOH-WOH		6"	MC=30.4%; LL=44; PL=18; PI=26
	-20			ST-1		14"	
15		Very loose, gray, clayey SAND (SC)		WOH-WOH-WOH		6"	
10	-25	Very loose, brown to gray, clayey SAND (SC)		WOR-WOR-WOR			MC=32.0%; Gravel=0.0%; Sand=58.2%; Fines=41.8%; LL=44; PL=15; PI=29
5	-30	Very loose, gray, clayey SAND (SC)		WOR-WOR-WOR		6"	

All depths referenced to ground surface.

Total Depth: 49.7 ft bgs

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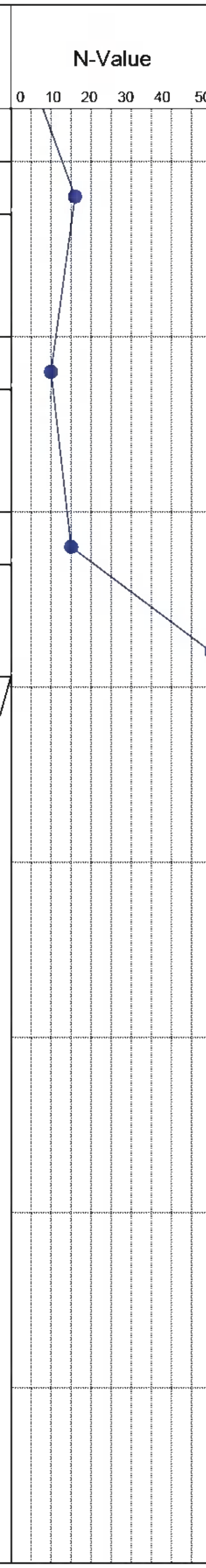
BORING LOG

Borehole ID: *GSB-11*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	-35	Medium dense, black, clayey sand (SC), potentially CCR		3-6-10	18"	MC=83.5%; Gravel=2.0%; Sand=57.4%; Silt=19.6%; Clay=21.0%; Fines=40.6%; LL=114; PL=88; PI=26; Slight H2S Odor
0						
	-40	Medium dense, tan, clayey fine SAND (SC)		2-5-5	18"	MC=168.5%; Fines=49.5%;
-5						
	-45	Medium dense, tan, clayey fine SAND (SC)		2-5-10	18"	MC=140.6%; Fines=42.6%;
-10					0"	
		Refusal at 49.7 ft bgs.		50/0"		Hard drilling at 49.7 ft bgs. 50 blows/0" (Chicora)
	-50					
-15						
	-55					
-20						
	-60					
-25						
	-65					
-30						
	-70					
-35						
	-75					



All depths referenced to ground surface.

Total Depth: 49.7 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-11A*

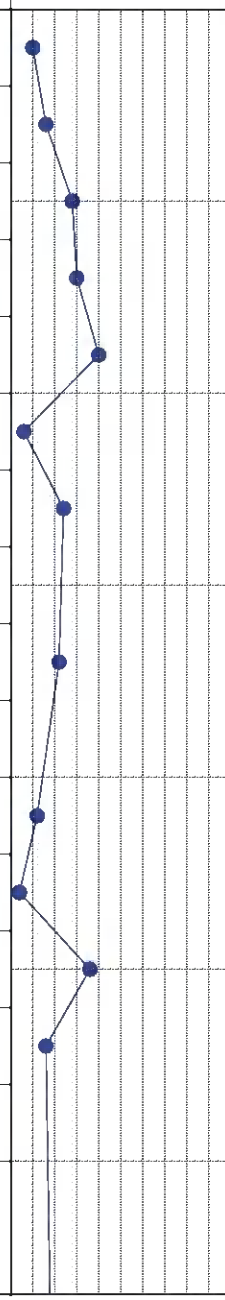
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/30/2016*
GEOSYNTEC REPRESENTATIVE: *A. Sivashanthan*
DRILLING CONTRACTOR: *Carolina Drilling, Co.*
DRILLER NAME: *J. Anderson*

TECHNICAL INFORMATION

DRILLING METHOD: *Mud Rotary*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *550302.83800*
EASTING: *2497846.43885*
GROUND ELEVATION: *37.78 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	Recovery	Comments
	0	Loose, orange, clayey SAND (SC), dry		1-2-3-5	10"	MC=17%
35		Loose, orange with gray pockets, clayey SAND (SC), moist		4-3-5-5	13"	MC=19%
	-5	Medium dense, orange with gray with brown, clayey SAND (SC), moist		8-6-8-9	20"	MC=20%
30		Medium dense, orange with gray with brown, clayey SAND (SC), moist		5-6-9-11	17"	MC=20% Water level was measured 8.2 ft bgs at 8.50 am on 03/31/2016
		Medium dense, yellowish brown and gray, fine SAND (SP), moist		8-11-9-8	19"	MC=27%
	-10	Very loose, yellowish brown and tan, clayey SAND (SC), moist		1-1-2-3	12"	MC=27%
25		Medium dense, orange to black, clayey SAND (SC), moist		4-7-5-5	17"	MC=23% Shelby tube pushed from 14.0 to 16.0 ft bgs.
				ST-1	24"	
	-15					
		Stiff, orange and mottled, sandy CLAY (CL), moist		4-5-6-9	9"	MC=21% Shelby tube pushed from 18.0 to 20.0 ft bgs.
20				ST-2	24"	MC=18%; Gravel=0.0%; Sand=61.2%; Fines=39%; LL=36; PL=13; PI=23;
	-20	Firm, gray, brown, and yellow, CLAY with sand (CL), moist		2-2-4-5	17"	MC=25%
15		Soft, yellow and dark brown, sandy CLAY (CL), moist		1-1-1-2	3"	MC=39%
	-25	Medium dense, orange to black and gray, clayey SAND (SC), moist		8-9-9-8	15"	MC=18%
		Loose, orange to black and gray, clayey SAND (SC), moist		4-4-4-4	24"	MC=26%
10				ST-3	24"	Shelby tube pushed from 28.0 to 30.0 ft bgs.
	-30					
5						

N-Value


All depths referenced to ground surface.
Total Depth: 50.0 ft bgs

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BORING LOG

Borehole ID: *GSB-11A*

Project No: *GSC5242*

Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	Loose, orange and gray to black, clayey SAND to SAND (SP), moist	2-4-5	10"	
		ST-4	0	MC=23%
	Soft, gray, sandy CLAY (CL), moist	2-1-2	17"	MC=28%
	Loose, light brown and tan, coarse SAND (SP) with clay, with shell and rock fragments	2-2-3	18"	MC=34%
		ST-4	0	Shelby tube pushed from 44.0 to 46.0 ft bgs.
	Very dense, gray, rock fragments, moist Boring was terminated at 50 feet.	50/0"	0.5"	MC=12% 50 blows /3" (Chicora)

Elev. (ft NGVD29): 0, -5, -10, -15, -20, -25, -30, -35

Depth (ft): -35, -40, -45, -50, -55, -60, -65, -70, -75

N-Value: 0, 10, 20, 30, 40, 50

All depths referenced to ground surface.

Total Depth: 50.0 ft bgs

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BORING LOG

BOREHOLE ID: *GSB-12*

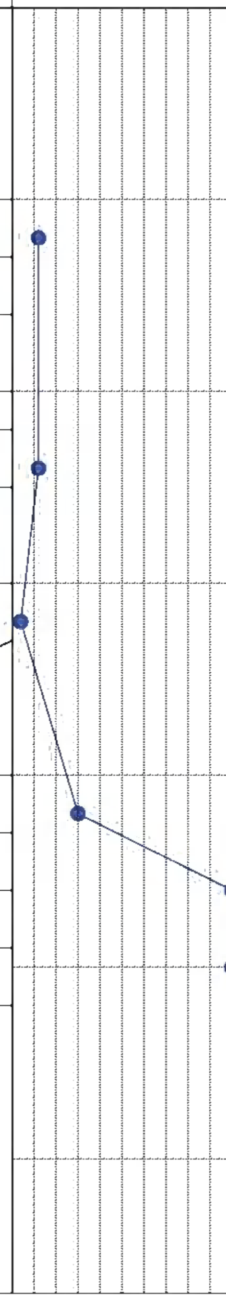
GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *3/4/2013*
GEOSYNTEC REPRESENTATIVE: *J. McNash*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*
DRILLER NAME: *M. Grimball*

TECHNICAL INFORMATION

DRILLING METHOD: *Rotary Wash*
RIG TYPE: *CME 550X*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *550278.856*
EASTING: *2497716.803*
GROUND ELEVATION: *12.31 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
10	0						Boring located at toe of dike
5	-5	Loose, brown to gray, clayey SAND (SC)		2-3-3		18"	
				ST-1		20"	
0	-10						MC=22.9%; Fines=9.6%
		Loose, tan, medium to fine silty SAND (SM)		3-3-3		6"	
-5	-15	Very loose, gray, clayey SAND (SC) with shells at bottom		WOR-WOR-2		18"	MC=22.9%; Gravel=0.3%; Sand=84.0%; Fines=15.7%
-10	-20	Medium dense, gray, clayey SAND (SC) with shells		3-12-3		18"	
		Very dense, gray, silty SAND (SM) with shells		11-20-32		12"	MC=11.5%; Fines=22.7%
-15	-25	Very dense, gray, coarse to medium clayey SAND (SC) with partially cemented shells		10-50/3"		9"	50 blows/3" (Chicora)
		Refusal at 26.0 ft bgs					
-20	-30						



All depths referenced to ground surface.
Total Depth: 26.0 ft bgs

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BORING LOG

BOREHOLE ID: *HSA-1*

GENERAL INFORMATION	TECHNICAL INFORMATION
PROJECT NAME: *Winyah Generating Station*	**DRILLING METHOD:** *Hollow Stem Auger*
PROJECT NO: *GSC5242*	**RIG TYPE:** *CME 550X*
SITE LOCATION: *Georgetown, South Carolina*	**BOREHOLE DIA:** *4"*
BORING DATE: *3/5/2013*	**SAMPLING METHOD:** *No Sampling*
GEOSYNTEC REPRESENTATIVE: *J. McNash*	**NORTHING:** *549602.193*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*	**EASTING:** *2500529.564*
DRILLER NAME: *M. Grimball*	**GROUND ELEVATION:** *38.51 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0	Boring not sampled					Boring located at CPT-7
20							
	-5						
15							
	-10						24-hr Water Level at 10 ft bgs
10							
	-15						1-hr Water Level at 16 ft bgs
5							
	-20						
0							
	-25						
-5							
	-30	Boring terminated at 30.0 ft bgs					
-10							

All depths referenced to ground surface.

Total Depth: 30.0 ft bgs

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BORING LOG

BOREHOLE ID: *HSA-2*

GENERAL INFORMATION	TECHNICAL INFORMATION
PROJECT NAME: *Winyah Generating Station*	**DRILLING METHOD:** *Hollow Stem Auger*
PROJECT NO: *GSC5242*	**RIG TYPE:** *CME 550X*
SITE LOCATION: *Georgetown, South Carolina*	**BOREHOLE DIA:** *4"*
BORING DATE: *3/5/2013*	**SAMPLING METHOD:** *No Sampling*
GEOSYNTEC REPRESENTATIVE: *J. McNash*	**NORTHING:** *551266.120*
DRILLING CONTRACTOR: *Soil Consultants, Inc.*	**EASTING:** *2499180.501*
DRILLER NAME: *M. Grimball*	**GROUND ELEVATION:** *38.61 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0	Boring not sampled					Boring located at CPT-24
20							
	-5						1-hr Water Level found Dry
15							
	-10						
10							
	-15						
	▼						24-hr Water Level at 16 ft bgs
5							
	-20						
0							
	-25	Boring terminated at 25.0 ft bgs					
-5							
	-30						
-10							

All depths referenced to ground surface.

Total Depth: 25.0 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-106*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *10/7/2013*
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: *6"*
SAMPLING METHOD: *SPT w/ split spoon*
NORTHING: *549753.9370*
EASTING: *2499185.8013*
GROUND ELEVATION: *38.44 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0						
35							
	-5	Medium dense, dark gray to black, slightly silty, fine SAND (SP), some organics.		4-5-8		12"	
30							
	-10	Firm, dark gray, fine sandy CLAY (CL).		3-3-4		6"	
25							
	-15	Dense, dark gray to brown, silty, fine SAND (SM).		12-13-12		10"	
20							
	-20	Dense, light brown to dark gray, silty, fine SAND (SM).		12-13-12		18"	
15							
	-25	Firm, gray to brown CLAY (CH), very slight sand, trace organics (twigs), medium to high plasticty.		2-3-3		15"	MC=31.4%; Gravel = 0.0%; Sand = 22.3%; Silt=11.3%; Clay=66.3%. LL=63%; PL=20%; PI=43%.
10							
	-30	Dense, gray to brown, fine, clean SAND (SP).		9-15-16		12"	
5							

All depths referenced to ground surface.

Total Depth: 101.50 ft bgs

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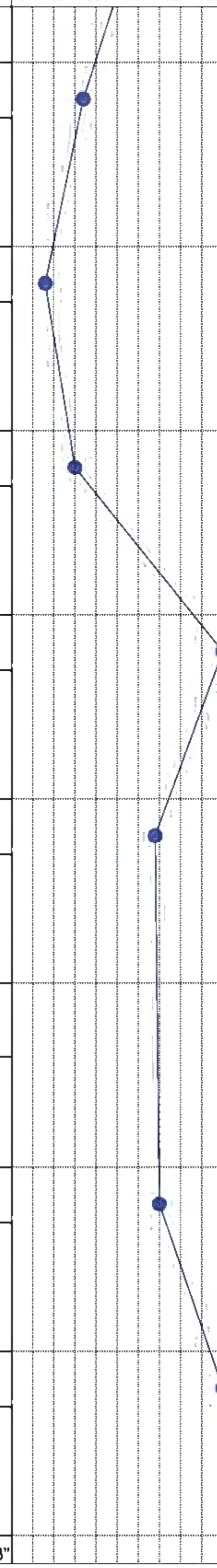
BORING LOG

Borehole ID: *SPT-106*

Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
	-35	Medium dense, gray to brown, fine SAND (SP), slightly silty.	4-7-10	12"	
0	-40	Loose, dark gray to brown, fine SAND (SP).	4-4-4	6"	MC=26.2%; Fines=6.9%.
-5					Hard drilling between 43.00 and 44.50 ft bgs. (Chicora)
	-45	Medium dense, gray, silty, fine SAND (SM).	5-5-10	12"	Hard drilling between 46.50 and 47.50 ft bgs. (Chicora)
-10	-50	Very dense, gray to light gray, soft, silty SAND (SM), with a 0.5" cemented fragement.	5-7-50/5"	6"	N-value = 57 blows/ft.
-15	-55	Hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy, some interbeded shells (Williamsburg Formation Clay).	15-16-18	18"	
-20	-60		ST-1	14"	Shelby Tube advanced 24" by means of a Pitcher Sampler from 60.00 to 62.00 ft bgs. MC=40.5%; k=4.5E-8 cm/s Switch to a 4" bit at 62.00 ft bgs.
-25	-65	Hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	26-18-17	15"	
-30	-70	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	17-22-28	15"	
-35	-75	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation	21-32-50/3"	13"	N-value = 82 blows/ft.

N-Value



All depths referenced to ground surface.

Total Depth: 101.50 ft bgs

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BORING LOG

Borehole ID: *SPT-106*
Project No: *GSC5242*

Elev. (ft NGVD29)	Depth (ft)	Lithologic Description	SPT Blows	Recovery	Comments
		Clay).			MC=42.2%;Gravel=8.8%; Sand=17.2%; Silt=32.7%; Clay=41.3%. LL=69; PL=31; PI=38.
-40					
	-80	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	19-37-50/3"	15"	N-value = 87 blows/ft.
-45					
	-85	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	22-32-38	15"	N-value = 70 blows/ft.
-50					
	-90	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	24-26-50	5"	N-value = 76 blows/ft.
-55					
	-95	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay).	19-37-34	18"	N-value = 71 blows/ft. MC=45.3%; Gravel=0.6%; Sand=25.7%; Silt=35.6%; Clay=38.1%. LL=61; PL=30; PI=31.
-60					
	-100	Very hard, dark gray CLAY (CH), dry, high plasticity, slightly sandy (Williamsburg Formation Clay). Boring terminated at 101.50 ft bgs.	17-35-39	18"	N-value = 74 blows/ft. Borehole did not collapse prior to abandonment. Tremie grouted from 101.50 ft bgs.
-65					
	-105				
-70					
	-110				
-75					
	-115				

N-Value scale: 0 10 20 30 40 50

All depths referenced to ground surface.
Total Depth: 101.50 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-106A*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *11/26/2013*
GEOSYNTEC REPRESENTATIVE: *Weston Shin*
DRILLING CONTRACTOR: *Mid Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Hollow Stem Auger*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *Shelby Tube with Piston Sampler*
NORTHING: *550054.560*
EASTING: *2499454.253*
GROUND ELEVATION: *~34.00 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0						
30	-5	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.					
25	-10			S-1		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 8.00 to 10.00 ft bgs. MC=25.8%; Gravel=0.0%; Sand=0.7%; Silt=92.9%; Clay=6.4%
20				S-2		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 12.00 to 14.00 ft bgs.
	-15	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-3		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 16.00 to 18.00 ft bgs. Gravel=0.0%; Sand=0.2%; Fines=99.8%.
15	-20			S-4		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 20.00 to 22.00 ft bgs.
		Boring terminated at 22.00 ft bgs.					
10	-25						
5	-30						

All depths referenced to ground surface.
Total Depth: 22.00 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-106B*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *11/25/2013*
GEOSYNTEC REPRESENTATIVE: *Weston Shin*
DRILLING CONTRACTOR: *Mid-Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Hollow Stem Auger*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *Shelby Tube with Piston Sampler*
NORTHING: *549087.910*
EASTING: *2499968.775*
GROUND ELEVATION: *~34.00 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
34	0						
30	-5						
		Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-1		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 6.00 to 8.00 ft bgs. MC=56.2%; Gravel=0.0%; Sand=5.9%; Fines=94.1%.
25	-10			S-2		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 10.00 to 12.00 ft bgs.
20	-15	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-3		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 14.00 to 16.00 ft bgs. Gravel=0.0%; Sand=0.7%; Silt=74.3%; Fines=94.1%.
15				S-4		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 18.00 to 20.00 ft bgs.
	-20	Boring terminated at 20.00 ft bgs.					
10	-25						
5	-30						

All depths referenced to ground surface.

Total Depth: 20.00 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-107A*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *11/26/2013*
GEOSYNTEC REPRESENTATIVE: *Weston Shin*
DRILLING CONTRACTOR: *Mid-Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Hollow Stem Auger*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *Shleby Tube with Piston Sampler*
NORTHING: *549785.095*
EASTING: *2498232.382*
GROUND ELEVATION: *~34.00 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0						
30	-5						
		Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-1		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 6.00 to 8.00 ft bgs. Gravel=0.0%; Sand=1.9%; Silt=89.8%; Clay=8.3%.
25	-10			S-2		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 10.00 to 12.00 ft bgs.
20	-15	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-3		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 14.00 to 16.00 ft bgs. Gravel=0.0%; Sand=0.6%; Fines=99.4%.
				S-4		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 18.00 to 20.00 ft bgs.
15	-20	Boring terminated at 20.00 ft bgs.					
10	-25						
5	-30						

All depths referenced to ground surface.

Total Depth: 20.00 ft bgs

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BORING LOG

BOREHOLE ID: *SPT-107B*

GENERAL INFORMATION

PROJECT NAME: *Winyah Generating Station*
PROJECT NO: *GSC5242*
SITE LOCATION: *Georgetown, South Carolina*
BORING DATE: *11/27/2013*
GEOSYNTEC REPRESENTATIVE: *Weston Shin*
DRILLING CONTRACTOR: *Mid-Atlantic Drilling, Inc.*
DRILLER NAME: *B. Fowler*

TECHNICAL INFORMATION

DRILLING METHOD: *Hollow Stem Auger*
RIG TYPE: *CME 45C*
BOREHOLE DIA: *4"*
SAMPLING METHOD: *Shelby Tube with Piston Sampler*
NORTHING: *549126.015*
EASTING: *2499299.918*
GROUND ELEVATION: *~34.00 ft NGVD29*

Elev. (ft NGVD 29)	Depth (ft)	Lithologic Description	Pattern	SPT Blows	N-Value (0 10 20 30 40 50)	Recovery	Comments
	0						
30	-5						
25	-10	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-1		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 8.00 to 10.00 ft bgs. Gravel=0.0%; Sand=0.3%; Fines=99.7%.
				S-2		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 12.00 to 14.00 ft bgs.
20	-15	Gray to dark gray, sandy SILT (ML) (FGD Residuals), low plasticity, wet.		S-3		24"	Shelby Tube advanced by means of a Piston Sampler 24" from 16.00 to 18.00 ft bgs. Gravel=0.0%; Sand=0.1%; Silt=88.7%; Clay=11.2%. LL=NP; PL=NP; PI=NP.
		Boring terminated at 18.00 ft bgs.					
15	-20						
10	-25						
5	-30						

All depths referenced to ground surface.
Total Depth: 18.00 ft bgs

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ATTACHMENT 2-B

S&ME Boring Logs

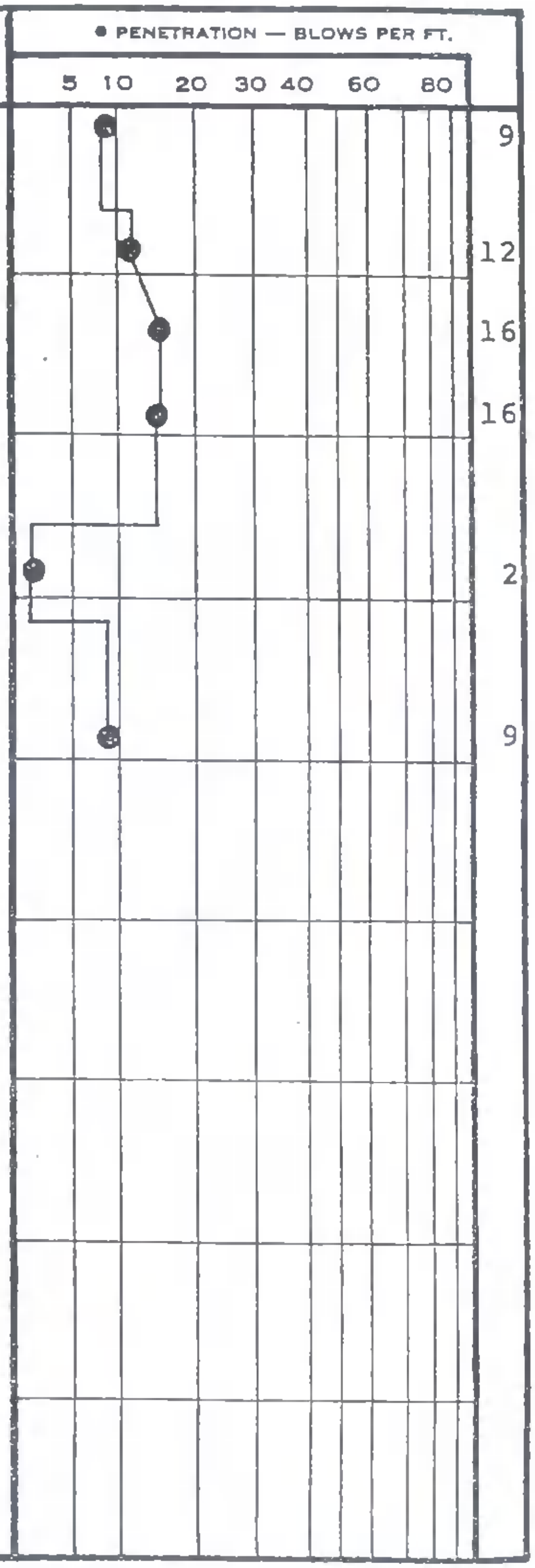
SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.	● PENETRATION — BLOWS PER FT.
		17.5	
0	Loose Black Silty Fine to Medium SAND With Roots*		9
	Stiff Brown Gray Fine Sandy Silty CLAY	14.5	12
5–10	Firm Tan Gray Silty Fine to Medium SAND With Gray Silty Clay Seams		16 16
15	Very Soft Gray Fine Slightly Sandy Silty CLAY With Shells	4.5	2
20	Loose Gray Silty Clayey SHELLS	1.5	9
25	Boring Terminated Encountered Shell Limestone	-5.5	

Water Level @ 1.5' @ TOB

*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.

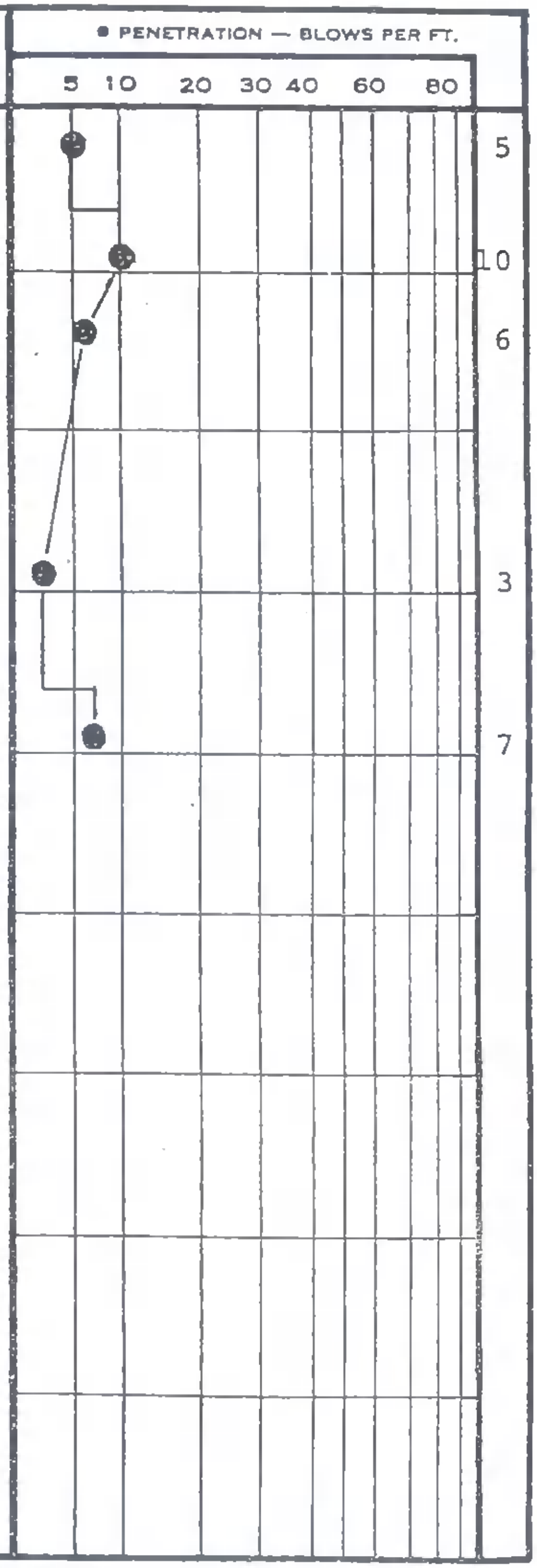
WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-20
DATE DRILLED 12-19-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 15.8
	Silty Fine SAND	
	Firm Gray Brown Fine to Medium Sandy Silty CLAY	12.8
5, 10, 15	Loose to Very Loose Brown Gray Clayey Silty Fine SAND with Occasional Shells	-2.2
20	Shells with Fine to Medium SAND and Silty Clayey Seams	-7.7
25	Boring Terminated Encountered Shell Limestone Water Level 2' @ TOB	
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-21

DATE DRILLED 1-24-78

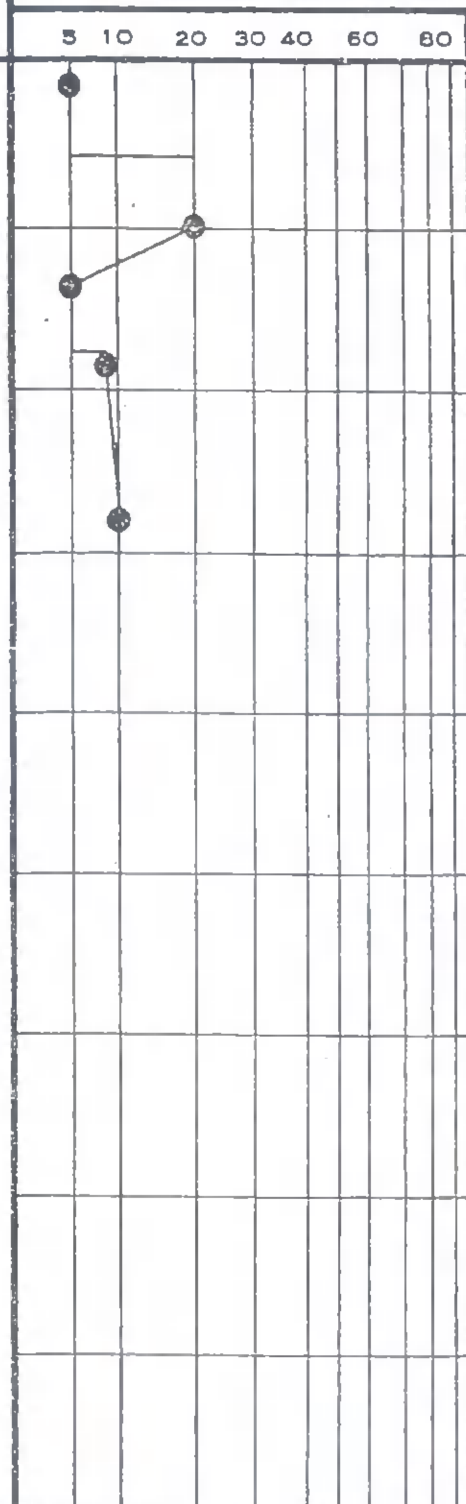
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.	● PENETRATION — BLOWS PER FT.
		9.0	
	Firm Brown Fine Sandy Very Silty CLAY		5
		6.0	
5	Loose to Firm Black Gray Slightly Silty Fine to Medium SAND		20
			5
		0.0	
10	Loose Gray Silty Fine to Coarse Sandy SHELLS		8
15			10
		-9.3	
20	Boring Terminated Encountered Shell Limestone Water Level @ Surface @ TOB		
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.

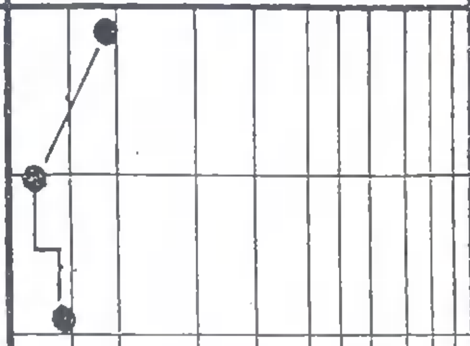
[50] % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

BORING NO. SC-22
DATE DRILLED 12-16-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 5.7	● PENETRATION — BLOWS PER FT. (5 10 20 30 40 60 80)
0	Tan Silty Fine SAND	(water table — 1 hr.)		9
	Loose Brown Silty Fine to Medium SAND	(undisturbed sample)		
5	Very Loose Brown Gray Slightly Silty Fine to *		.7	2
	Very Loose Gray Slightly Clayey Fine to Medium **		-1.8	4
10	Boring Terminated Encountered Shell Limestone		-6.2	
15	Water Level @ Surface @ TOB			
20				
25				
30	*Medium SAND ** 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.

WATER TABLE — 24 HR.

UNDISTURBED SAMPLE — WATER TABLE — 1 HR.

(30) % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

BORING NO. SC-23

DATE DRILLED 1-24-78

JOB NO. SS7735

PAGE 1 OF 1

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SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV.	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
			17.9	
.5	Organic Silty Sandy Top-soil			
	Red Gray Tan Silty Clayey Fine to Medium SAND			
5				
	White Tan Fine to Medium Silty SAND			
10			7.9	
	Pit Terminated Water Level @ 1' @ TOB			
15	Bag Sample 3 - 4'			
	Bag Sample 6 - 8'			
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-25

DATE DRILLED 12-15-77

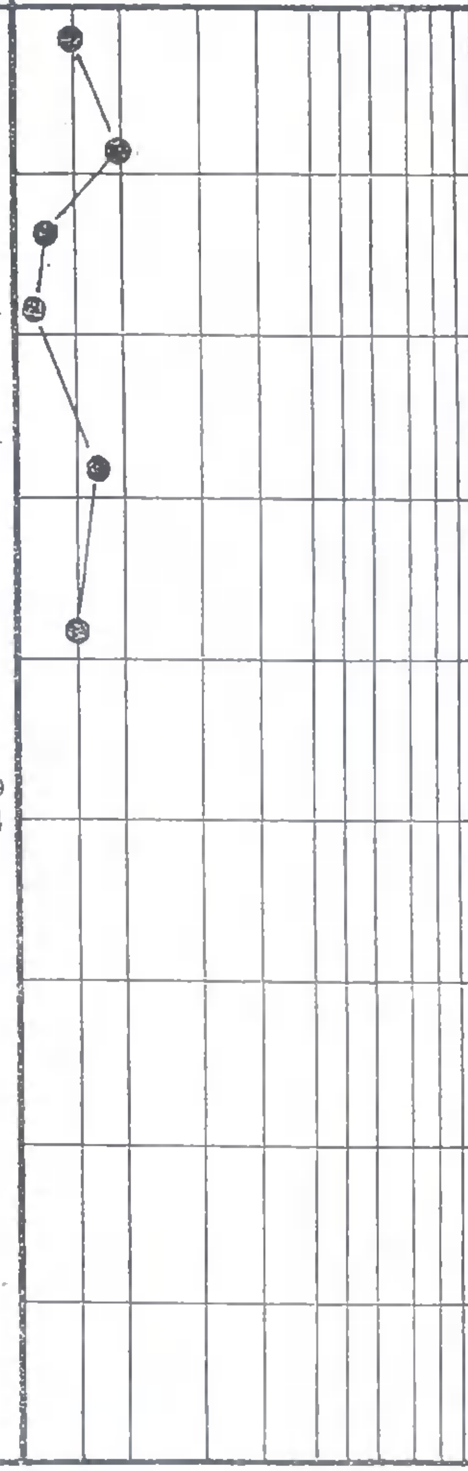
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 17.1	● PENETRATION — BLOWS PER FT.
0–9	Loose to Very Loose Gray Brown Silty Fine SAND	8.1	5, 10, 3, 2
9–13	Very Loose Gray Clayey Silty Fine SAND	4.1	
13–24	Loose Gray Fine to Coarse SAND and Shells	-6.9	7, 5
24.5	Boring Terminated Encountered Shell Limestone Water Level 2'1" @ TOB	-7.4	



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-26

DATE DRILLED 1-24-78

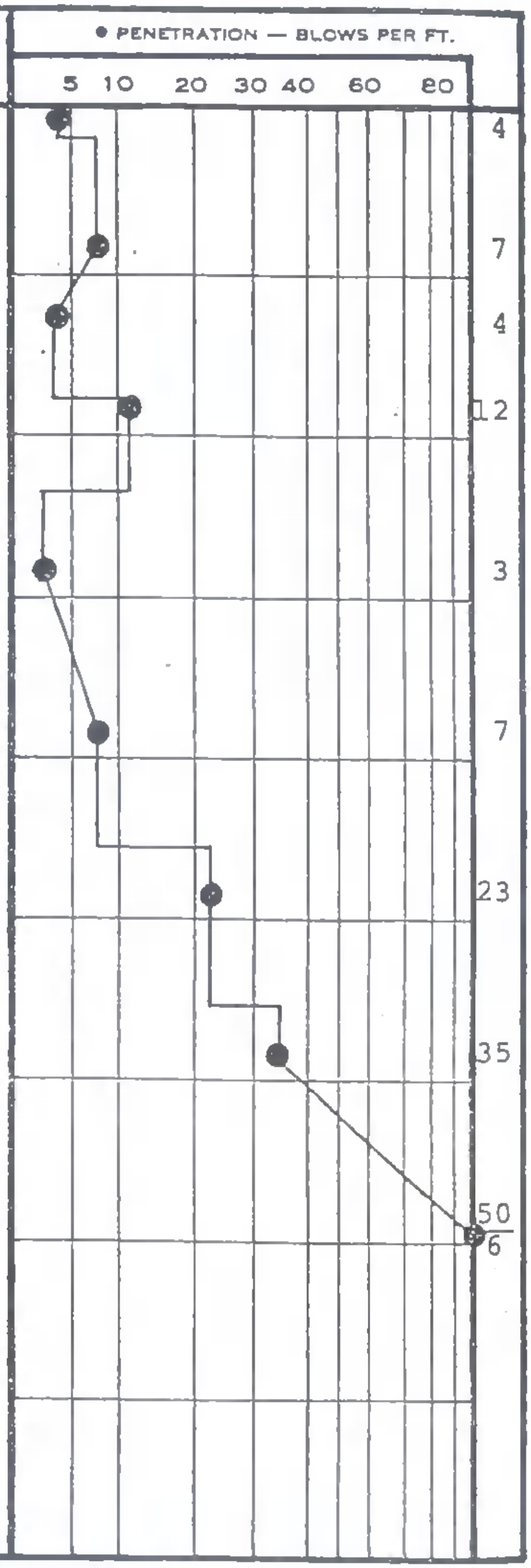
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 16.4	● PENETRATION — BLOWS PER FT.
	Loose Black Silty Fine SAND With Roots		4
5	Soft Gray Fine Sandy Very Silty CLAY With Organic Material		7 4
10	Firm Gray Slightly Silty Fine Coarse Medium SAND		12
15	Very Loose Gray Silty Fine to Medium SAND With Clayey Silt Seams		3 7
20	Loose Gray Fine to Medium Sandy SHELLS		
25	Very Stiff Dark Gray Fine Slightly Sandy Silty CLAY		23
30	Hard Dark Gray Fine Slightly Sandy Silty CLAY With Medium Fine Sandy Seams (Cemented)		35
35			50/6
	Boring Terminated Water Level @ 1' @ TOB NOTE: Boring offset 15' because of water		
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-27
DATE DRILLED 12-19-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

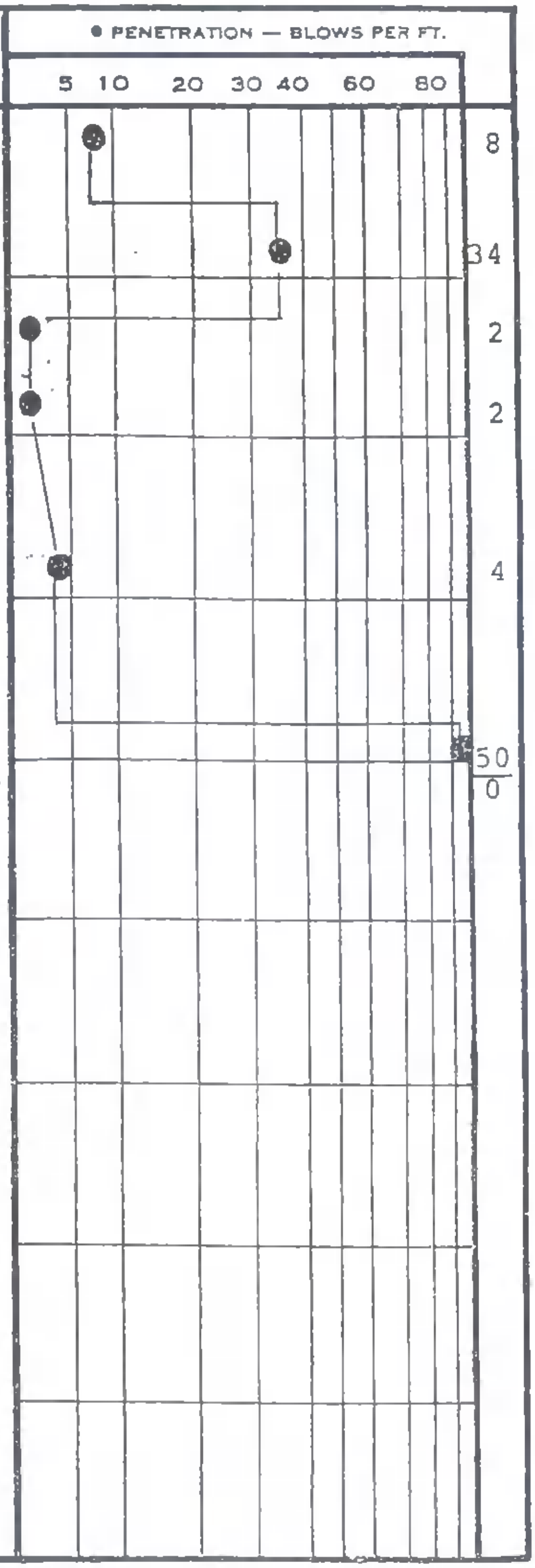
SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.	● PENETRATION — BLOWS PER FT.
		15.3	
0–3	Firm Brown Gray Fine Sandy Silty CLAY		8
3–6.5	Dense Light Gray Fine to Medium SAND	12.3	34
6.5–19	Very Loose Gray Medium Fine SAND with Silty Clayey Seams and Many Shells	8.8	2 2 4
19–20.5	Very Soft Gray Silty CLAY*	-3.7	50/0
		-5.2	

Boring Terminated
Encountered Shell Limestone

Water Level 2'3" @ TOB

*with Fine Sandy Seams and Shells



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-28
DATE DRILLED 1-24-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 18.3	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.5	Organic Silty Sandy Topsoil			
	Red Orange Gray Silty Clayey Fine to Medium SAND			
5				
	Tan Orange White Fine to Medium Silty SAND			
10	Dark Gray Green Fine to Medium Silty SAND		6.3	
15	Pit Terminated Water Level @ 2.0' @ TOB Hole Caved Partially Near Edge of Ditch Bag Sample 2-3'			
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

TEST PIT

BORING NO. SC-30

DATE DRILLED 12-15-77

JOB NO. SS7735

PAGE 1 OF 1

DEPTH FT.	DESCRIPTION		ELEV. 16.9	• PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
	Organic Tan Silty Sandy Top-soil			
	Orange Red Gray Slightly Silty Clayey Fine to Medium SAND			
5	Tan White Medium Silty SAND		8.9	
10	Pit Terminated Hole Caved			
	Water @ 2' @ Caving			
15	Bag Sample 2-4'			
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.

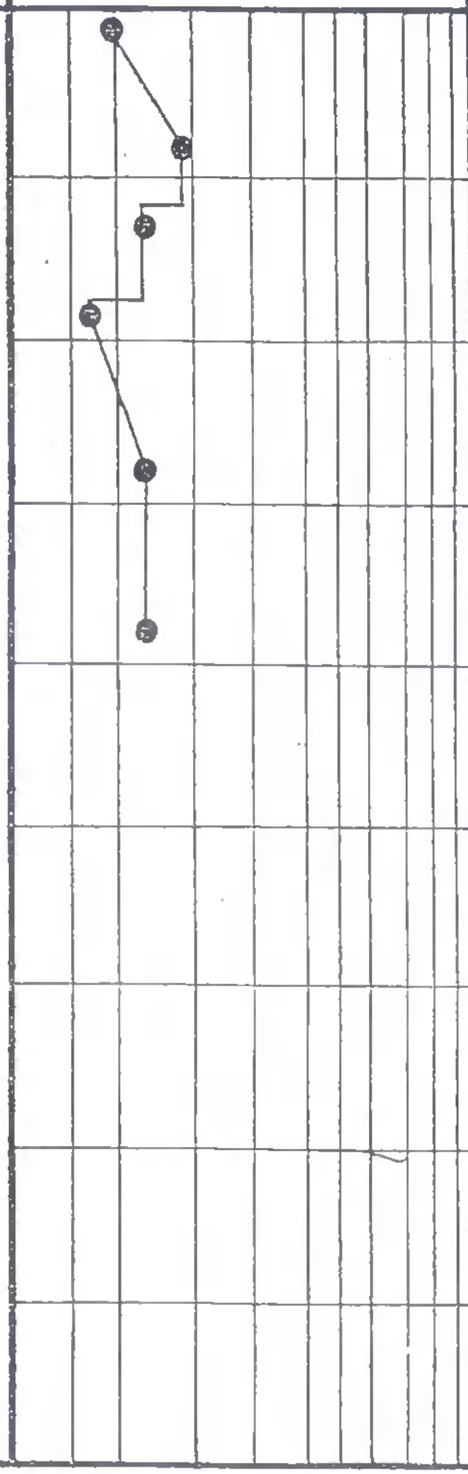
(50) % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

TEST PIT
BORING NO. SC-31
DATE DRILLED 12-15-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 12.7	● PENETRATION — BLOWS PER FT. (5 10 20 30 40 60 80)
0 – 6	Stiff Red Gray Brown Fine Sandy Silty CLAY	6.7	10, 19
6 – 9	Very Stiff to Firm Brown Gray Slightly Silty Fine to Medium SAND	3.7	14
9 – 22	Loose to Firm Gray Fine to Coarse SAND With Shells and Shell Limestone Fragments	-9.3	7, 14, 14
	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.

UNDISTURBED SAMPLE — WATER TABLE — 24 HR. — WATER TABLE — 1 HR.

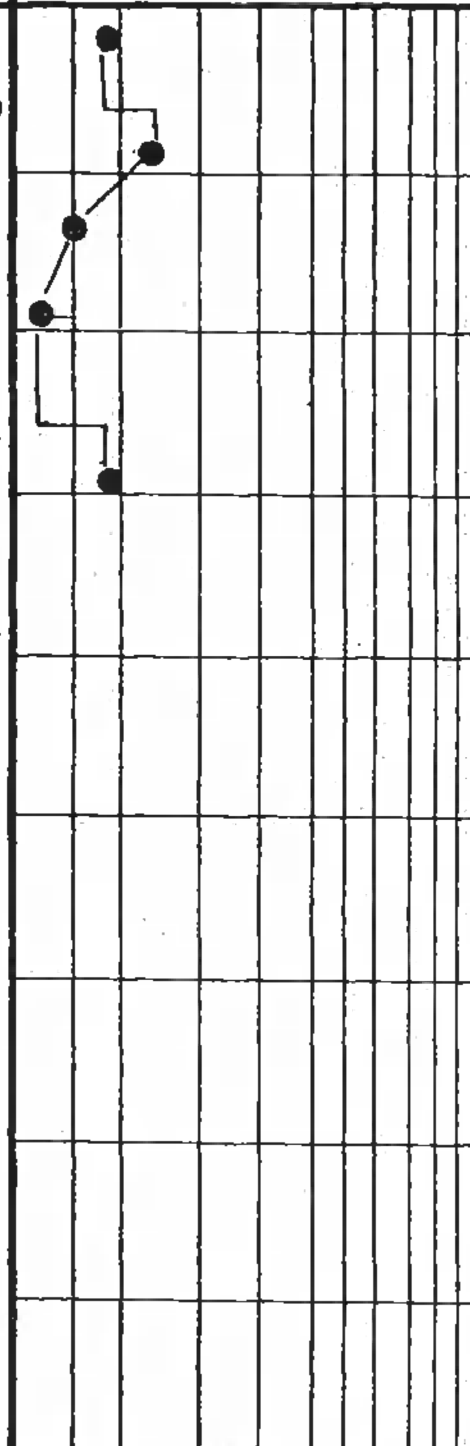
[50] % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

BORING NO. SC-32
DATE DRILLED 12-16-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG • RALEIGH • NORFOLK • GREENSBORO • ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 10.9	• PENETRATION — BLOWS PER FT. (5 10 20 30 40 60 80)	
0–3	Stiff Brown Gray Fine Sandy Silty CLAY	7.9		9
3–13	Loose Red Gray Brown Clayey Silty Fine to Medium SAND	-2.1		13 5 3
13–17	Shells With Fine to Medium SAND	-6.1		9
17	Boring Terminated Encountered Shell Limestone Water Level 2" @ 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-33

DATE DRILLED 1-19-78

JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 15.5	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.7	Organic Roots Silty Sandy Topsoil 0 - 8"			
	Tan Sitly Fine to Medium SAND 8" - 10"			
	Tan Gray Clayey Fine to *			
5	Gray Tan Slightly Clayey Silty Fine to Medium SAND		6.5	
10	Pit Terminated Hole Caved Water @ 3.0'			
15	Note: Bag Samples: 1.5-3.0' 7.0-9.0'			
20				
25	*Medium SAND			
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

TEST PIT

BORING NO. SC-34

DATE DRILLED 12-15-77

JOB NO. SS7735

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SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 17.9	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.6	Organic Topsoil			
	Tan Red Gray Slightly Sandy Silty CLAY	═		
5	Tan Gray Slightly Clayey Silty Fine to Medium SAND		8.9	
10	Pit Terminated Water @ 3.0' Hole Caved @ 3.0' Bag Sample 3.0-4.0'			
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

TEST PIT

BORING NO. SC-36

DATE DRILLED 12-15-77

JOB NO. SS7735

PAGE 1 OF 1

DEPTH FT.	DESCRIPTION		ELEV.	● PENETRATION — BLOWS PER FT. (5 10 20 30 40 60 80)
			15.3	
.8	Organic Silty Sandy Top-soil			
	Organic Tan Slightly Sandy Silty CLAY			
5	White Light Brown Slightly Silty Medium SAND			
	Gray Green Slightly Clayey Medium SAND With Numerous Small Shells			
10			5.3	
	Pit Terminated Water Level @ 1' @ TOB			
15				
	Hole Caved			
20	Note: Clay 0-4' sticks in Backhoe Bucket			
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

TEST PIT

BORING NO. SC-37

DATE DRILLED 12-15-77

JOB NO. SS7735

PAGE 1 OF 1

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 16.2	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.5	Organic Silty Sandy Top-soil	(water table symbol)		
	Orange Gray Slightly Sandy Silty CLAY			
5				
	White Gray Medium Silty SAND			
	Gray Green Slightly Clayey Medium SAND With Numerous Small Shells			
10			6.2	
	Pit Terminated Water @ 1.0' Hole Caved Note: Clay from 1-6' sticks in Backhoe Bucket Bag Samples: 1.5-3.0 Bag Samples: 6.0-7.0			
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. TEST PIT SC-38

DATE DRILLED 12-15-77

JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 15.8	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.6	Organic Silty Sandy Topsoil			
	Orange Gray Slightly Sandy Silty CLAY			
5	Gray Green Silty Clayey Medium SAND	⚌		
10	Gray White Slightly Clayey Medium SAND with Numerous Small Shells		3.8	
15	Pit Terminated Water Level @ 6.0' Hole Caved @ 4.0'			
20	Note: Clay from 1-5 sticks in Backhoe Bucket			
25	Bag Sample: 1.5-3.0 Bag Sample: 6.0-7.0			
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

TEST PIT

BORING NO. SC-39

DATE DRILLED 12-15-77

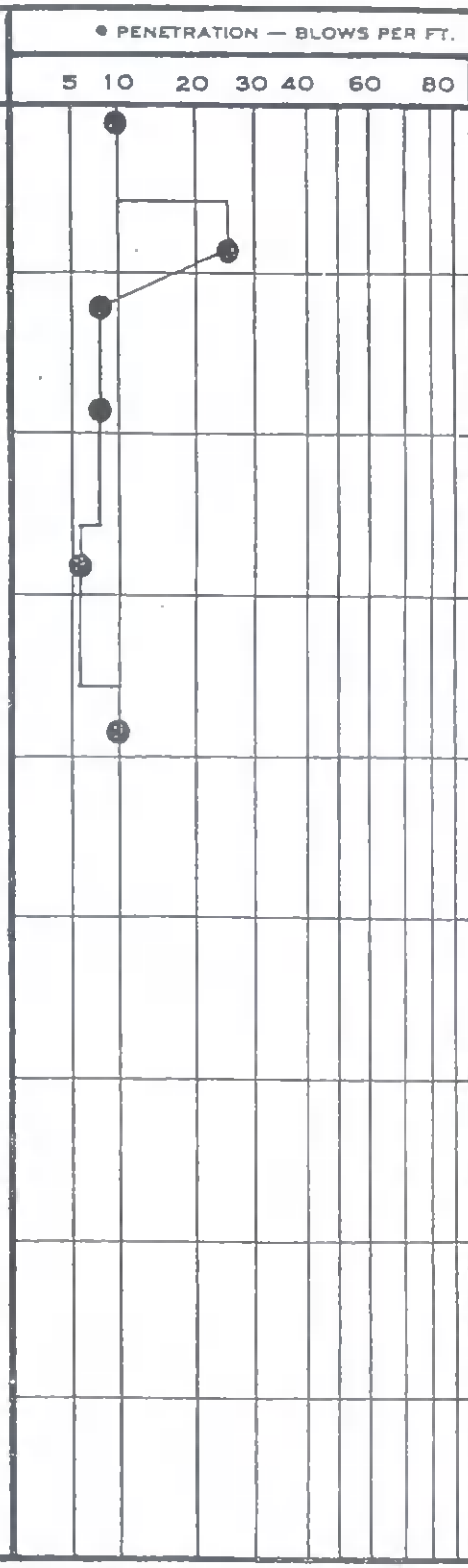
JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 18.2	● PENETRATION — BLOWS PER FT.
0	Loose Gray Black Silty Fine SAND (Organically Stained Wood and Organic Material*		10
		15.2	
5	Firm to Loose Gray Tan Slightly Silty Fine to Medium SAND		26
			8
10			8
		5.2	
15	Firm Dark Gray Fine Sandy Very Silty CLAY With Fine to Medium Sand Seams		6
		0.2	
20	Loose Dark Gray Fine to Medium SAND With Shells		10
		-4.9	
25	Boring Terminated Encountered Shell Limestone		
	Water Level @ 0.5' @ TOB		
30	*With Gray Black Silty Fine SAND		
35			
40			



SOIL BORING RECORD

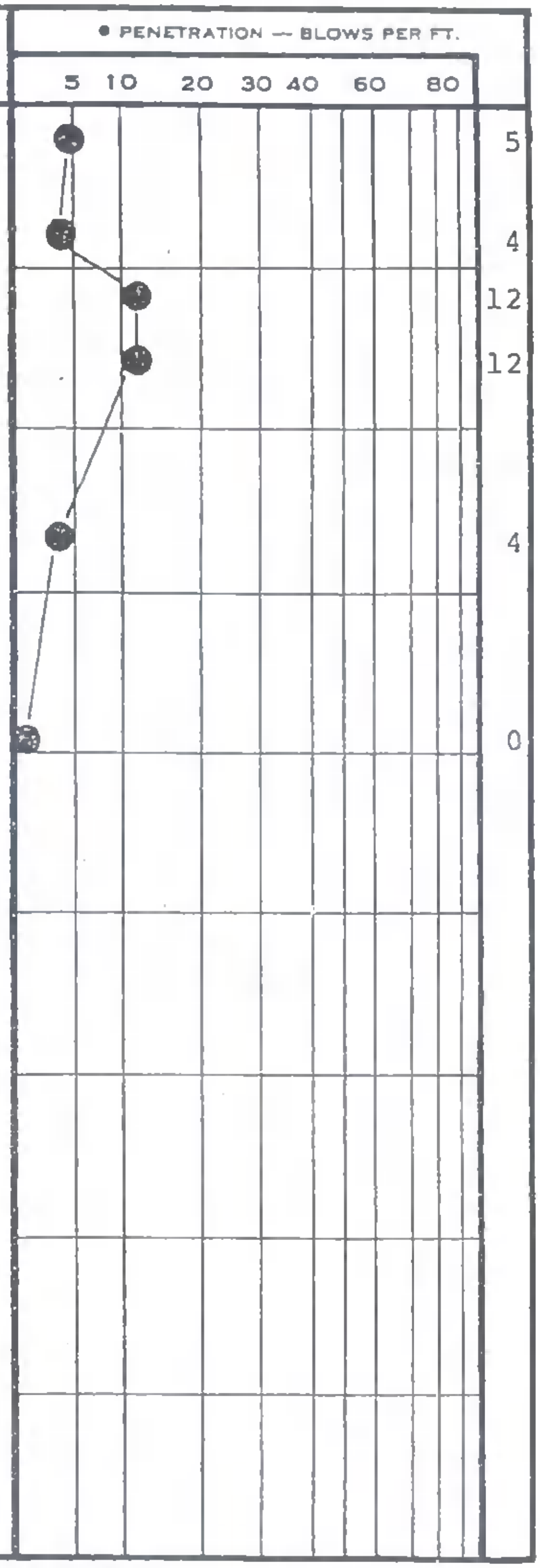
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

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-40
DATE DRILLED 12-19-77
JOB NO. SS7735
PAGE 1 OF 1

DEPTH FT.	DESCRIPTION	ELEV. 17.5
0–6.5	Firm Brown Gray Sandy Silty CLAY	11.0
6.5–14	Firm Gray Silty Fine SAND	3.5
14–20	Very Loose Green Gray Silty Fine SAND with Sandy Seams	-3.5
20–25	Loose Shells and SAND	-7.0
25	Boring Terminated Water Level 1' @ 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.

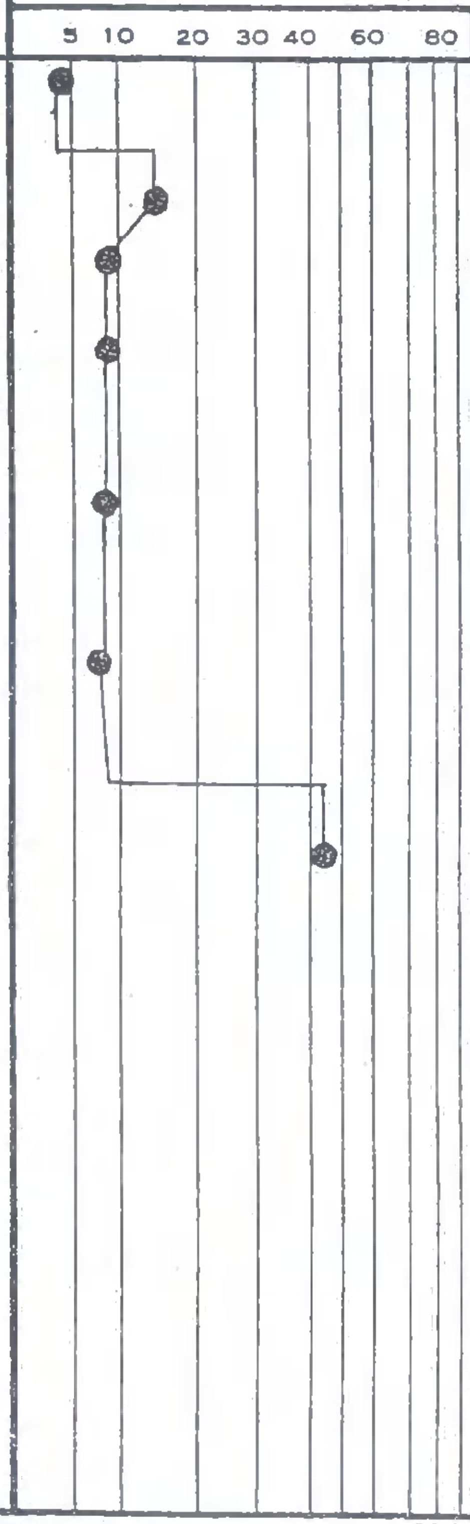
WATER TABLE — 24 HR.
UNDISTURBED SAMPLE
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-41
DATE DRILLED 2-1-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 19.0	● PENETRATION — BLOWS PER FT.
0	Soft Gray and Tan Silty Sandy CLAY		4
		16.0	
5	Firm Tan Gray Silty Fine SAND with Occasional Clay		15
			9
10			10
15			8
20			7
		-4.0	
25	Cemented Sand and Shells		44
		-7.0	
	Boring Terminated		
30	Water Level 3.3" 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.

UNDISTURBED SAMPLE — WATER TABLE — 24 HR. — WATER TABLE — 1 HR.

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

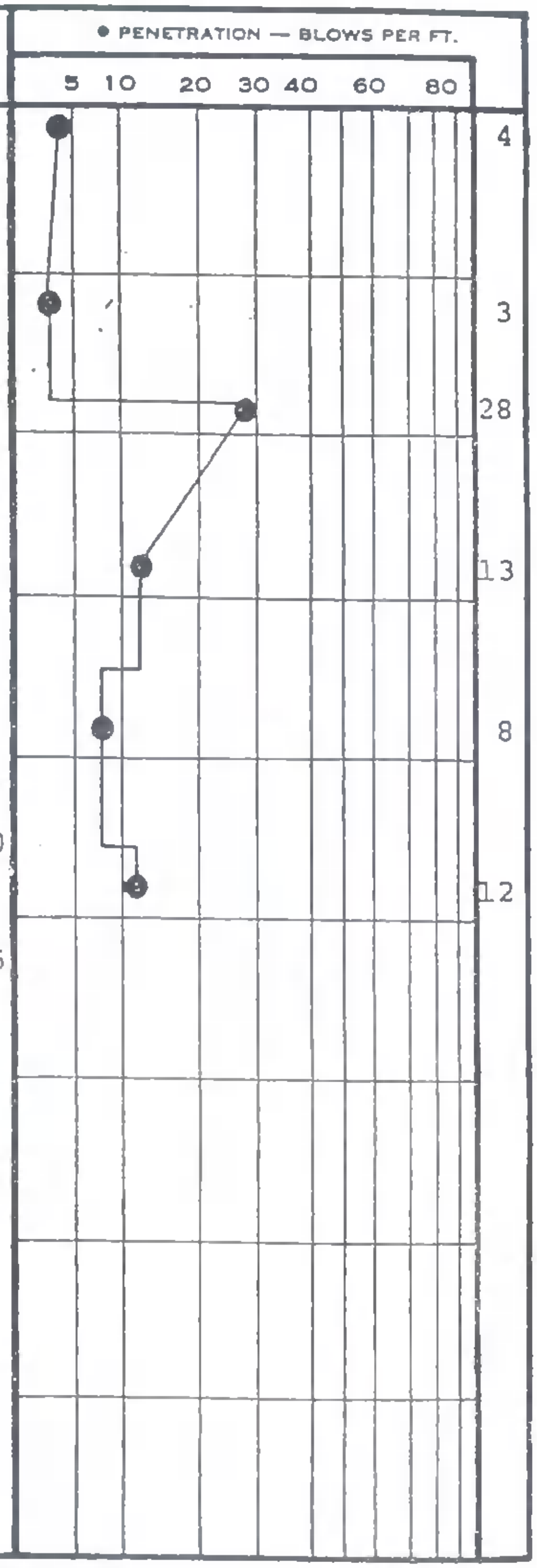
BORING NO. SC42
DATE DRILLED 2-1-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 20.0	BLOWS PER FT.
0	Loose Silty Fine SAND With Roots (Organically Stained)		4
	Soft Gray Brown Fine Sandy Silty CLAY		3
	Very Loose Dark Brown Slightly Silty Fine to Medium SAND	11.0	28
	Firm Gray Fine to Medium SAND		13
	Loose Gray Black Silty Clayey Fine to Medium SAND With Shells	2.5	8
	Firm Gray Slightly Silty Fine to Medium Sandy SHELLS	-3.0	12
	Boring Terminated Encountered Shell Limestone	-6.5	
	Water Level @ 1.0' @ TOB		

Depth scale: 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.

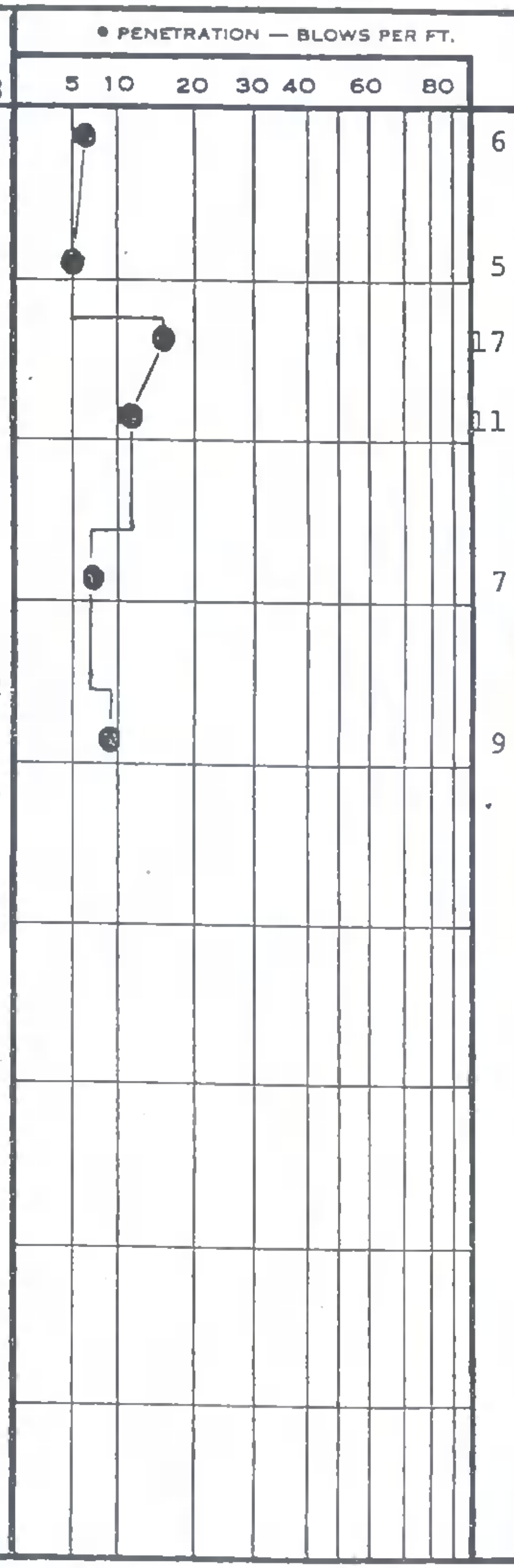
UNDISTURBED SAMPLE
WATER TABLE — 24 HR.
WATER TABLE — 1 HR.
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-43
DATE DRILLED 12-15-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 15.8	PENETRATION — BLOWS PER FT.
0–6.5	Loose Brown Gray Silty Fine SAND	9.3	6, 5
6.5–13	Firm Gray Fine SAND	2.8	17, 11
13–18	Loose Green Gray Silty Fine Medium SAND with Shells and Some Clayey Seams	-2.2	7
18–21.5	Shells and Limestone with Fine Medium SAND	-5.7	9
	Boring Terminated Encountered Red Shell Limestone Water Level 2' @ 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 — WATER TABLE — 24 HR. — WATER TABLE — 1 HR.

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

BORING NO. SC-44
DATE DRILLED 1-24-78
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG • RALEIGH • NORFOLK • GREENSBORO • ATLANTA

DEPTH FT.	DESCRIPTION	ELEV.	PENETRATION — BLOWS PER FT.
		19.2	
0–1	Loose Black Silty Fine SAND*		8
1–3	Firm Gray Brown Fine Sandy Silty CLAY	16.2	
3–9	Firm Gray Tan Silty Fine to Medium SAND		16
			10
		10.2	
9–12	Very Soft Gray Silty CLAY With Fine to Medium Sand Seams		WOH
		7.2	
12–17	Loose Gray Slightly Silty Fine to Medium SAND		6
		2.2	
17–27	Firm Gray Slightly Silty Fine to Medium Sandy SHELLS		13
			17
		-7.8	

Boring Terminated
Encountered Shell Limestone

Water Level @ 2' @ TOB

*With Roots

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.

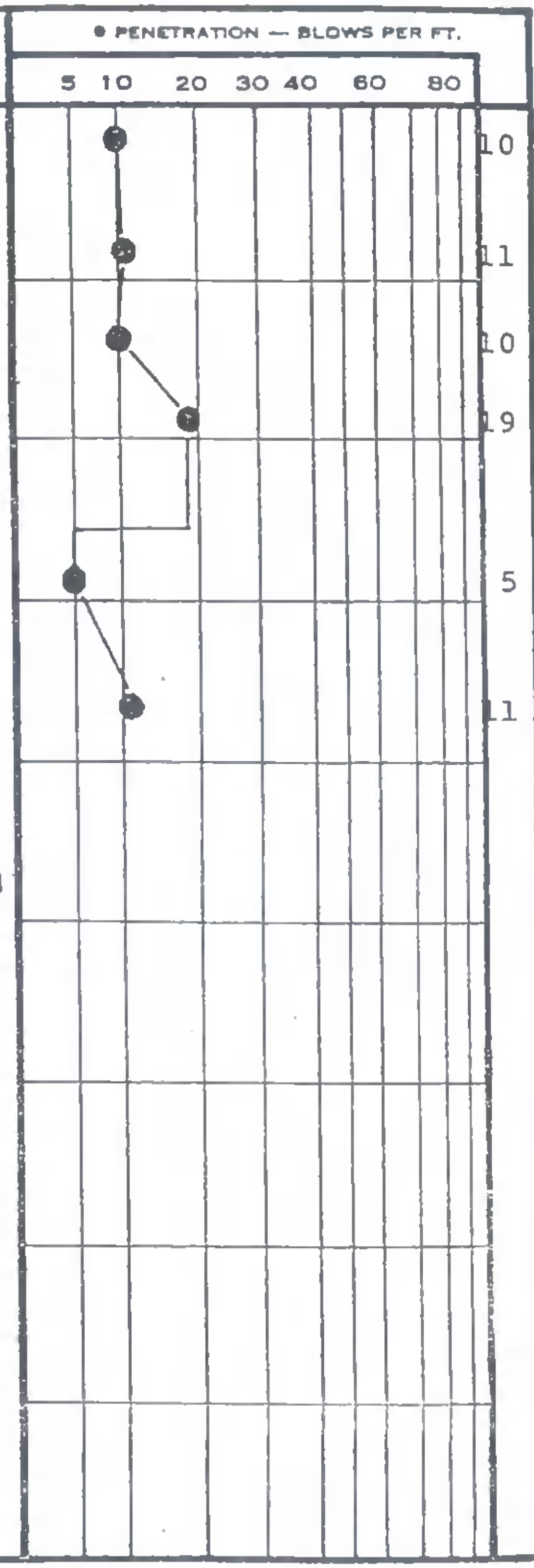
WATER TABLE — 24 HR.
WATER TABLE — 1 HR.
UNDISTURBED SAMPLE
[50] % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. SC-45
DATE DRILLED 12-15-77
JOB NO. SS7735
PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION	ELEV. 18.7
0–3	Stiff Red Gray Brown Fine Sandy Silty CLAY	15.7
3–13	Firm to Loose Gray Brown Silty Fine SAND	5.7
13–18	Firm Gray Silty CLAY with Fine to Medium Sandy Seams	
18–24	Shells with Fine to Medium SAND	-5.3
24	Boring Terminated Encountered Shell Limestone Water Level 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-46
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. 18.7	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
.6	Organic Topsoil-Silty SAND			
	Tan Red Gray Silty Clayey Fine to Medium SAND	(water table — 1 hr. symbol)		
5				
	Brown Gray Fine to Medium Slightly Clayey SAND		9.7	
10				
	Pit Terminated Water @ 3.0' Hole Caved @ 3.0'			
15				
20				
25				
30				
35				
40				

SOIL BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1585
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. TEST PIT SC-46

DATE DRILLED 12-14-77

JOB NO. SS7735

PAGE 1 OF 1

SOIL & MATERIAL ENGINEERS, INC.

SPARTANBURG ● RALEIGH ● NORFOLK ● GREENSBORO ● ATLANTA

DEPTH FT.	DESCRIPTION		ELEV. 14.3	● PENETRATION — BLOWS PER FT. 5 10 20 30 40 60 80
	Gray Red Medium Sandy Silty CLAY			
	Gray Tan Silty Medium SAND			
5				
	Dark Gray Fine to Medium Silty SAND			
10				
15			-0.7	
	Pit Terminated Water Level @ 9.0' @ TOB Hole Caved @ 9.0'			
20				
25	Note: No Topsoil -inside Borrow Pit			
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-56

DATE DRILLED 12-15-77

JOB NO. SS7735

PAGE 1 OF 1

ATTACHMENT 2-C

PCRA Boring Logs

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-201, STA. 254+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES: N ______ E ______
SURFACE EL: 38.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	5-5 7-5 (12)	1.2		CLAYEY SAND, MOTTLED ORANGE, RED, AND GRAY, MEDIUM DENSE TO LOOSE. (FILL) 3.2'	sc	START @ 0830 4/22/99
		S-2	6-4 5-6 (9)	1.4		SANDY CLAY, MOTTLED ORANGE AND GRAY, MEDIUM TO STIFF. (FILL)		PP=1.25 TSF
	5	S-3	4-5 6-10 (11)	1.3			cl	PP= 1.5 TSF
		S-4	2-3 9-18 (12)	1.1		7.0'		PP= 0.75 TSF
						CLAYEY SAND, MOTTLED ORANGE AND GRAY, MEDIUM DENSE. (FILL)		
		S-5	2-6 16-10 (22)	1.7		10.0'	SM	
	10	S-6	2-3 5-7 (8)	1.5		SANDY CLAY, MOTTLED ORANGE, BROWN, RED, AND GRAY, STIFF TO VERY STIFF. (FILL)		PP=1.5 TSF
		S-7	5-4 5-7 (9)	1.5		-0.2' CLAYEY SAND LAYER @ 14.0'	SC	PP=1.5 TSF
	15	S-8	5-4 6-9 (10)	1.7		-MANY WOOD CHIPS 16.0' TO 17.0'.		PP=2.25 TSF
		S-9	2-2 6-8 (8)	2.0		-OCCASIONAL WOOD CHIPS.		
		S-10	2-5 8-9 (13)	2.0				PP=2.75 TSF
	20	S-11	4-3 8-10 (11)	1.6		-WOOD CHIPS 20.5' TO 21.0'		PP=2.5 TSF
		S-12	2-4 7-10 (11)	1.9		23.0'		PP= 4.0 TSF
						CLAYEY SAND, MOTTLED GRY/ BRN, MED DENSE. (FILL)	sc	WOOD @ 23.2'
	25	S-13	5-6 10-11 (16)	1.7		SANDY CLAY, MOTTLED ORG, BRN, GRAY, V. STIFF. (FILL) -WOOD CHIPS THOUGHOUT S-13. 26.0'	cl	PP=2.25 TSF
		S-14	1-5 7-9 (12)	2.0		SANDY CLAY, MOTTLED ORANGE, BROWN, AND GRAY, STIFF TO VERY STIFF.		PP=1.75 TSF
		S-15	2-4 6-8 (10)	2.0		-28.2' TO 29.2' VERY SOFT.	cl	PP=1.5 TSF
	30	S-16	4-4 7-8 (11)	2.0		-MORE RED AND SANDY @ 31.5' 32.0'		PP=2.5 TSF
		S-17	6-11 15-18 (26)			SAND, LIGHT GRAY, MEDIUM DENSE.	sp	
		S-18	2-3-4-5	1.9		-OCCASIONAL VERY SOFT GRAY CLAY LENSES 34.0' TO 35.0'.		

DATE BEGAN: 4/22/99
DATE COMPLETED: 4/22/99
FIELD GEOLOGIST: RMW
CHECKED BY: RMW/JAA
DRILLING CO.: CAROLINA DRILLING

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY IN SOIL
NQ CORE IN ROCK
DRILLER: GERALD HELPER: BRETT

NOTES:
RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-201, STA. 254+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(7)	1.9				
		S-19	4-3 4-3 (7)	2.0		-CLAYEY SAND AND GRAVEL 37.'2-37.4' 37.4'		
		S-20	4-8 6-8 (14)	1.6		CLAYEY SAND, GRAY, MEDIUM DENSE, SHELL FRAGMENTS..		
	40	S-21	4-5 7-8 (12)	1.7			SC	
		S-22	8-8 16-23 (24)	1.5				
	45	S-23 / S-24	16-4 9-17 (13)	1.4		- LIMESTONE ROCK FRAGMENTS 44.0' TO 46.0'. 46.0'		1.75" LIMESTONE FRAGMENT IN TIP.
		R-1	25% (0%)	0.5		LIMESTONE, GRAY, INTERBEDDED SHELLS, WEATHERED, SOFT TO MODERATELY HARD, VERY BROKEN TO SLIGHTLY BROKEN.		-S-24 25/0.0', 0% recovery. -core barrel very tight going in casing may be skewed.
		S-25	65/0.5	0.5		-VERY SOFT, LIGHT GRAY CLAY LENS 47.9' TO 48.0'.		-lost circulation @ 48.0' due to clay in bit. -switch to SPT spoon 48.0' to 48.5. -return to NQ core 48.5.
	50	R-2	68% (9%)	2.7		-BECOMES PROGRESSIVELY SOFTER TOWARD BOTTOM OF RUN R-2. 53.5'		
	55					BOTTOM OF BORING @ 53.5' 4/22/99 15:00		
	60							
	65							

DATE BEGAN: 4/22/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 4/22/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: RMW	DRILLING METHOD: MUD ROTARY SOIL NQ CORE FOR ROCK		
CHECKED BY: RMW/JAA			
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-201, STA. 254+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	5-5 7-5 (12)	1.2		CLAYEY SAND, MOTTLED ORANGE, RED, AND GRAY, MEDIUM DENSE TO LOOSE. (FILL) 3.2'	sc	START @ 0830 4/22/99
		S-2	6-4 5-6 (9)	1.4		SANDY CLAY, MOTTLED ORANGE AND GRAY, MEDIUM TO STIFF. (FILL)	cl	PP=1.25 TSF
	5	S-3	4-5 6-10 (11)	1.3				PP= 1.5 TSF
		S-4	2-3 9-18 (12)	1.1		7.0'		PP= 0.75 TSF
		S-5	2-6 16-10 (22)	1.7		CLAYEY SAND, MOTTLED ORANGE AND GRAY, MEDIUM DENSE. (FILL) 10.0'	SM	
	10	S-6	2-3 5-7 (8)	1.5		SANDY CLAY, MOTTLED ORANGE, BROWN, RED, AND GRAY, STIFF TO VERY STIFF. (FILL)	SC	PP=1.5 TSF
		S-7	5-4 5-7 (9)	1.5		-0.2' CLAYEY SAND LAYER @ 14.0'		PP=1.5 TSF
	15	S-8	5-4 6-9 (10)	1.7		-MANY WOOD CHIPS 16.0' TO 17.0'.		PP=2.25 TSF
		S-9	2-2 6-8 (8)	2.0		-OCCASIONAL WOOD CHIPS.		
		S-10	2-5 8-9 (13)	2.0				PP=2.75 TSF
	20	S-11	4-3 8-10 (11)	1.6		-WOOD CHIPS 20.5' TO 21.0'		PP=2.5 TSF
		S-12	2-4 7-10 (11)	1.9		23.0'		PP= 4.0 TSF
						CLAYEY SAND, MOTTLED GRY/ BRN, MED DENSE. (FILL)	sc	WOOD @ 23.2'
	25	S-13	5-6 10-11 (16)	1.7		SANDY CLAY, MOTTLED ORG, BRN, GRAY, V. STIFF. (FILL) -WOOD CHIPS THOUGHOUT S-13. 26.0'	cl	PP=2.25 TSF
		S-14	1-5 7-9 (12)	2.0		SANDY CLAY, MOTTLED ORANGE, BROWN, AND GRAY, STIFF TO VERY STIFF.		PP=1.75 TSF
		S-15	2-4 6-8 (10)	2.0		-28.2' TO 29.2' VERY SOFT.	cl	PP=1.5 TSF
	30	S-16	4-4 7-8 (11)	2.0		-MORE RED AND SANDY @ 31.5' 32.0'		PP=2.5 TSF
		S-17	6-11 15-18 (26)			SAND, LIGHT GRAY, MEDIUM DENSE.	sp	
		S-18	2-3-4-5	1.9		-OCCASIONAL VERY SOFT GRAY CLAY LENSES 34.0' TO 35.0'.		

DATE BEGAN: 4/22/99 DATE COMPLETED: 4/22/99 FIELD GEOLOGIST: RMW CHECKED BY: RMW/JAA	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked. GWL: DEPTH: DATE/TIME: DRILLING METHOD: MUD ROTARY IN SOIL NQ CORE IN ROCK	NOTES:
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-201, STA. 254+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES: N ________ E ________

SURFACE EL: 38.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(7)	1.9				
		S-19	4-3 4-3 (7)	2.0		-CLAYEY SAND AND GRAVEL 37.'2-37.4' — 37.4'		
		S-20	4-8 6-8 (14)	1.6		CLAYEY SAND, GRAY, MEDIUM DENSE, SHELL FRAGMENTS..	SC	
	40	S-21	4-5 7-8 (12)	1.7				
		S-22	8-8 16-23 (24)	1.5				
	45	S-23 / S-24	16-4 9-17 (13)	1.4		- LIMESTONE ROCK FRAGMENTS 44.0' TO 46.0'. — 46.0'		1.75" LIMESTONE FRAGMENT IN TIP.
		R-1	25% (0%)	0.5		LIMESTONE, GRAY, INTERBEDDED SHELLS, WEATHERED, SOFT TO MODERATELY HARD, VERY BROKEN TO SLIGHTLY BROKEN.		-S-24 25/0.0', 0% recovery. -core barrel very tight going in casing may be skewed.
		S-25	65/0.5	0.5		-VERY SOFT, LIGHT GRAY CLAY LENS 47.9' TO 48.0'.		-lost circulation @ 48.0' due to clay in bit. -switch to SPT spoon 48.0' to 48.5. -return to NQ core 48.5.
	50	R-2	68% (9%)	2.7		-BECOMES PROGRESSIVELY SOFTER TOWARD BOTTOM OF RUN R-2. — 53.5'		
	55					BOTTOM OF BORING @ 53.5' 4/22/99 15:00		
	60							
	65							

DATE BEGAN: 4/22/99
DATE COMPLETED: 4/22/99
FIELD GEOLOGIST: RMW
CHECKED BY: RMW/JAA
DRILLING CO.: CAROLINA DRILLING

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY SOIL NQ CORE FOR ROCK
DRILLER: GERALD HELPER: BRETT

NOTES:
RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-202, STA. 233+75 (UNIT 3&4 SLURRY POND)

COORDINATES: N ______ E ______

SURFACE EL: 38.2'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	7-8 9-10 (17)	1.6		SANDY CLAY, MOTTLED ORANGE AND BROWN, STIFF. (FILL) 2.0'	cl	begin @ 17:30 4/22/99 PP= 1.0 TSF
		S-2	10-13 7-8 (20)	1.3		CLAYEY SAND, MOTTLED GRAY AND ORANGE, MEDIUM DENSE. (FILL) 4.0'	sc	
	5	S-3	2-2 4-3 (6)	1.0		SANDY CLAY, MOTTLED ORANGE AND GRAY, MEDIUM TO VERY STIFF. (FILL)	cl	PP=1.0 TSF
		S-4	1-1 3-4 (4)	1.3				PP= 1.5 TSF
		S-5	2-3 7-10 (10)	1.2				PP=2.0 TSF
	10	S-6	8-13 15-10 (28)	1.5		11.0'		PP= 2.25 TSF
						CLAYEY SAND, MOTTLED ORNG / GRAY MEDIUM DENSE.(FILL)	sc	Stop for the day @ 19:00 14.0' 4/22/99 -Resume 4/23/99
		S-7	7-7 12-14 (19)	1.2		SANDY CLAY, MOTTLED RED,GRAY, AND ORANGE, VERY STIFF HARD. (FILL)	cl	PP= 4.5 TSF
	15	S-8	7-12 12-14 (24)	1.1				PP> 4.5 TSF
		S-9	1-2 7-9 (9)	2.0				PP= 3.0 TSF
		S-10	2-6 10-13 (16)	1.6		- WOOD CHIPS @ 19.5' 20.0'		PP= 2.5 TSF
	20	S-11	5-8 9-11 (17)	1.6		SANDY CLAY, MOTTLED RED,GRAY / ORANGE, STIFF TO VERY STIFF. BLACK ZONE W/ WOOD CHIPS @ 20.4'-20.6'.	cl	PP= 2.0 TSF PP= 1.0 TSF
		S-12	4-4 6-6 (10)	1.7				PP= 3.5 TSF
	25	S-13	4-5 8-9 (13)	1.2		26.3'		
		S-14	5-11 12-14 (23)	1.6		SITLY SAND, GRAY, MEDIUM DENSE. -SAND TURNS ORANGE 27.6' TO 29.5' -0.5" SOFT GRAY CLAY SEAM @ 29.5'	sm	
		S-15	5-5 7-9 (12)	1.5		29.5'		
	30	S-16	4-4 10-13 (14)	1.1		SAND, GRAY, MEDIUM DENSE TO DENSE.	sp	
		S-17	14-17 15-10 (32)	0.8		34.0'		
		S-18	2-2-3-5	2.0		SILTY SAND, GRAY, LOOSE, SHELL FRAGS.	sm	

DATE BEGAN: 4/22/99
DATE COMPLETED: 4/23/99
FIELD GEOLOGIST: J.Lilly
CHECKED BY: RMW/JAA
DRILLING CO.: CAROLINA DRILLING

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY IN SOIL
NQ CORE IN ROCK
DRILLER: GERALD HELPER: BRETT

NOTES:

RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-202, STA. 233+75 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 38.2'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(5)	2.0				
		S-19	6-7 5-6 (12)	1.7				
		S-20	1-2 3-5 (5)	0.0		-Sand and shells in circulation fluid.		
	40	S-21	5-5 5-6 (10)	2.0		-Brown sand and gravel and shells 42.0' TO 42.5'		
		S-22	4-4 5-25 (9)	2.0		-Becomes clayey w/ broken limestone 44.0 to 44.6'.		-TOP 1.3' OF R-1: 6" PIECE, 3" PIECE, AND 2" PIECE, REST 1" PIECES BELOW MUCH MORE BROKEN.
		S-23	8-50/0.4	0.6		44.9'		
	45	R-1	42% (10%)	2.1		LIMESTONE, LIGHT GRAY AND WHITE, SHELLS, MODERATELY HARD, VERY CLOSELY TO CLOSELY SPACED FRACTURES, VERY BROKEN TO SLIGHTLY BROKEN.		
	50	R-2	0% (0%)	0.0		54.3'		No recovery in R:2, but clay in core bit teeth. Increased circulation pressure suggests depth of clay.
	55					SILTY CLAY, DARK GRAY, SOFT. (BLACK MINGO CLAY) 54.9'	cl	- Drill rods accidently dropped after core barrel removed; when retrieved, a 6" piece of clay from bottom of hole was attached. Clay saved in jar in core box.
						BOTTOM OF BORING @ 54.9'		
	60							
	65							

DATE BEGAN: 4/22/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 4/23/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: RMW	DRILLING METHOD: MUD ROTARY SOIL	
CHECKED BY: RMW/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — **Project No. 991988.01**

Log of Boring : B-203, STA. 243+00 (UNIT 3&4 SLURRY POND)

COORDINATES
N ________ E ________

SURFACE EL: 39.9'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)
		S-1	2-2 3-4 (5)	1.2
		S-2	5-4 5-7 (9)	0.4
	5	S-3	4-5 8-13 (13)	1.7
		S-4	1-1 4-8 (5)	1.7
		S-5	1-1 6-7 (7)	1.7
	10	S-6	7-6 7-10 (13)	1.2
		S-7	5-5 7-8 (12)	1.5
	15	S-8	15-12 13-10 (25)	1.3
		S-9	5-3 3-5 (6)	1.2
		S-10	3-3 5-8 (8)	1.3
	20	S-11	5-8 12-14 (20)	1.2
		S-12	8-10 12-10 (22)	1.8
	25	S-13	9-12 16-19 (28)	1.9
		S-14	13-16 16-13 (32)	1.6
	30	S-15	2-3 5-10 (8)	1.1
		S-16	7-7 9-9 (16)	1.9
		S-17	6-10 15-14 (25)	1.6
		S-18	8-12-18-10	1.0

DESCRIPTION	USCS SYMBOL	REMARKS
SANDY CLAY, VARIABLE LIGHT BROWN, MOTTLED ORANGE / BROWN GRADING TO MOTTLED GRAY / BROWN, VERY SOFT TO VERY STIFF. (FILL) -very soft 6.0' to 7.5'. -very soft 8.5' to 9.5'. 14.5'	cl	Drill rig breaks down after S-3 (9:00-9:40). Soft material may be sluff since hole has been sitting. Since soft material showed up again, likely not sluff. Driller comments he is loosing water. Could be in this layer. PP=2.5 TSF
CLAYEY SAND, MOTTLED RED-BROWN, LOOSE TO MEDIUM DENSE. 17.0'	sc	
SANDY CLAY, MOTTLED GRAY/ BROWN, MEDIUM STIFF. (FILL). 20.0'	cl	
SAND, GRAY, MEDIUM DENSE, TRACE CLAY. (FILL) 22.0'	sp	
SANDY CLAY, MOTTLED GRAY/ BROWN, VERY STIFF. (FILL). 24.0'	cl	
SAND, BLACK, MEDIUM DENSE TO DENSE, TRACE CLAY. (FILL) 27.0'	sp	
CLAYEY SAND, LIGHT BROWN, MEDIUM DENSE. (FILL) 28.0'	sc	
SANDY CLAY, DARK GRAY, MEDIUM STIFF. 30.0'	cl	
CLAYEY SAND, LIGHT BROWN, MEDIUM DENSE. 33.0'	sc	
SAND, LIGHT GRAY, MEDIUM DENSE TO DENSE, COARSE-GRAINED.	sp	Bag sample taken in addition to S-17.

DATE BEGAN: 4/30/99 DATE COMPLETED: 5/3/99 FIELD GEOLOGIST: J.Lilly CHECKED BY: RMW/JAA	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked. GWL: DEPTH: DATE/TIME: DRILLING METHOD: MUD ROTARY IN SOIL NQ CORE IN ROCK	NOTES:
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER- WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-203, STA. 243+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 39.9'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(30)					
		S-19	5-9 7-9 (16)	1.2		SAND, GRAY,MEDIUM DENSE, COARSE-GRAINED, WITH INTERBEDDED SHELL FRAGMENTS, TRACE CLAY.		
		S-20	11-5 10-8 (15)	2.0				
	40	S-21	6-5 6-4 (11)	1.6			sp	
		S-22	4-10 10-46 (20)	1.3				
		S-23	4-50/6"	1.0		TOP OF ROCK 45.0'		
	45	R-1	46% (20%)	2.3		LIMESTONE, BROKEN. -BOTTOM OF SAMPLE MORE POROUS THAN TOP.		Casing sit & work stop @ 45' (4/30/99). 10:00 (5/22/99) R-1 started. Driller comments rock is hard. 46.5' soft, 47.0' very soft.
	50	R-2	18% (0%)	0.9		55.0'		
	55	S-24	7-5 5-6 (10)	2.0		CLAY, BLACK, VERY STIFF. (BLACK MINGO CLAY). 57.0'		PP=2.5-3.5 TSF
						BOTTOM OF BORING @ 57.0'		
	60							
	30							

DATE BEGAN: 4/30/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/3/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL. NQ CORE IN ROCK	
CHECKED BY: RMW/JAA		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-204, STA. 257+00 (UNIT 3&4 SLURRY POND)

COORDINATES: N ______ E ______
SURFACE EL: 38.6'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					UNSAMPLED.		-UNSAMPLED PORTION OF LOG ABBREVIATED.
	45					TOP OF ROCK 46.5'		
	50	R-1	77% (52%)	3.8		LIMESTONE, GRAY, INTERBEDED SHELL, BOTTOM 1.5' UNBROKEN.		
		R-2	53% (19%)	0.6' rock + 2.0' clay		54.5'		
	55					CLAY, BLACK, (BLACK MINGO CLAY)		
		S-1	5-10 19-12 (29)			58.5'	cl	
	60					BOTTOM OF BORING @ 58.5'		
	65							
	70							

DATE BEGAN: 5/2/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/2/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL. NQ CORE IN ROCK		
CHECKED BY: RMW/JAA			
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART	HELPER: RON STEWART	RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-205, STA. 265+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 39.0'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	4-8 9-4 (17)	1.3		SAND, LIGHT TO DARK BROWN, MEDIUM DENSE.(FILL) 2.0'	sp	
		S-2	12-17 19-13 (36)	1.3		CLAYEY SAND, MOTTLED GREY BROWN, DENSE.(FILL) 4.0'	sc	
	5	S-3	13-20 30-39 (50)	1.0		SAND, DARK BROWN, OCCASIONAL WOOD FRAGMENTS, DENSE.(FILL)	sp	
		S-4	3-11 21-30 (32)	1.2				
		S-5	4-15 26-31 (41)	1.3				
	10	S-6	9-21 27-35 (48)	1.1				
		S-7	14-21 31-36 (52)	1.1				
	15	S-8	15-21 25-21 (46)	0.9				
		S-9	6-13 17-19 (30)	1.4		-LT. BROWN CLAY LENSE @ 17.5' 17.5'		
		S-10	2-14 20-17 (34)	1.2		SAND, LIGHT BROWN, DENSE.(FILL) 20.0'	sp	
	20	S-11	4-8 6-7 (14)	1.5		SANDY CLAY, MOTTLED GRAY, LIGHT BROWN TO RED, MEDIUM DENSE, OCCASIONAL WOOD CHIPS.		
		S-12	3-5 6-6 (11)	1.5				- Bag sample taken
	25	S-13	2-2 3-4 (5)	2.0		26.0'		
		S-14	5-9 13-19 (22)	2.0		SILTY SAND, LIGHT BROWN, MEDIUM DENSE 27.0'	sm	
						SAND, LIGHT GRAY, MEDIUM DENSE, FINE GRAINED.	sp	
		S-15	1-1 7-12 (8)	1.4				
	30	S-16	14-20 21-24 (41)	1.4				
		S-17	9-7 10-10 (17)	1.3				
		S-18	4-4-1-2	1.2				

DATE BEGAN: 5/3/99
DATE COMPLETED: 5/3/99
FIELD GEOLOGIST: J.Lilly
CHECKED BY: RMW,JAA

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY IN SOIL
NQ CORE IN ROCK

NOTES:

DRILLING CO.: CAROLINA DRILLING
DRILLER: GERALD HELPER: BRETT
RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-205, STA. 265+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 39.0'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(5)			35.5'		
						CLAY, GREEN, SOFT. 36.5'	ch	
		S-19	1-1 1-1 (2)	1.7		SAND, GRAY, VERY LOOSE TO MEDIUM DENSE, COARSE-GRAINED, INTERBEDDED SHELL FRAGMENTS.		
		S-20	11-8 6-5 (14)	2.0				
	40	S-21	2-4 3-6 (7)	2.0			sp	
		S-22	6-6 7-10 (13)	1.6		-INTERBEDDED ROCK AND SHELL FRAGMENTS 43.5' TO 49.5'		
	45	S-23	13-9 11-9 (20)	1.5				
		S-24	10-9 12-24 (21)	1.3				
		S-25	8-18-52/6"	0.8		TOP OF ROCK 49.5'		
	50	R-1	17% (0%)	0.9		LIMESTONE, GRAY, INTERBEDDED SHELL FRAGMENTS, VERY BROKEN.		
	55	R-2	27% (0%)	0.9 rock + 0.5 clay		59.0'		
						CLAY, BLACK, (BLACK MINGO CLAY) 59.5'	cl	
	60					BOTTOM OF BORING @ 59.5'		
	65							

DATE BEGAN: 5/3/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/3/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW,JAA	NQ CORE IN ROCK		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-206, STA. 204+85 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 38.9'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	5-9 9-9 (18)	1.3		CLAYEY SAND, LIGHT BROWN TO GRAY, MEDIUM DENSE TO DENSE. (FILL) -MANY WOOD CHIPS @ 2.0'.	sc	
		S-2	10-16 20-17 (36)	1.1		4.0'		
	5	S-3	4-5 7-14 (12)	1.3		SANDY CLAY, MOTTLED GRAY TO BROWN, STIFF, TRACE SHELL FRAGMENTS. (FILL) 6.0'	cl	
		S-4	4-6 11-11 (17)	1.6		SAND, LT. GRAY, MEDIUM DENSE, (FILL)	sp	
		S-5	4-6 7-5 (13)	1.5		-GREENISH GRAY CLAY LENS @ 9.5' 10.5'		
	10	S-6	2-3 2-2 (5)	1.5		CLAY, GRAY TO LIGHT BROWN TO RED, MEDIUM STIFF TO STIFF, TRACE SHELL, OCCASIONAL SAND LENS. (FILL)	cl	
		S-7	2-5 6-12 (11)	2.0		15.0'		
	15	S-8	5-6 6-8 (12)	1.5		CLAYEY SAND, DARK BROWN TO GRAY, MEDIUM DENSE. (FILL)	sc	
		S-9	2-7 12-14 (19)	1.2		-TRACE WOOD CHIPS IN S-9.		
		S-10	2-2 7-10 (9)	0.8		20.0'		
	20	S-11	2-3 4-5 (7)	1.5		SANDY CLAY, MOTTLED GRAY / LT BROWN, MEDIUM STIFF. 23.0'	cl	
		S-12	2-3 14-19 (17)	1.7		CLAYEY SAND, LIGHT GRAY, MEDIUM DENSE. 25.0'	sc	
	25	S-13	11-15 17-20 (32)	1.0		SAND, LIGHT BROWN TO GRAY, MEDIUM DENSE TO MED. DENSE.	sp	
		S-14	5-9 15-20 (24)	1.2				Bag sample taken.
		S-15	2-7 15-20 (22)	1.2		30.0'		
	30	S-16	18-15 17-19 (32)	1.3		SAND, GRAY, LOOSE TO DENSE INTERBEDDED SHELL.		
		S-17	4-5 4-13 (9)	1.3				
		S-18	12-4-3-4	1.2			SM	

DATE BEGAN: 5/4/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/4/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL	
CHECKED BY: RMW/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — **Project No. 991988.01**

Log of Boring : B-206, STA. 204+85 (UNIT 3&4 SLURRY POND)

COORDINATES: N ________ E ________

SURFACE EL: 38.9'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(7)					
		S-19	1-2 2-2 (4)	2.0		37.5'		
		S-20	3-2 4-5 (6)	1.8		CLAYEY SAND, DARK GRAY,LOOSE TO MEDIUM DENSE, INTERBEDDED SHELL FRAGMENTS.	sc	
	40	S-21	7-7 6-5 (13)	1.6				-LOOSE CIRCULATION OF WATER.
		S-22	5-2 3-8 (5)	1.9				
		S-23	20-28 30/1"	0.7		TOP OF ROCK 45.0'		
	45	R-1	43% (7%)	2.2		LIMESTONE, GRAY, INTERBEDDED SHELL.		
	50	R-2	0% (0%)	0.0		55.0'		Driller says that R-2 was easy drilling the entire way.
	55	S-24	13-5 7-4 (12)	2.0		CLAY, BLACK, STIFF. (BLACK MINGO CLAY). 57.0'	cl	
						BOTTOM OF BORING @ 57.0'		
	60							
	65							

DATE BEGAN: 5/4/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/4/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW,JAA	NQ CORE IN ROCK		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-207, STA. 209+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______
SURFACE EL: 39.0'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED		* Unsampled portion of log abbeviated.
	40					TOP OF ROCK 44.0'		
	45	R-1	33% (22%)	1.7		LIMESTONE, GRAY, BROKEN TO VERY BROKEN, INTERBEDDED SHELL.		
	50	R-2	43% (0%)	0.5 rock + 1.7 clay		52.3'		
						CLAY, BLACK. (BLACK MINGO CLAY) 54.0'	cl	
						BOTTOM OF BORING @ 54.0'		
	55							
	60							
	65							

DATE BEGAN: 5/4/99
DATE COMPLETED: 5/4/99
FIELD GEOLOGIST: J.Lilly
CHECKED BY: RMW,JAA
DRILLING CO.: MID ATLANTIC

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY IN SOIL
NQ CORE IN ROCK
DRILLER: JEFF STEWART HELPER: RON STEWART

NOTES:

RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-208, STA. 218+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.8'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED		* Unsampled portion of log abbreviated.
	40					TOP OF ROCK 44.0'		
	45	R-1	60% (23%)	1.9 rock + 1.1 clay		LIMESTONE, GRAY, INTERBEDDED SHELL. 47.9'		
						CLAY, (BLACK MINGO CLAY). 49.0'	cl	
	50					BOTTOM OF BORING @ 49.0'		
	55							
	60							
	65							

DATE BEGAN: 5/4/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/4/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW/JAA	NQ CORE IN ROCK		
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART		RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-209, STA. 223+50 (UNIT 3&4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.8'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	5-6 6-8 (12)	1.2	SAND, MOTTLED LT BROWN / GRAY / ORANGE, MEDIUM DENSE. (FIILL)	sp	
		S-2	11-7 9-12 (16)	0.9	4.0'		
	5	S-3	3-9 10-13 (19)	1.5	SANDY CLAY, MOTTLED GRAY / BROWN / BLACK, SOFT TO VERY STIFF. (FILL)	cl	
		S-4	1-1 3-5 (4)	1.2			
		S-5	2-3 9-11 (12)	1.9			
	10	S-6	6-5 6-7 (11)	1.5			
		S-7	6-10 12-15 (22)	1.5	-TRACE WOOD CHIPS.		
	15	S-8	3-5 7-8 (12)	1.6			
		S-9	1-5 10-11 (15)	1.9	-SAMPLE HAS VERY LOOSE, SANDY CLAY.		Driller suggests loose material is due to side discharge bit.
	20	S-10	2-3 6-7 (9)	1.9			
		S-11	5-7 10-12 (17)	1.7	22.0'		PP=2.5TSF
		S-12	WOH-5 7-11 (12)	1.8	SANDY CLAY, MOTTLED GRAY / BROWN, STIFF. 24.0'	cl	
	25	S-13	8-8 7-7 (15)	1.5	SAND, MOTTLED GRAY TO BROWN, MEDIUM DENSE.	sp	
		S-14	4-9 15-25 (24)	1.7			
	30	S-15	8-6 8-14 (14)	1.1			
		S-16	8-12 17-14 (29)	1.1	31.5'		
		S-17	2-4 10-15 (14)	1.3	SAND, GRAY TO BLACK, MEDIUM DENSE.	sp	
		S-18	10-6-7-4				

DATE BEGAN: 5/4/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/5/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL	
CHECKED BY: RMW/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION | Project No. 991988.01

Log of Boring : B-209, STA. 223+50 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______
SURFACE EL: 38.8'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(15)					
		S-19	2-2 2-3 (4)	1.2		-Dark green clay lens (3") @37.0'		
		S-20	1-2 2-2 (4)	2.0		-Dark gray clay lens, trace wood chips @ 39.5'		
	40					40.5'		
		S-21	2-3 2-1 (5)	1.8		SAND, LIGHT GRAY,COARSE-GRAINED, INTERBEDDED SHELL FRAGMENTS.	sp	
		S-22	4-17 17-18 (34)	2.0		Top of Rock 43.5'		CASING AND CORE RUN STARTS AT 44.0'
	45	R-1	22% (12%)	0.8 rock + 0 3 clay		LIMESTONE, GRAY, INTERBEDDED SHELLS, VERY BROKEN TO SLIGHTLY BROKEN. 47.5"		ROCK RECOVERED WAS 0.4' SAND SIZED PARTICLES AND 0.4' LIMESTONE CORE.
						SANDY CLAY, BLACK, (BLACK MINGO CLAY). 49.0'	sc	
						BOTTOM OF BORING @ 49.0'		
	50							
	55							
	60							
	65							

DATE BEGAN: 5/4/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/5/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL	
CHECKED BY: RMW/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — **Project No. 991988.01**

Log of Boring : B-210, STA. 260+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES: N ________ E ________

SURFACE EL: 38.8'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED.		-UNSAMPLED PORTION ABBREVIATED.
	40							
						TOP OF ROCK 44.0'		
	45					BOTTOM OF BORING @ 44.0'		
	50							
	55							
	60							
	65							

DATE BEGAN: 5/6/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/6/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL TO THE TOP OF ROCK.		
CHECKED BY: RMW/JAA			
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART		RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-211, STA. 270+00 (UNIT 3&4 SLURRY POND)

COORDINATES: N ________ E ________

SURFACE EL: 39.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED.		-UNSAMPLED PORTION ABBREVIATED.
	40							
						TOP OF ROCK 44.5'		
	45	R-1	65% (15%)	3.3		LIMESTONE, GRAY, VERY BROKEN, INTERBEDDED SHELL.		
	50	R-2	70% (25%)	1.5 rock + 2.0		53.5'		ALL OF CLAY BAGGED & INCLUDED IN CORE BOX.
	55					SANDY CLAY, BLACK, (BLACK MINGO CLAY). 55.5'	SC	
						BOTTOM OF BORING @ 55.5'		
	60							
	65							

DATE BEGAN: 5/5/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/5/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW/JAA	NQ CORE IN ROCK		
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART		RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-212, STA. 213+50 (UNIT 3&4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.6'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	5-5 7-9 (12)	1.5		SAND, DARK BROWN TO RED BROWN TO LT. BROWN, MED. DENSE. (FILL)	sp	
		S-2	9-9 10-9 (19)	0.8		-Gray sand lens @ 3.5' 4.5'		
	5	S-3	5-6 5-10 (11)	1.2		SANDY CLAY, MOTTLED GRAY TO BROWN, MEDIUM STIFF TO VERY STIFF. (FILL) -Trace wood chips@ 6.0'		
		S-4	1-4 5-8 (9)	1.3		-Trace wood chips @ 9.0'		
		S-5	1-3 5-9 (8)	1.5		-Wood chips (2") @ 11'		
	10	S-6	5-9 12-14 (21)	1.6		-Black sand lens (5") @ 13.5'		
		S-7	5-6 8-9 (14)	1.6		-Wood chips @ 14.0'		
	15	S-8	3-6 9-11 (15)	1.6		-Trace wood chips @ 16.5' -VERY SOFT LIGHT BROWN SANDY CLAY WITH INTERBEDDED SHELL AND ROCK 16.5' TO 17.0'.		-Soft layer between clay above and below in spoon
		S-9	2-2 7-10 (9)	1.9				
		S-10	1-1 6-7 (7)	2			SC	Entire sample (S-10) saved. Bottom 6" in jar, plus 3 bagies.
	20	S-11	3-4 6-6 (10)	2				
		S-12	3-4 6-7 (10)	2		-Dark brown sand lens @ 23' 23.0'		
	25	S-13	7-9 8-8 (17)	1.5		SANDY CLAY, MOTTLED GRAY / BROWN, MEDIUM DENSE, WOOD CHIPS,	sc	
		S-14	6-8 10-5 (18)	1.7		-WOOD CHIPS @ 27.0'. 27.5'		
		S-15	3-5 10-16 (15)	1.5		SAND, LIGHT GRAY, MEDIUM DENSE, COARSE-GRAINED. 29.5'	sp	
	30	S-16	7-8 9-12 (17)	0.7		CLAYEY SAND, GRAY AND BROWN, MEDIUM DENSE, - WOOD CHIPS @ 31.0'.	sc	
		S-17	2-6 8-8 (14)	1.1		33.5'		
		S-18	1-3-3-7	1.7		SAND, DARK GRAY, LOOSE, INTERBEDDED SHELL. -2.5" Dark gray clay lens @ 34.5'	sp	

DATE BEGAN: 5/5/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/6/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL	
CHECKED BY: RWM/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-212, STA. 213+50 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 38.6'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	DESCRIPTION	USCS SYMBOL	REMARKS
		S-18	(6)		-Interbedded shell fragments @ 35.5'		
		S-19	2-1 4-5 (5)	1.6	-Trace shell fragments @ 37.0'		
	40	S-20	2-2 4-4 (8)	1.4	40.5'		
		S-21	4-3 4-4 (7)	1.7	CLAYEY SAND, DARK GRAY, LOOSE, INTERBEDDED SHELL FRAGMENTS. Top of Rock 42.8'	SC	
		S-22	4-50/1"	0.7	LIMESTONE, GRAY, INTERBEDDED SHELL FRAGMENTS, BROKEN TO VERY BROKEN.		BOTTOM .8' OF SAMPLE VERY BROKEN & VERY POROUS. TOP 0.4' BROKEN & MORE CEMENTED.
	45	R-1	24% (0%)	1.2	47.5'		
					SANDY CLAY, DARK GRAY, VERY STIFF,OCCASIONAL ROCK FRAGS, DRY TO VERY DRY.	SC	
	50	S-23	10-29 98-82 (127)	2.0'			
		S-24	18-52	1.0'	-rock fragment.		
		S-25	27-53/6"	1.6			
		S-26	25-33-50/4	1.6	-1" rock lens @ 56.0'.		
	55	S-27	28-37 50/4"	1.6			
		S-28	28-39 29-50/3"	1.6	58.5'		
	60				BOTTOM OF BORING @ 58.5'		
	65						

DATE BEGAN: 5/5/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/6/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RWM/JAA	NQ CORE IN ROCK		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-213, STA. 281+50 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 40.5'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED.		-UNSAMPLED PORTION ABBREVIATED.
	40							
	45					TOP OF ROCK 48.0'		
						BOTTOM OF BORING @ 48.0'		
	50							
	55							
	60							
	65							

DATE BEGAN: 5/5/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/5/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL TO THE TOP OF ROCK.		
CHECKED BY: RWM/JAA			
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART		RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-214, STA. 279+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 39.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	DESCRIPTION	USCS SYMBOL	REMARKS
					SAND, LIGHT GRAY, MED. DENSE. (FILL) 0.5'	sp	
		S-1	8-10 15-23 (25)	1.1	CLAYEY SAND, MOTTLED GRY / BRN / ORNG, MED. DENSE. (FIL 2.5'	sc	
		S-2	13-16 16-24 (32)	1.1	SAND, DARK BROWN, DENSE TO VERY DENSE, TRACE CLAY. (FILL)	sp	ALTHOUGH HIGH BLOW COUNTS, SAMPLE DOESN'T SEAM OVER-COMPACTED IN SPOON. -SAMPLE DRY, YET USING MUD ROTARY METHOD.
	5	S-3	17-24 40-52/5" (64)	1.2	-wood chips@ 5.5'		
		S-4	7-24 40-52/5 (64)	1.3	-wood chips@ 7.0'		
		S-5	5-17 29-35 (46)	1.5			
	10	S-6	15-21 33-39 (54)	1.1	-LIGHT BROWN 12.0' TO 13.0'.		
		S-7	5-17 25-25 (42)	1.1	13.0' SAND, DARK BROWN, MEDIUM DENSE TO DENSE.	sp	
	15	S-8	5-7 12-14 (19)	1	-SAMPLE VERY DRY.		
		S-9	4-12 18-19 (30)	1.2			
		S-10	3-14 19-21 (33)	1			
	20	S-11	13-12 10-9 (22)	1.4			
		S-12	5-6 7-5 (13)	1.5			
	25	S-13	5-9 4-4 (13)	1.2	-SAMPLE WET, COLOR LIGHTENS TO LIGHT BROWN. 27.0'		
		S-14	1/12" 1-2 (1)	1.5	SANDY CLAY, DARK GRAY, VERY SOFT, INTERBEDDED SHELL.	cl	
		S-15	1-1 1-1 (2)	1.3			
	30	S-16	1-1 1-1 (2)	2	-1" GRAY CLAY LENS @ 30.5'		
		S-17	WOH/18 3 (0)	2			
		S-18	3-1-1-2	2.0'			

DATE BEGAN: 5/6/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/6/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW/JAA	NQ CORE IN ROCK		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-214, STA. 279+00 (UNIT 3&4 SLURRY POND)

COORDINATES N ____ E ____
SURFACE EL: 39.1'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	8-10 15-23 (25)	1.1		SAND, LIGHT GRAY, MED. DENSE. (FILL) 0.5' CLAYEY SAND, MOTTLED GRY / BRN / ORNG, MED. DENSE. (FIL 2.5'	sp sc	
		S-2	13-16 16-24 (32)	1.1		SAND, DARK BROWN, DENSE TO VERY DENSE, TRACE CLAY. (FILL)		ALTHOUGH HIGH BLOW COUNTS, SAMPLE DOESN'T SEAM OVER-COMPACTED IN SPOON. -SAMPLE DRY, YET USING MUD ROTARY METHOD.
	5	S-3	17-24 40-52/5" (64)	1.2		-wood chips@ 5.5'		
		S-4	7-24 40-52/5 (64)	1.3		-wood chips@ 7.0'	sp	
		S-5	5-17 29-35 (46)	1.5				
	10	S-6	15-21 33-39 (54)	1.1		-LIGHT BROWN 12.0' TO 13.0'.		
		S-7	5-17 25-25 (42)	1.1		13.0' SAND, DARK BROWN, MEDIUM DENSE TO DENSE.		
	15	S-8	5-7 12-14 (19)	1		-SAMPLE VERY DRY.		
		S-9	4-12 18-19 (30)	1.2				
		S-10	3-14 19-21 (33)	1			sp	
	20	S-11	13-12 10-9 (22)	1.4				
		S-12	5-6 7-5 (13)	1.5				
	25	S-13	5-9 4-4 (13)	1.2		-SAMPLE WET, COLOR LIGHTENS TO LIGHT BROWN.		
		S-14	1/12" 1-2 (1)	1.5		27.0' SANDY CLAY, DARK GRAY, VERY SOFT, INTERBEDDED SHELL.		
		S-15	1-1 1-1 (2)	1.3				
	30	S-16	1-1 1-1 (2)	2		-1" GRAY CLAY LENS @ 30.5'	cl	
		S-17	WOH/18 3 (0)	2				
		S-18	3-1-1-2	2.0'				

DATE BEGAN: 5/6/99
DATE COMPLETED: 5/6/99
FIELD GEOLOGIST: J.Lilly
CHECKED BY: RMW/JAA
DRILLING CO.: CAROLINA DRILLING

GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.
GWL: DEPTH: DATE/TIME:
DRILLING METHOD: MUD ROTARY IN SOIL
NQ CORE IN ROCK
DRILLER: GERALD HELPER: BRETT

NOTES:

RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-215, STA. 211+00 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 38.2'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED.		-UNSAMPLED PORTION ABBREVIATED.
	40							
	45							
						TOP OF ROCK 49.0'		
	50					BOTTOM OF BORING @ 49.0'		
	55							
	60							
	65							

DATE BEGAN: 5/6/99 DATE COMPLETED: 5/6/99 FIELD GEOLOGIST: J.Lilly CHECKED BY: RMW/JAA	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked. GWL: DEPTH: DATE/TIME: DRILLING METHOD: MUD ROTARY IN SOIL TO THE TOP OF ROCK.	NOTES:	
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART	RIG: CME-45	

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-216, STA. 246+00 (UNIT 2 SLURRY POND)

COORDINATES: N ________ E ________

SURFACE EL: 39.6'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					UNSAMPLED		-Unsampled portion abbreviated.
	40							
	45					TOP OF ROCK 46.0'		
		R-1	72% (20%)	3.6		LIMESTONE, GRAY, INTERBEDDED SHELL, VERY BROKEN TO UNBROKEN.		R-1: TOP 1.8' BROKEN TO VERY BROKEN AND NOT POROUS. BOTTOM 1.8' UNBROKEN (0.7' & 0.5') TO BROKEN, AND POROUS.
	50							
		R-2	20% (0%)	1		54.5'		R2: VERY BROKEN; TOP 0.5' NOT POROUS; BOTTOM 0.5' POROUS.
	55					SANDY CLAY, BLACK, DRY (BLACK MINGO CLAY). 56.0'	SC	ALL CLAY SAVED IN BAGGIES.
						BOTTOM OF BORING @ 56.0'		
	60							
	65							

DATE BEGAN: 5/6/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/6/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW/JAA	NQ CORE IN ROCK		
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT		RIG: CME-55

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-218, STA. 228+50 (UNIT 2 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 38.7'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED		-Unsampled portion abbreviated.
	46.0					TOP OF ROCK 46.0'		
	46.0–52.0	R-1	34% (7%)	1.7		LIMESTONE, GRAY, INTERBEDDED SHELL, VERY BROKEN TO BROKEN. 52.0'		-R-1 : Top 3" very broken shell and rock; next 2" interbedded shell, porous; next 10" broken to very broken, yet more cemented texture. Remaining 5" very broken w/ interbedded shell.
	52.0–56.0	R-2	15% (58%)	0.7 rock + 3.3		SANDY CLAY, BLACK, (BLACK MINGO CLAY). 56.0'		-R2: Unbroken, highly cemented.
						BOTTOM OF BORING @ 56.0'		

DATE BEGAN: 5/6/99	GWL: DEPTH: DATE/TIME:	Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/7/99	GWL: DEPTH: DATE/TIME:		
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL		
CHECKED BY: RMW/JAA	NQ CORE IN ROCK		
DRILLING CO.:MID ATLANTIC	DRILLER: JEFF STEWART HELPER: RON STEWART		RIG: CME-45

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : B-220, STA. 200+50 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ______ E ______

SURFACE EL: 39.7'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	PROFILE	DESCRIPTION	USCS SYMBOL	REMARKS
	1					-UNSAMPLED		Unsampled portion abbreviated.
	40							
	45					TOP OF ROCK 48.0'		
	48.0–52.5	R-1	57% (8%)	2.8		LIMESTONE, GRAY, INTERBEDDED SHELL, BROKEN TO VERY BROKEN.		PULLING BARREL OUT RODS DROP. -R-1: TOP 1' BROKEN (1"-4" PIECES) WITH SOLID TEXTURE. NEXT 0.7' FRAGMENTED TO 1" PIECES WITH PORUS TEXTURE. NEXT 1.2' BROKEN (1"-5" PIECES) WITH SLIGHTLY POROUS TEXTURE.
	50							
	52.5–55.5	R-2	0% (0%)	0		55.5'		
	55							
	55.5–57.5	S-1	10-10 18-34 (28)	1.8		SANDY CLAY, BLACK (BLACK MINGO CLAY). 57.5'		
						BOTTOM OF BORING @ 57.5'		
	60							
	65							

DATE BEGAN: 5/17/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/17/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.Lilly	DRILLING METHOD: MUD ROTARY IN SOIL TO THE TOP OF ROCK.	
CHECKED BY: RMW/JAA		
DRILLING CO.: MID ATLANTIC	DRILLER: JEFF HELPER: RON	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

SANTEE COOPER-WINYAH GENERATING STATION — Project No. 991988.01

Log of Boring : PZ-201, STA. 240+62 (UNIT 3 & 4 SLURRY POND)

COORDINATES N ________ E ________

SURFACE EL: 7.7'

ELEVATION (FEET MSL)	DEPTH (FEET)	SAMPLE NO. OR RUN NO.	BLOW/6" & (N) OR % REC. & (RQD)	RECOVERY (ft)	DESCRIPTION	USCS SYMBOL	REMARKS
		S-1	4-4 5-6 (9)	1.3	SAND, MOTTLED BROWN TO BLACK, LOOSE TO MEDIUM DENSE, TRACE CLAY.	sp	
		S-2	7-10 9-13 (19)	0.8	4.5'		
	5	S-3	7-5 10-14 (15)	1.5	SAND, LIGHT GRAY, MEDIUM DENSE, TRACE CLAY.	sp	
		S-4	5-5 8-11 (13)	1.5	9.0'		
		S-5	1/24"	1.0	CLAYEY SAND, DARK GRAY,VERY LOOSE 11.0'	sc	
	10	S-6	3-2 2-2 (4)	1.3	CLAYEY SAND, DARK GRAY, VERY LOOSE TO LOOSE, INTERBEDDED SHELL.	sc	
		S-7	2-4 5-7 (9)		TOP OF ROCK 14.0'		
		S-8	25/0"		LIMESTONE, GRAY, INTERBEDDED SHELL, VERY BROKEN TO BROKEN.		-S-8: HOLE SLUFF SAVED IN JAR.
	15	R-1	32% (0%)	1.6			-R-1: TOP 0 .5' BROKEN TO VERY BROKEN, WITH NON-POROUS TEXTURE. NEXT 0.4' VERY BROKEN TO FRAGMENTED. NEXT 0.2' BROKEN WITH POROUS/ SANDY TEXTURE.
	20	R-2	12% (0%)	.4 rock +0.2 clay	24.0'		-R-2: NEXT 0.4' VERY BROKEN TO BROKEN, INTERBEDDED SHELL. SAMPLE HAS PORUS TEXTURE.
	25				SANDY CLAY, BLACK, MEDIUM DENSE, DRY (BLACK MINGO CLAY).		CUTTINGS FROM CORE RUN R-2 SAVED. VERY SANDY.
	30	S-9	6-8 9-13 (17)	1.3		cl	
		S-10	10-11 15-17 (26)	0.7	END OF BORING @ 35'		

DATE BEGAN: 5/13/99	GWL: DEPTH: DATE/TIME: Since mud rotary, GWL not checked.	NOTES:
DATE COMPLETED: 5/13/99	GWL: DEPTH: DATE/TIME:	
FIELD GEOLOGIST: J.LILLY	DRILLING METHOD: MUD ROTARY IN SOIL	
CHECKED BY: RMW/JAA	NQ CORE IN ROCK	
DRILLING CO.: CAROLINA DRILLING	DRILLER: GERALD HELPER: BRETT	RIG: CME-55

Paul C. Rizzo Associates, Inc.
CONSULTANTS

MONITOR WELL INSTALLATION SHEET

WELL NO. PZ 201

PROJECT NAME: WINYAH BAY
PROJECT NO.: 991988.01
BORING NO.: PZ201
DATE OF INSTALLATION: 5/13/99

FIELD ENG./GEO.: J. LILLY DATE: 5/15/99
CHECKED BY: ______ DATE: ______
DATE OF DEVELOPMENT: 5/14/99

BOREHOLE DRILLING

DRILLING METHOD: MUD ROTARY TO TOP ROCK NQ CORE
DRILLING FLUID (S) USED:
FLUID: BENTONITE FROM 0 TO 35.0
FLUID: ______ FROM ______ TO ______

TYPE OF BIT: 3 7/8 DRAG BIT
CASING SIZE (S) USED:
SIZE 3 FROM 0.0 TO 14
SIZE ______ FROM ______ TO ______

MONITOR WELL DESCRIPTION

TYPE: PIEZOMETER
DIAMETER OF SCREENED SECTION: 3/4"
PERFORATION TYPE:
SLOTS ☐ HOLES ☐ SCREEN ☒
AVERAGE SIZE OF OPENINGS: ______
SCREEN MANUFACTURER: ______

RISER PIPE MATERIAL: PVC
RISER PIPE DIAMETERS:
O.D. 3/4" I.D. ~~1/2"~~ 0.75
LENGTH OF PIPE SECTIONS: ______
JOINING METHOD: SCREW

PROTECTION SYSTEM

RISER PROTECTIVE PIPE LENGTH: ______
PROTECTIVE PIPE O.D.: ______

OTHER PROTECTION: FLAG

ITEM	DISTANCE ABOVE / BELOW GROUND SURFACE (FT.)	ELEVATION (MSL)
TOP OF PROTECTIVE PIPE	NA	—
TOP OF RISER PIPE	+1.4'	9.06
GROUND SURFACE	0.0	7.72
BOTTOM OF PROTECTIVE PIPE	NA	
BOREHOLE FILL MATERIALS:		
GROUT / SLURRY	TOP 0.0 BOTTOM -26.0	TOP 7.72 BOTTOM -18.28
BENTONITE	TOP -26.0 BOTTOM -28.0	TOP -18.28 BOTTOM -16.28
FINE SAND	TOP — BOTTOM —	TOP — BOTTOM —
#2 COARSE SAND	TOP -28.0 BOTTOM -35.0	TOP -16.28 BOTTOM -27.28
SCREENED SECTION	TOP -31.0 BOTTOM -32.0	TOP -23.28 BOTTOM -24.28
PIEZOMETER/WELL TIP	-32.0	-24.28
BOTTOM OF BOREHOLE	-35.0	-27.28
GWL AFTER INSTALLATION		

WAS THE PIEZOMETER/WELL DEVELOPED AFTER INSTALLATION? YES ☒ NO ☐
WAS A SENSITIVITY TEST PERFORMED ? YES ☐ NO ☒

REMARKS: ______

ATTACHMENT 3

CPT Sounding Data

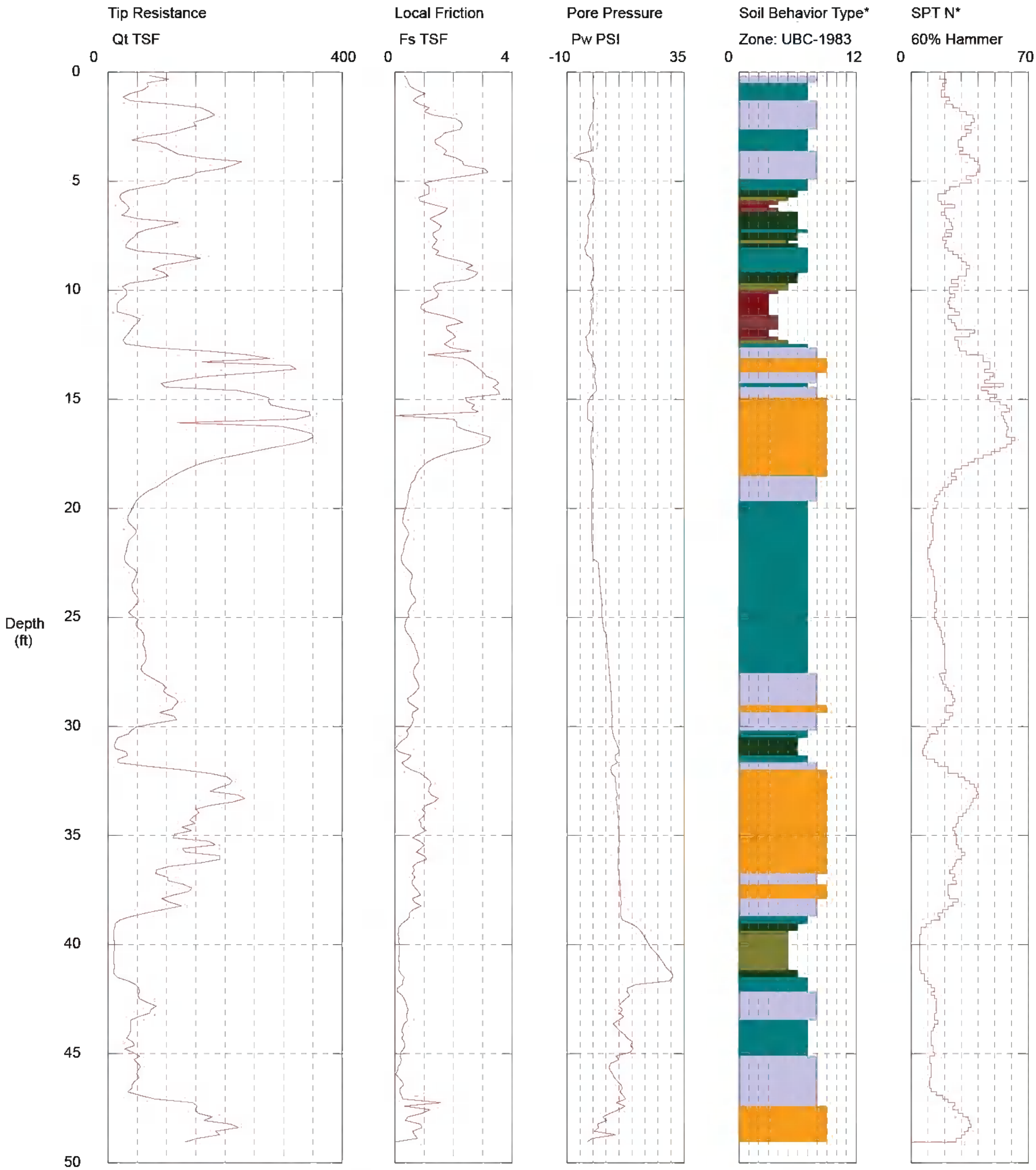
ATTACHMENT 3-A

CPT Sounding Logs
(Provided by Mid-Atlantic Drilling)

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-1
Cone Used: DSG1156

CPT Date/Time: 2/26/2013 5:01:34 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 49.05 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 (*)

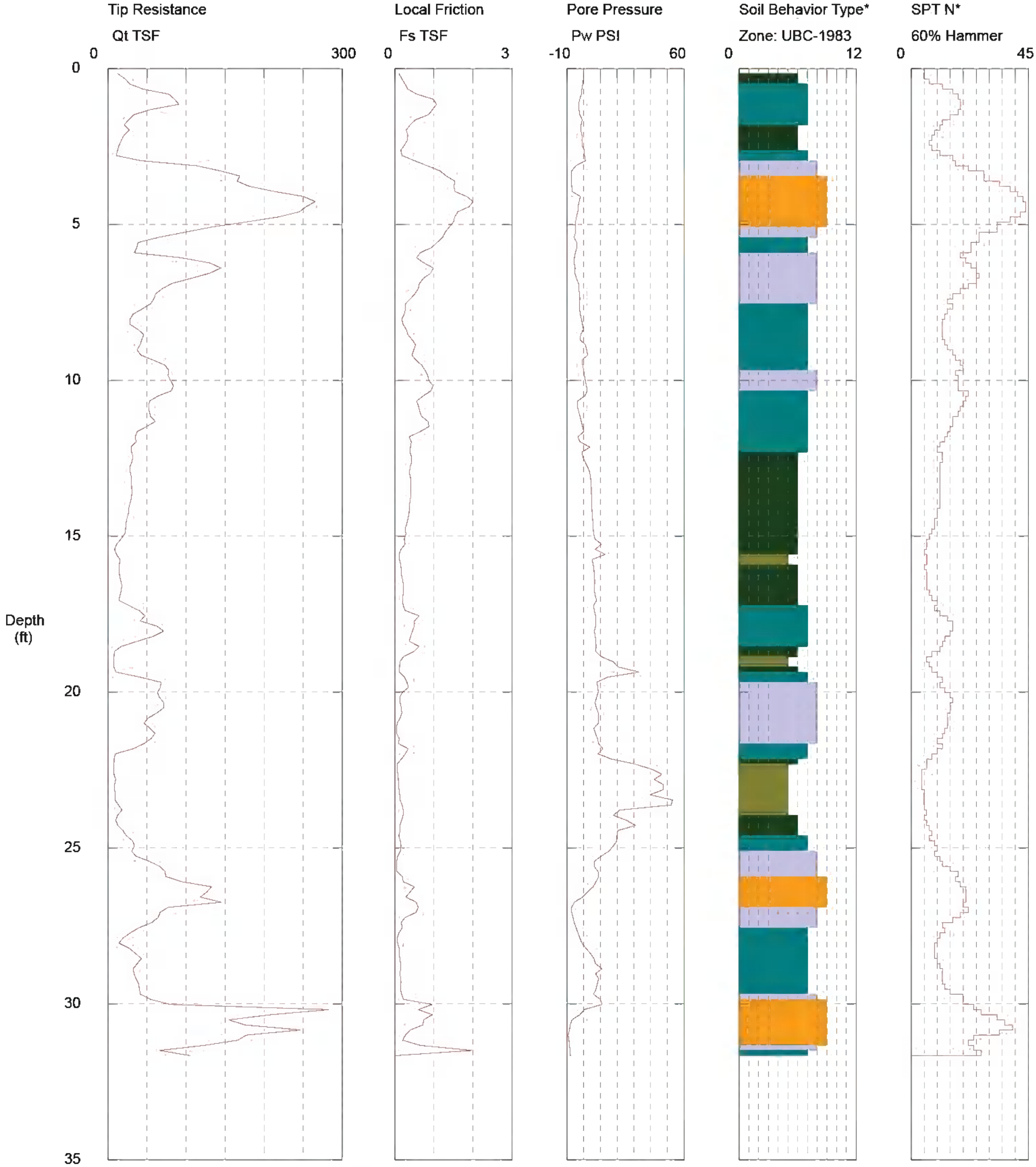
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-2
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 2:34:07 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 31.66 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 (*)

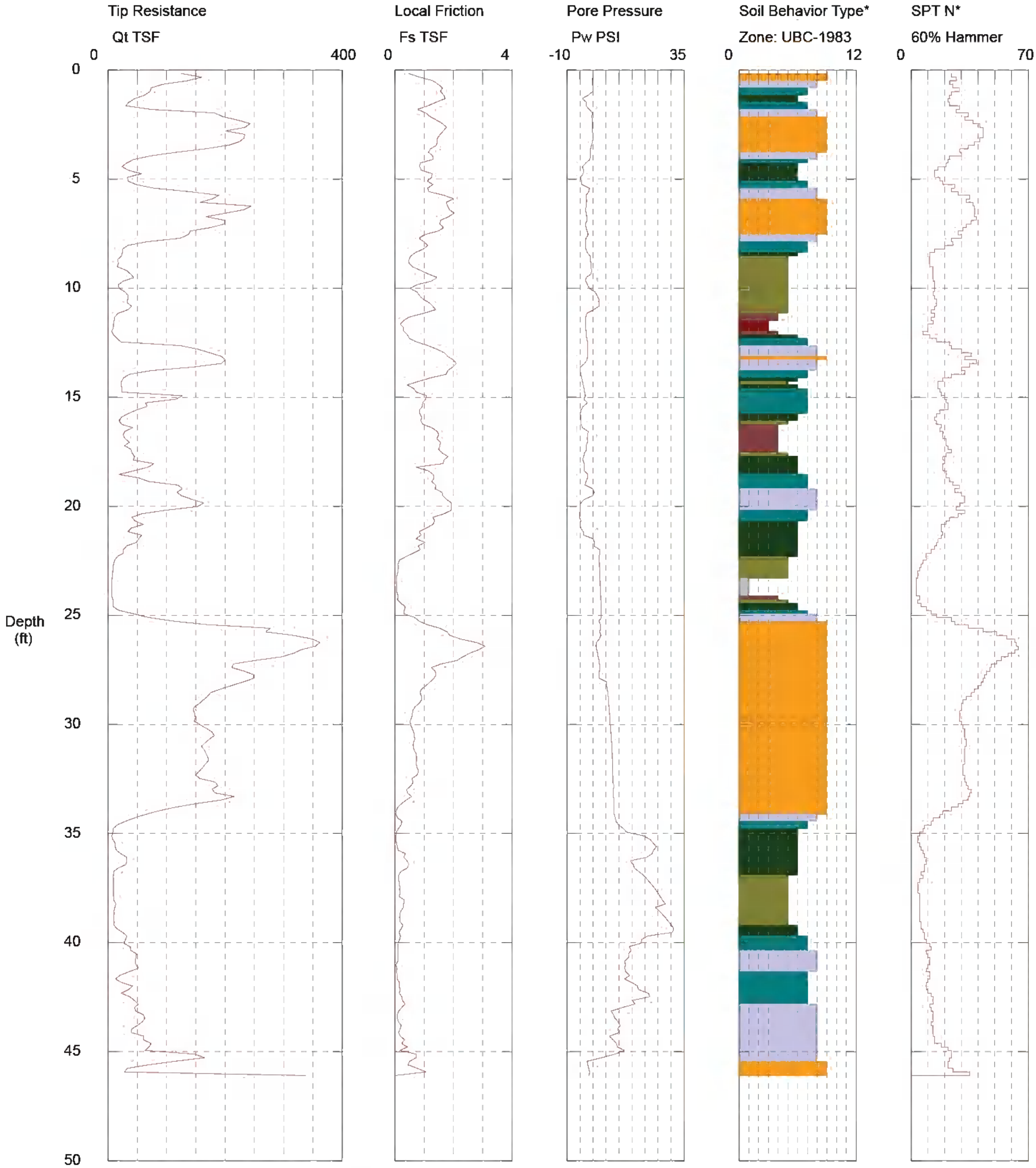
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-3
Cone Used: DSA1123

CPT Date/Time: 2/20/2013 11:07:32 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 46.10 feet

Depth Increment = 0.164 feet

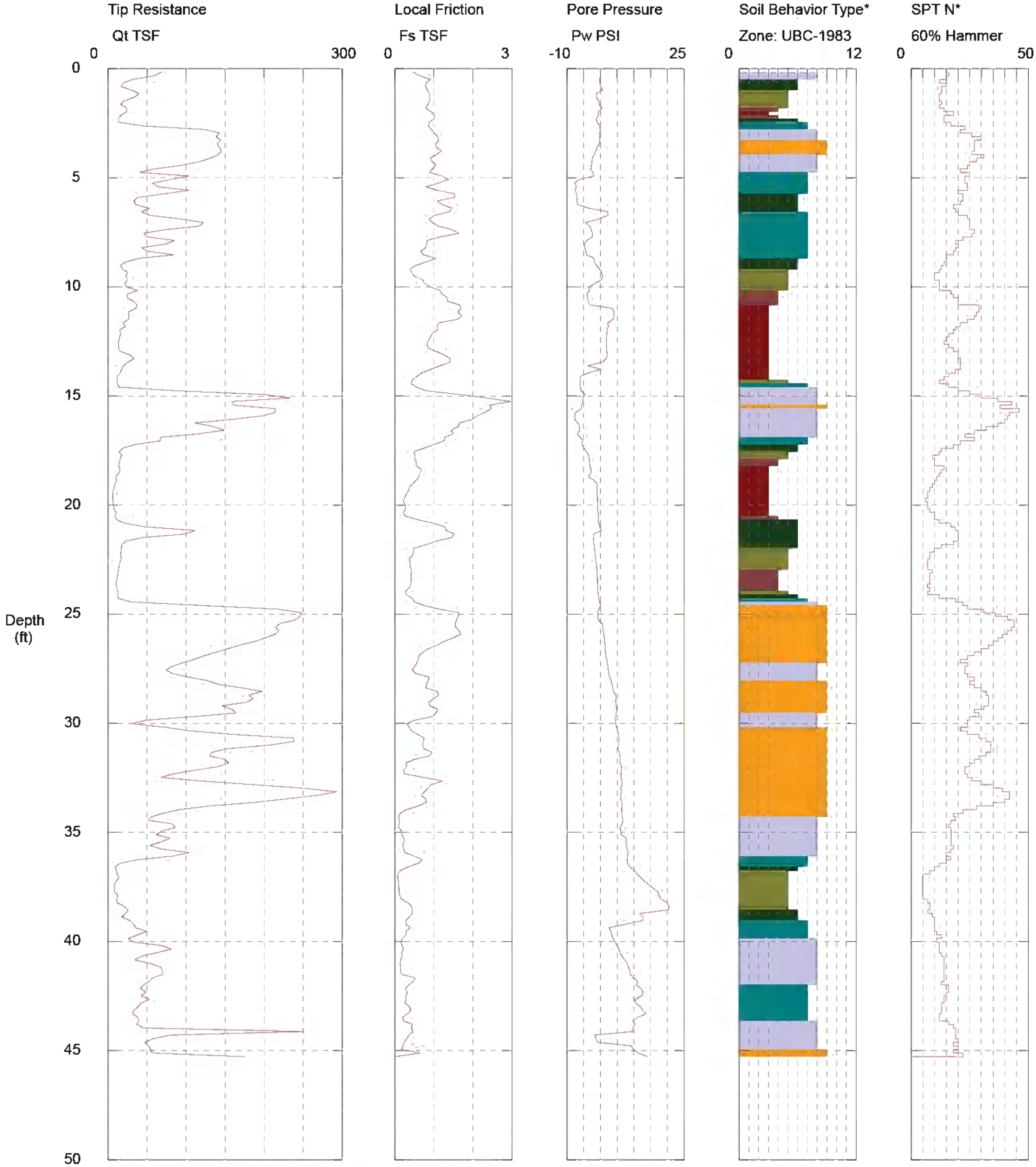
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-4
Cone Used: DSA1123

CPT Date/Time: 2/20/2013 2:11:22 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.28 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 (*)

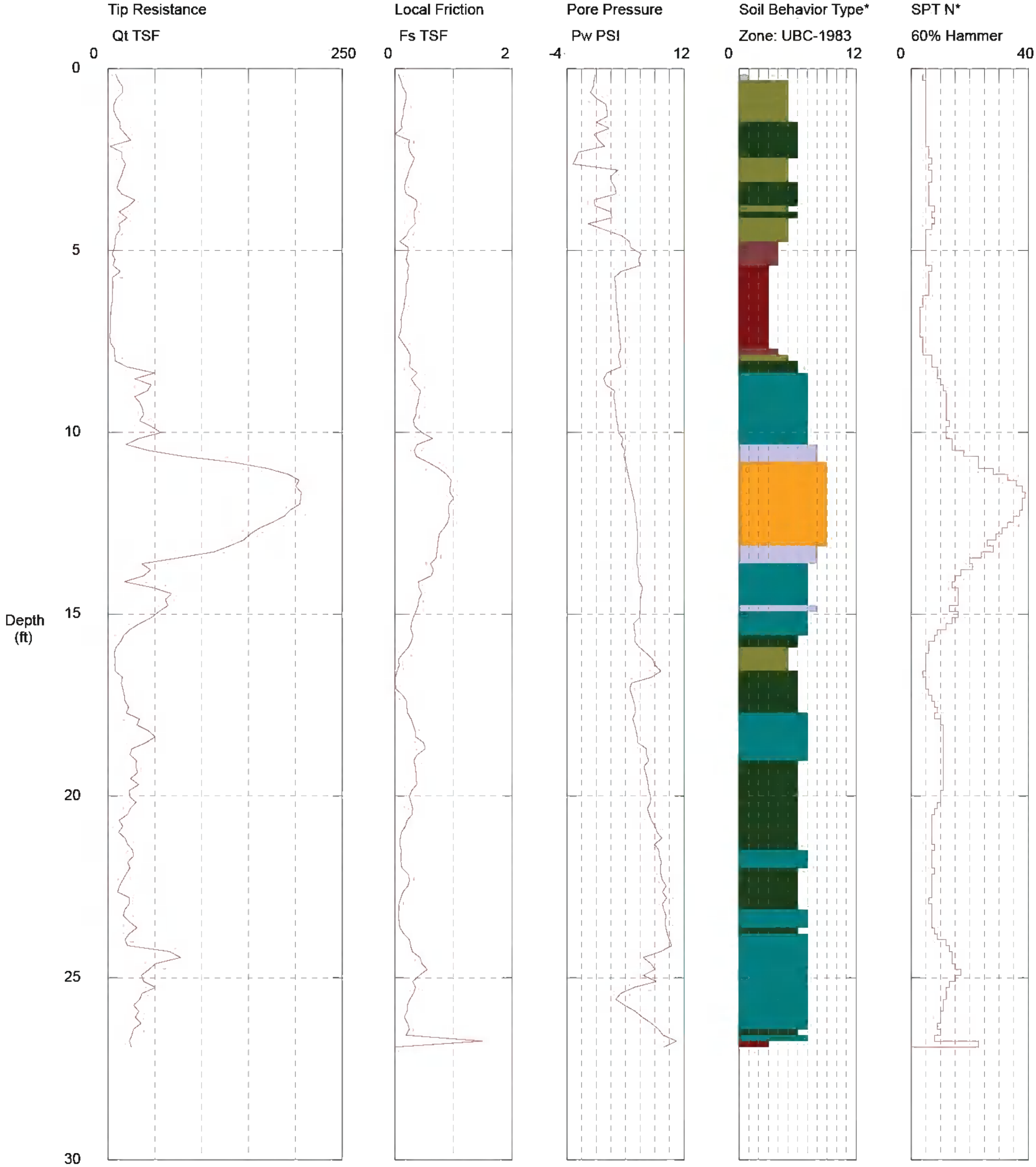
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-5
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 3:23:21 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 26.90 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 (*)

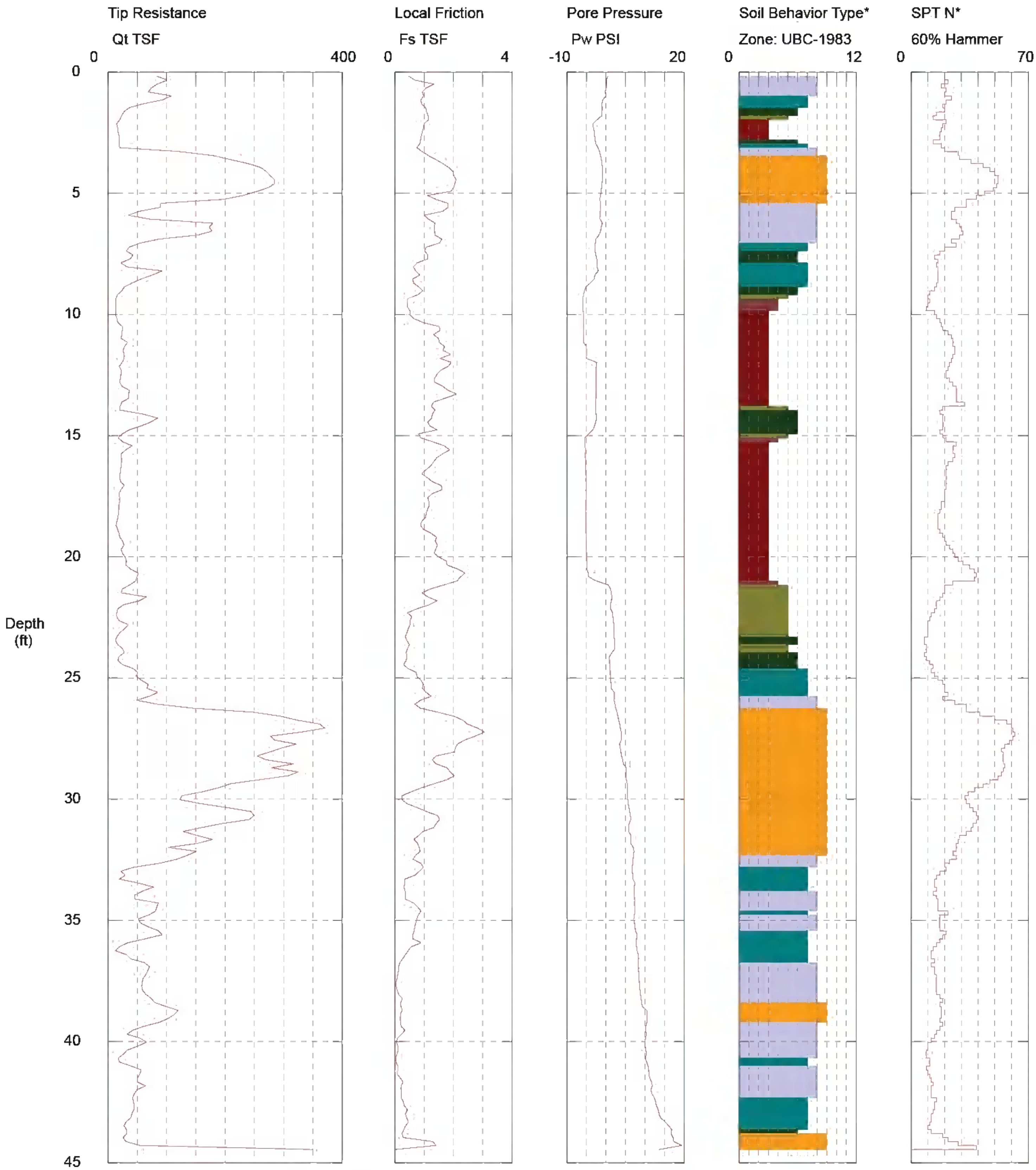
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-6
Cone Used: DSA1123

CPT Date/Time: 2/20/2013 3:00:47 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 44.46 feet

Depth Increment = 0.164 feet

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

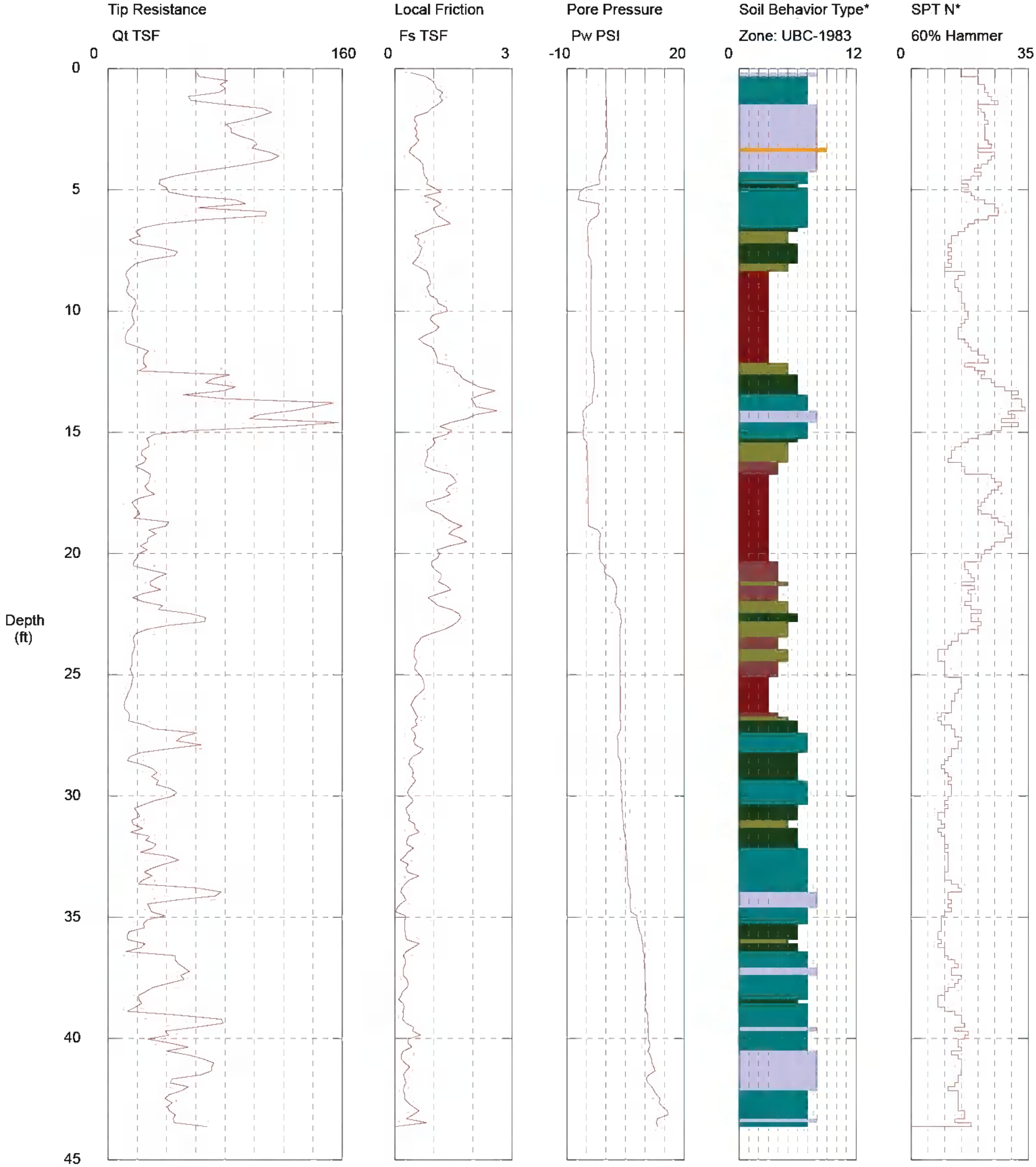
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-7
Cone Used: DSA1123

CPT Date/Time: 2/20/2013 3:49:12 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



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 (*)

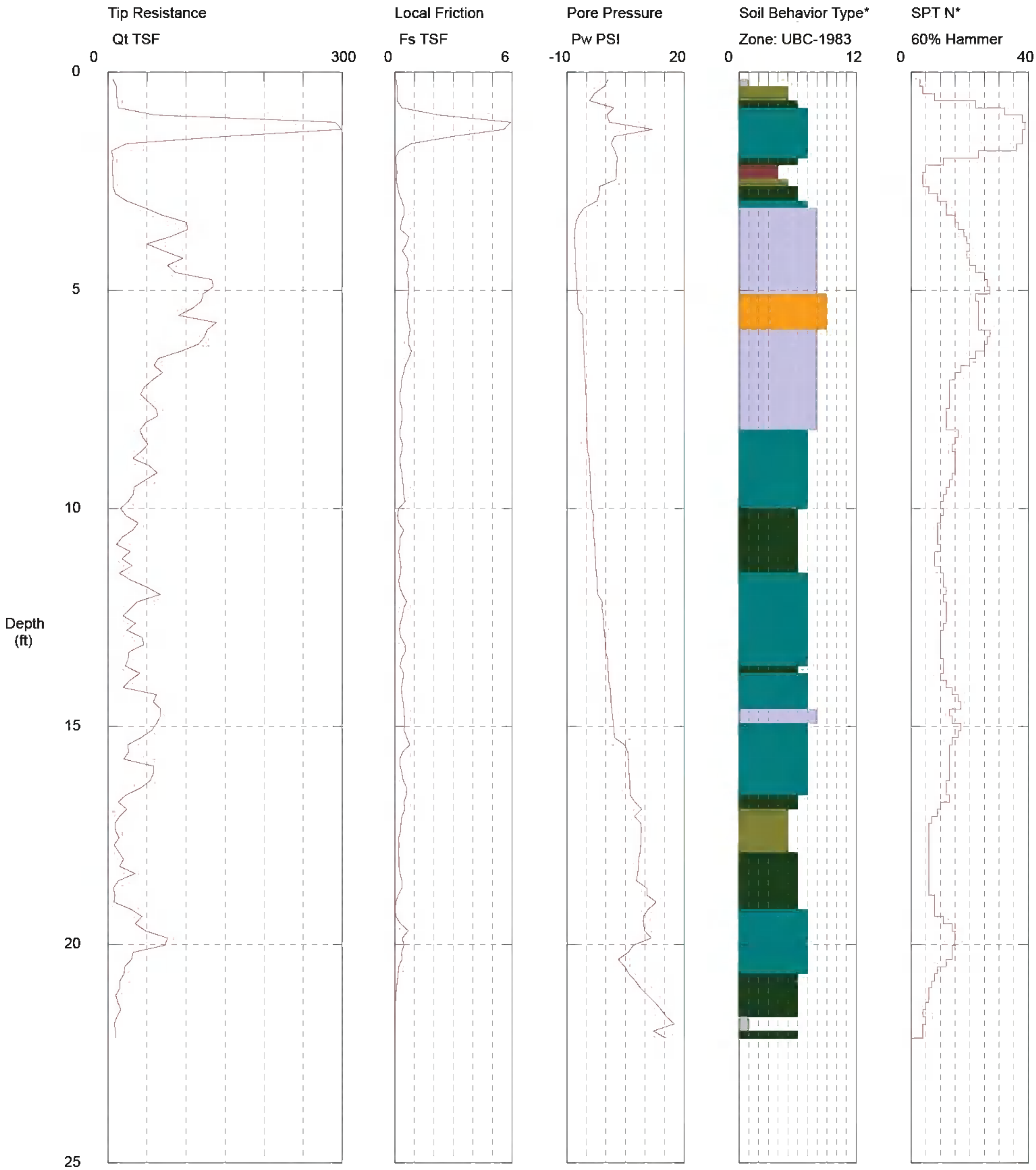
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-8
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 4:23:02 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 22.15 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 (*)

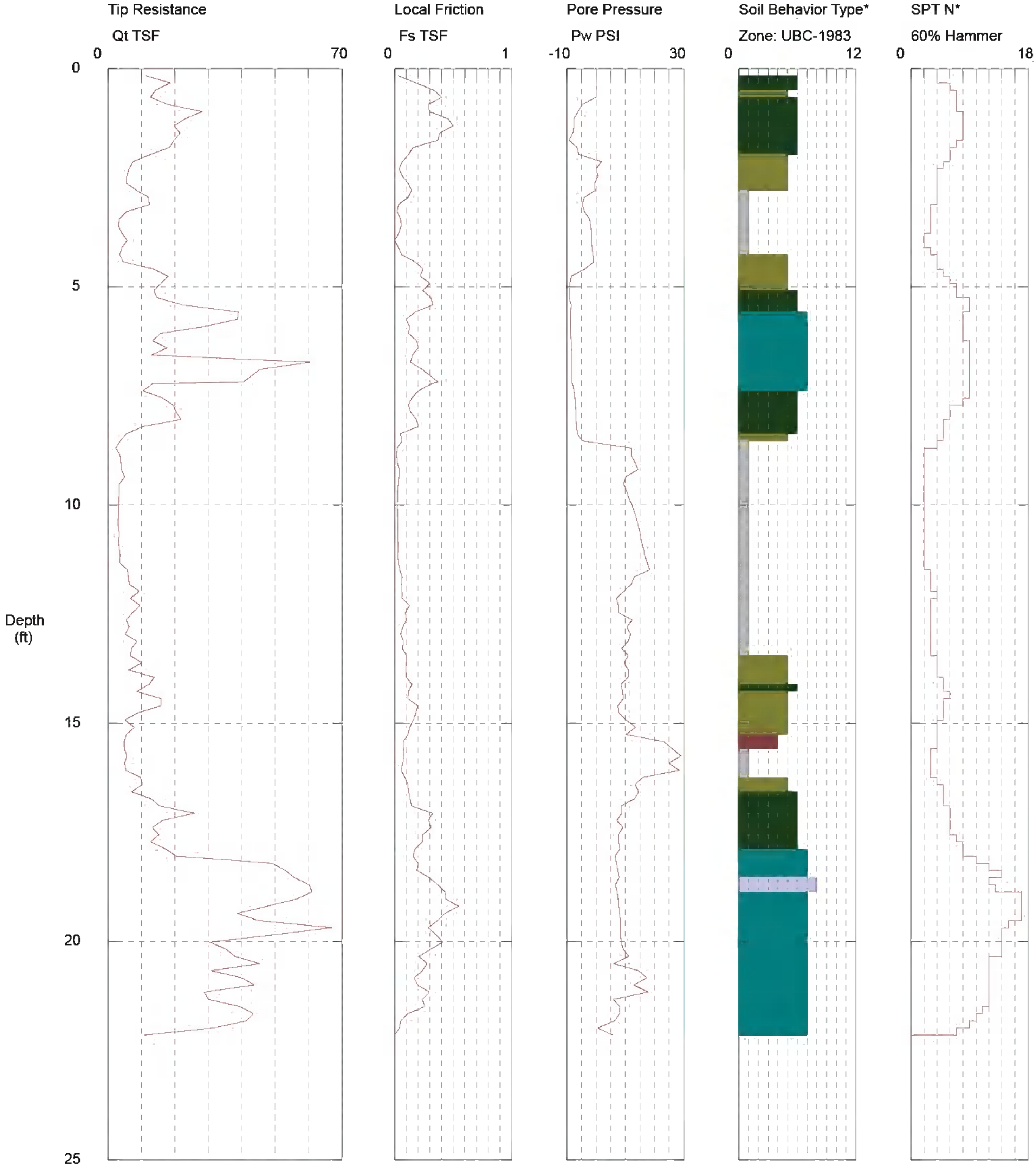
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-9
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 4:56:30 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 22.15 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 (*)

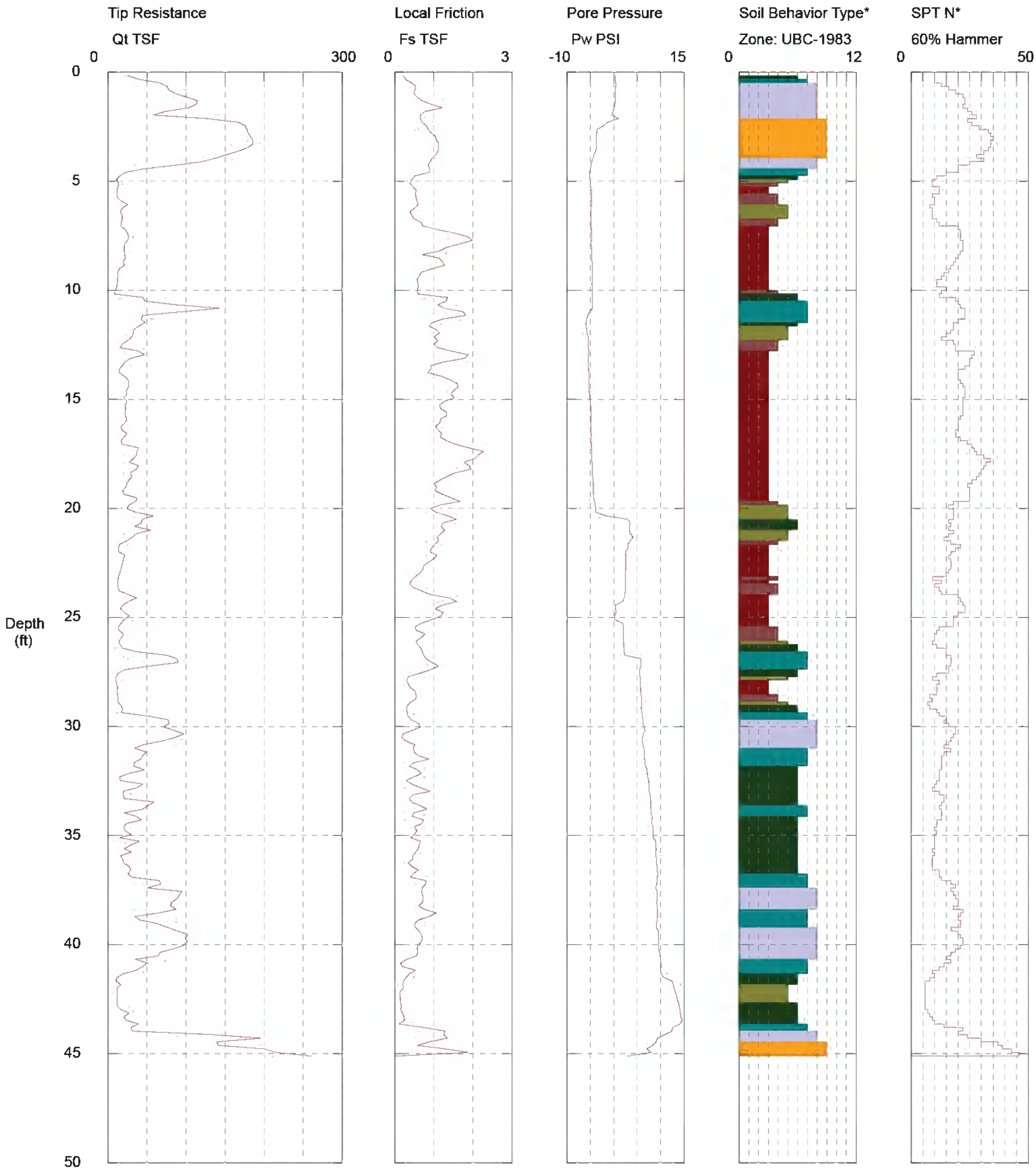
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-10
Cone Used: DSA1123

CPT Date/Time: 2/20/2013 4:39:42 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.11 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 (*)

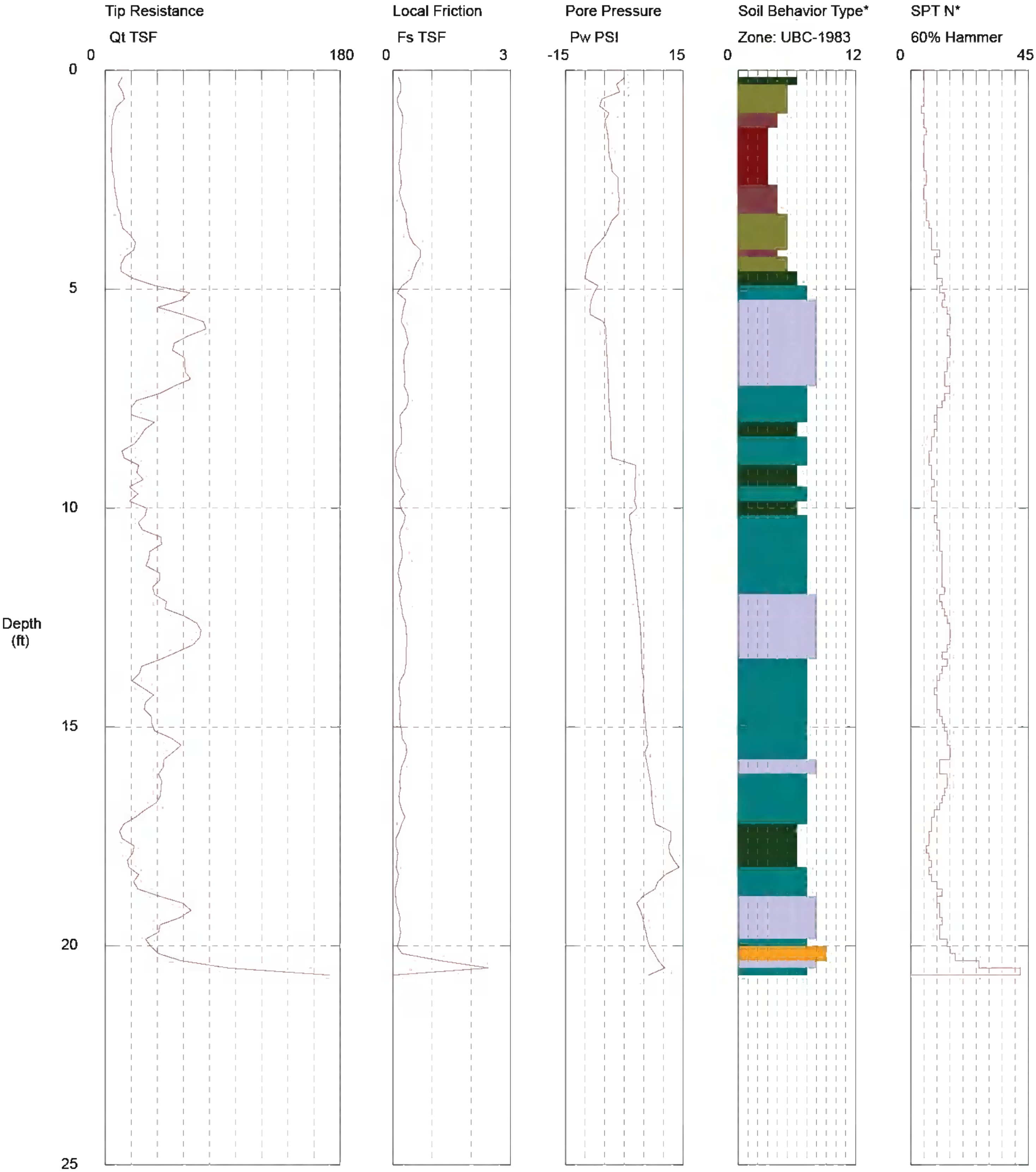
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-11
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 7:25:21 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 20.67 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 (*)

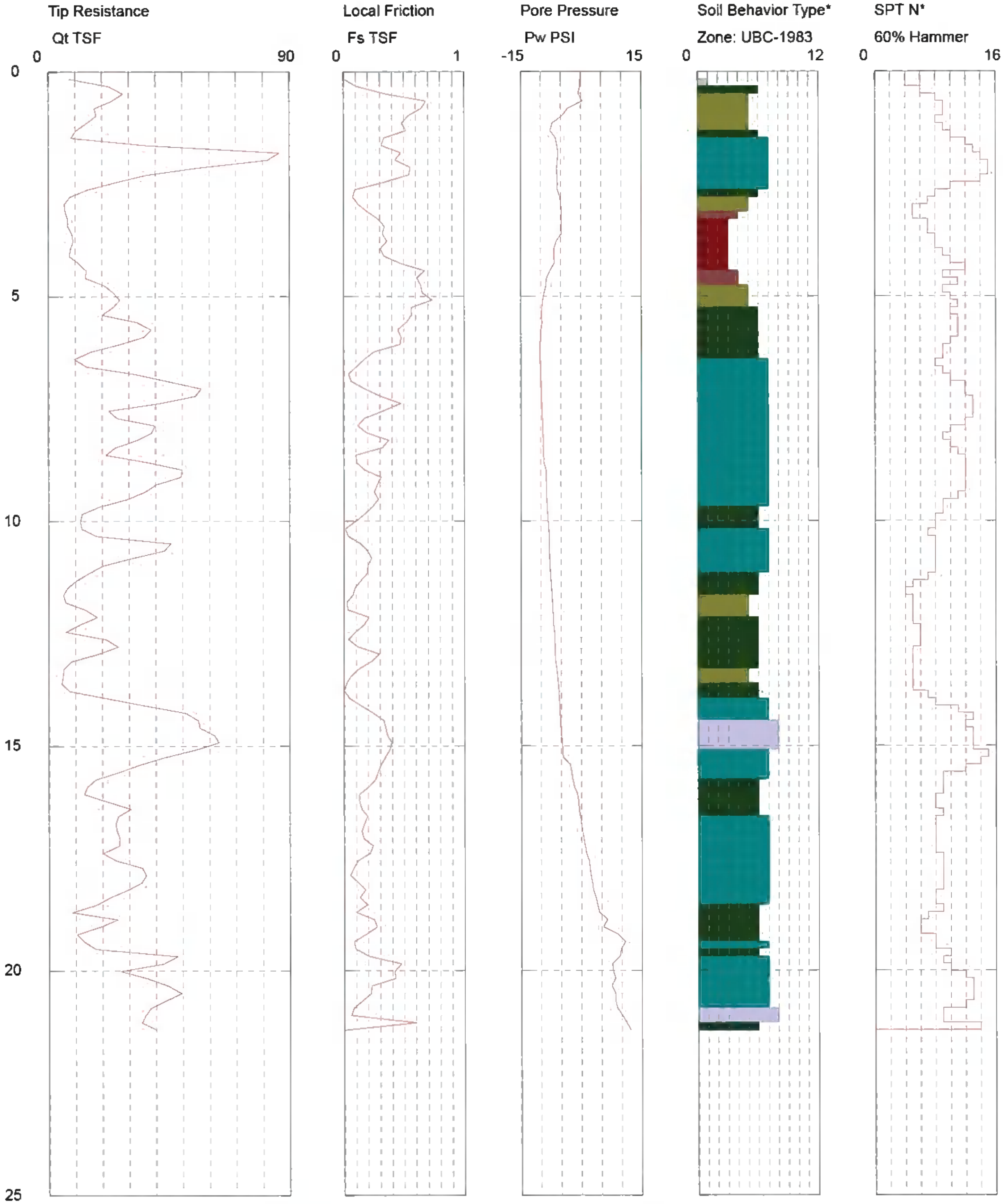
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-12
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 5:23:33 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 21.33 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 (*)

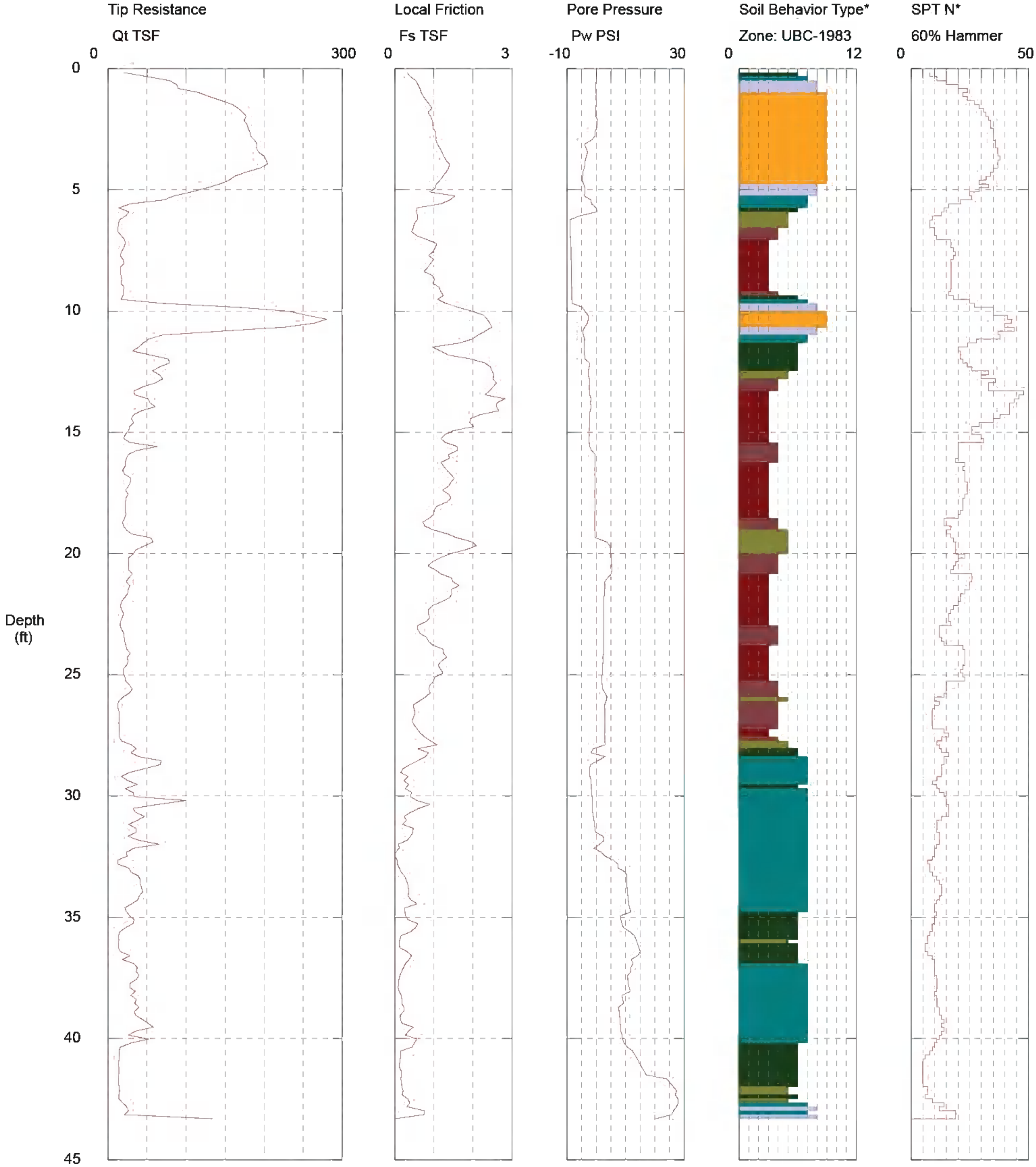
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-13
Cone Used: DSA1123

CPT Date/Time: 2/21/2013 8:12:07 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 43.31 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 (*)

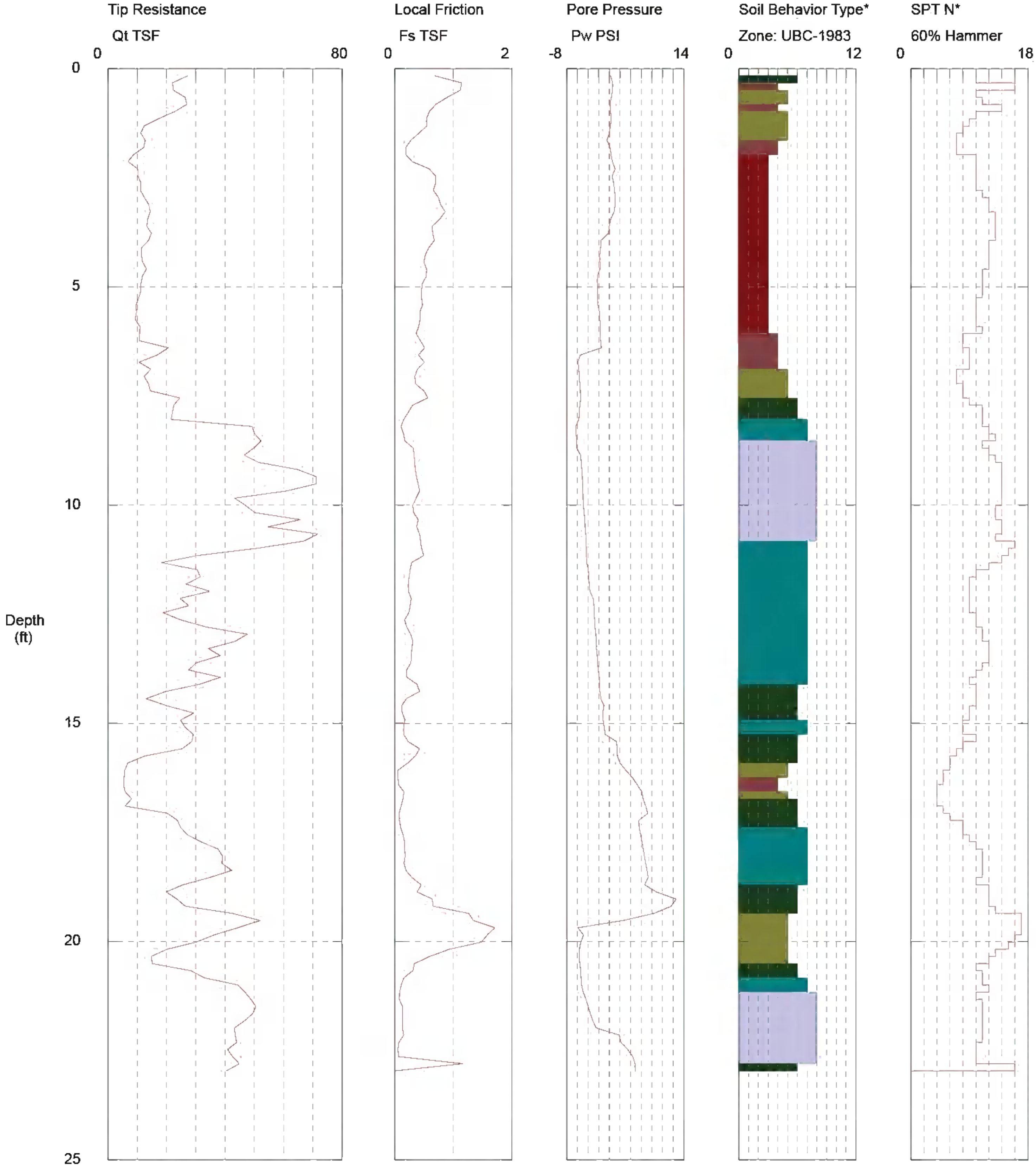
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-14
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 7:50:56 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 22.97 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 (*)

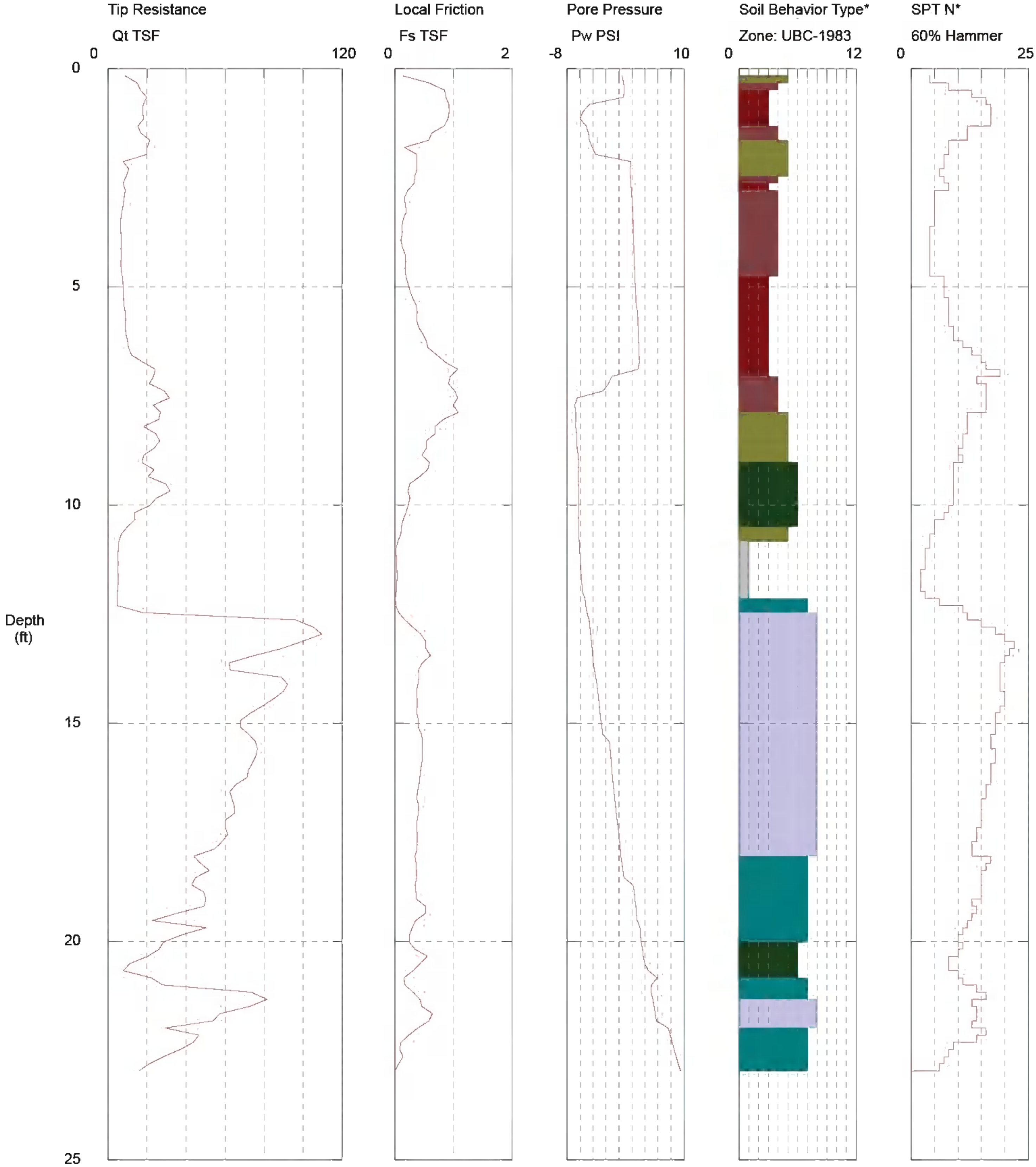
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-15
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 8:27:04 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 22.97 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 (*)

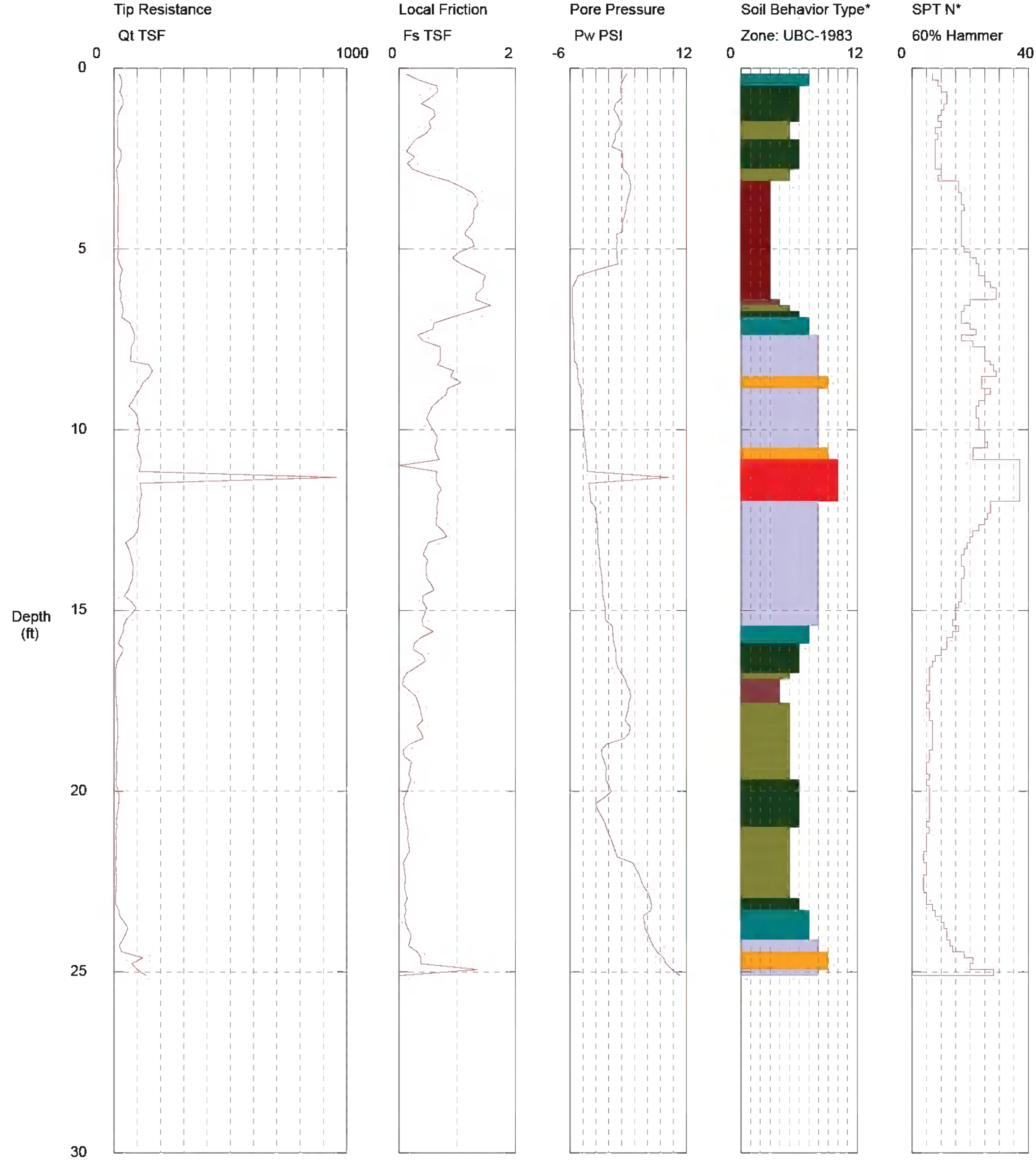
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-16
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 8:49:33 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 25.10 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 (*)

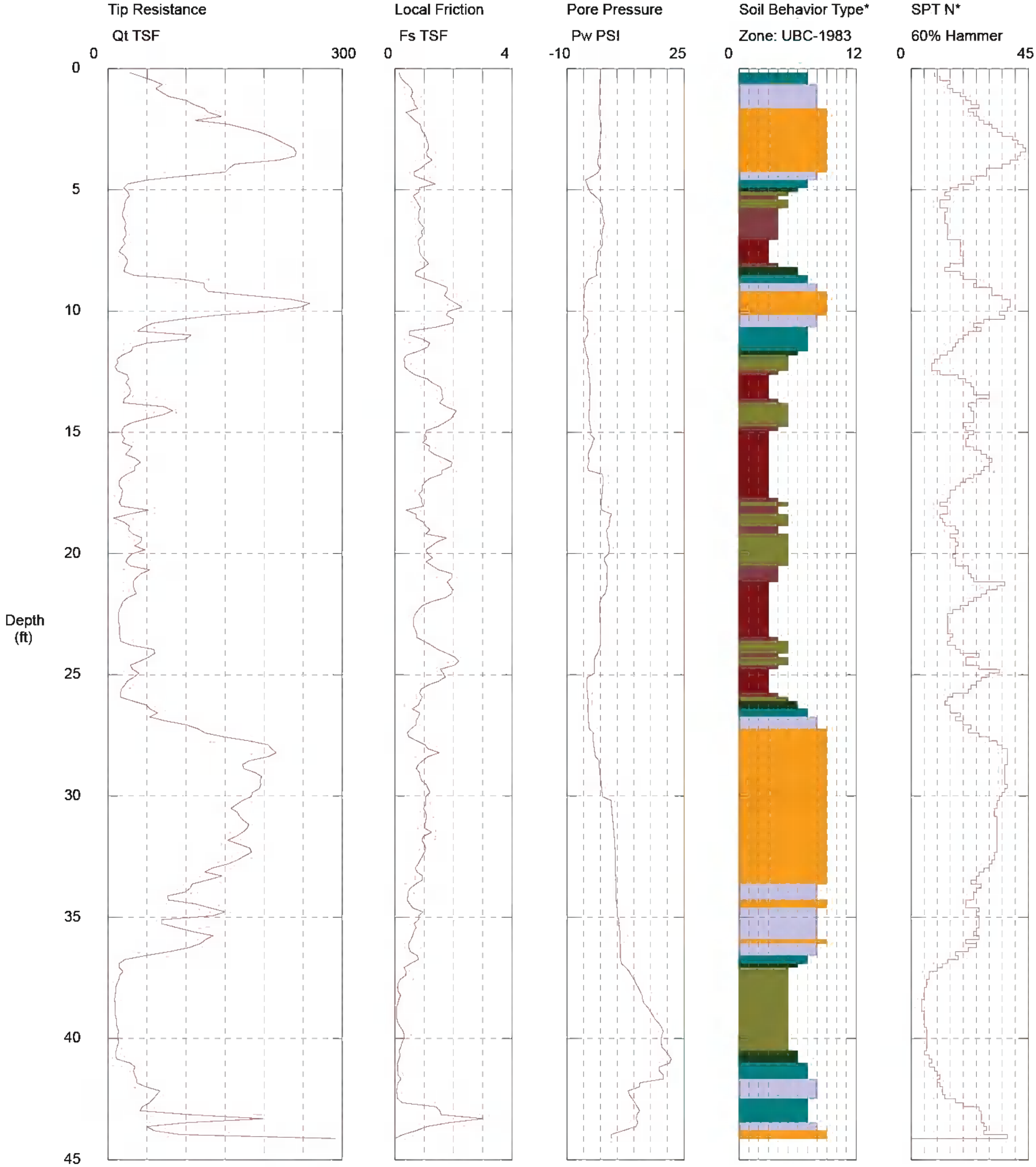
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-17A
Cone Used: DSG1156

CPT Date/Time: 2/21/2013 11:13:02 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 44.13 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 (*)

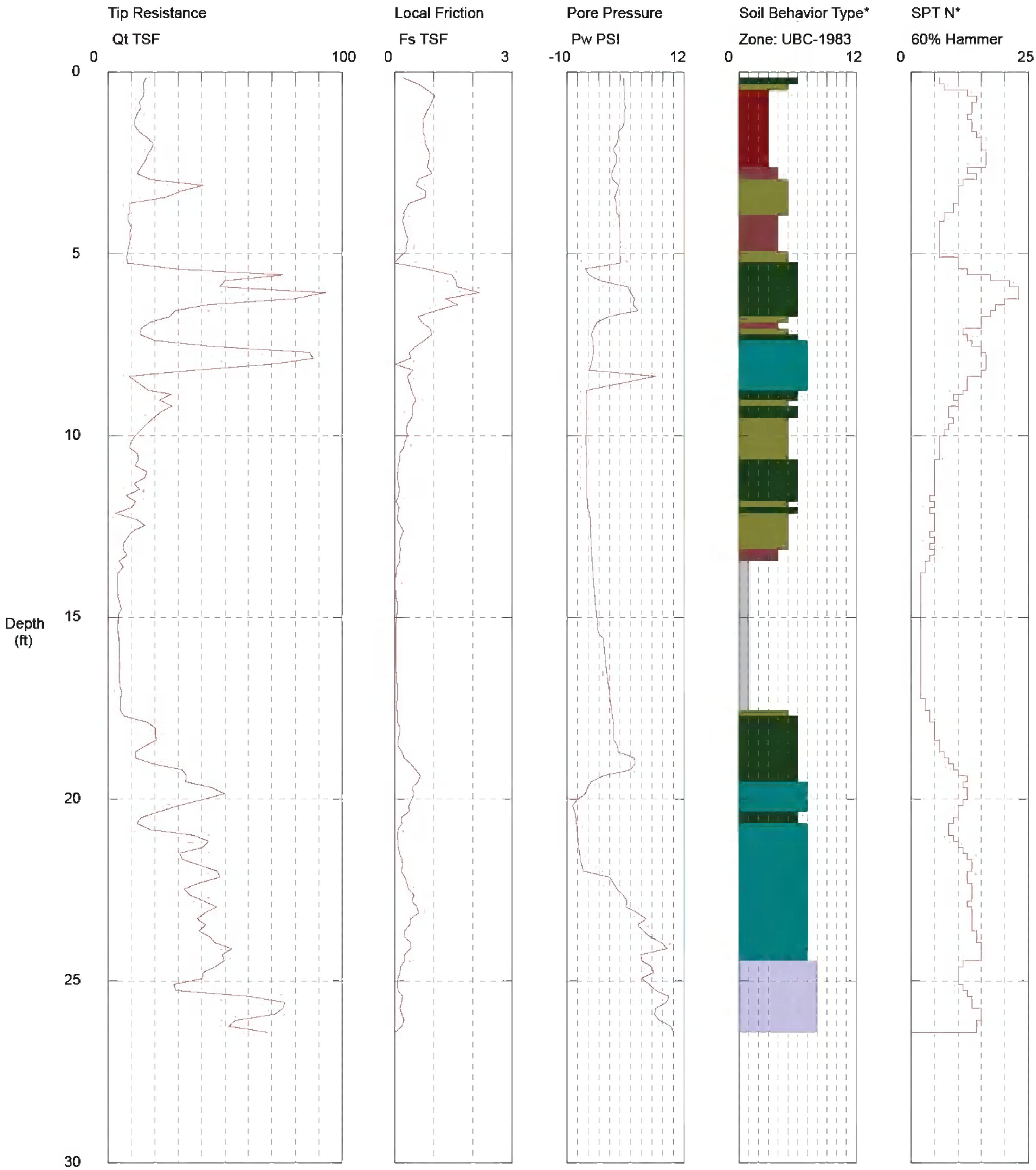
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-18
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 9:15:14 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 26.41 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 (*)

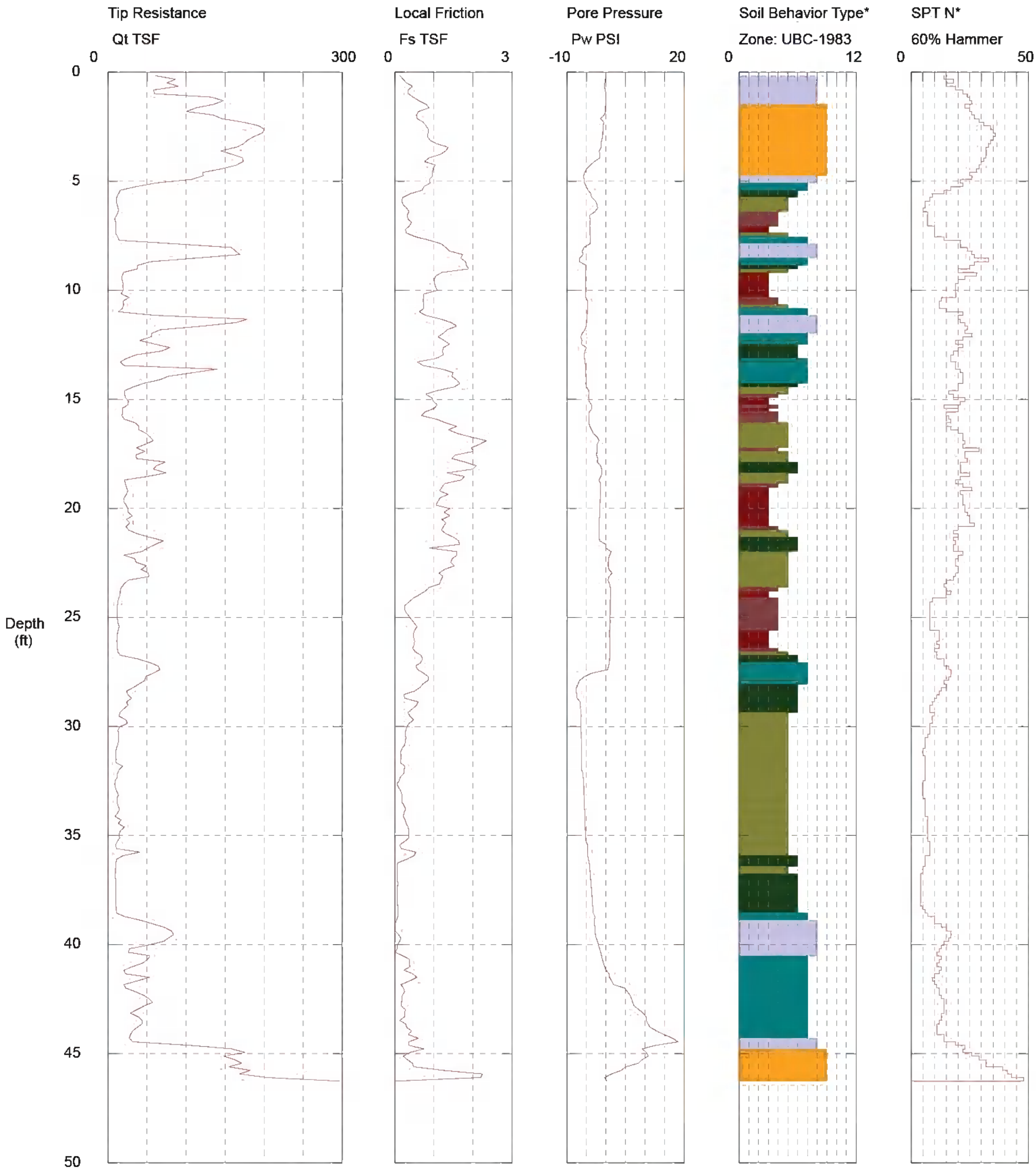
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-19
Cone Used: DSG1156

CPT Date/Time: 2/21/2013 1:46:12 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 46.26 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 (*)

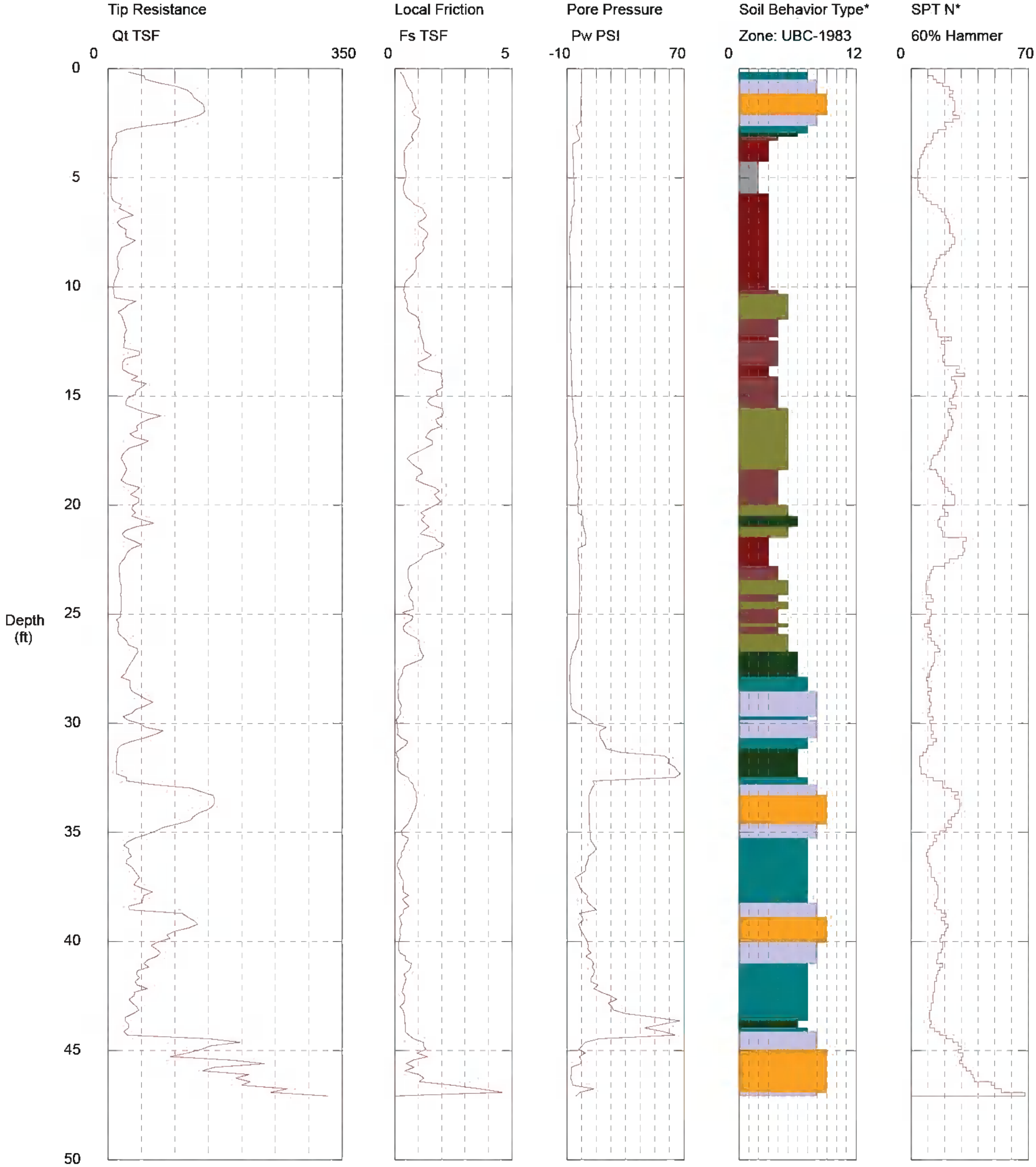
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-20
Cone Used: DSA1123

CPT Date/Time: 2/21/2013 2:36:51 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 47.08 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 (*)

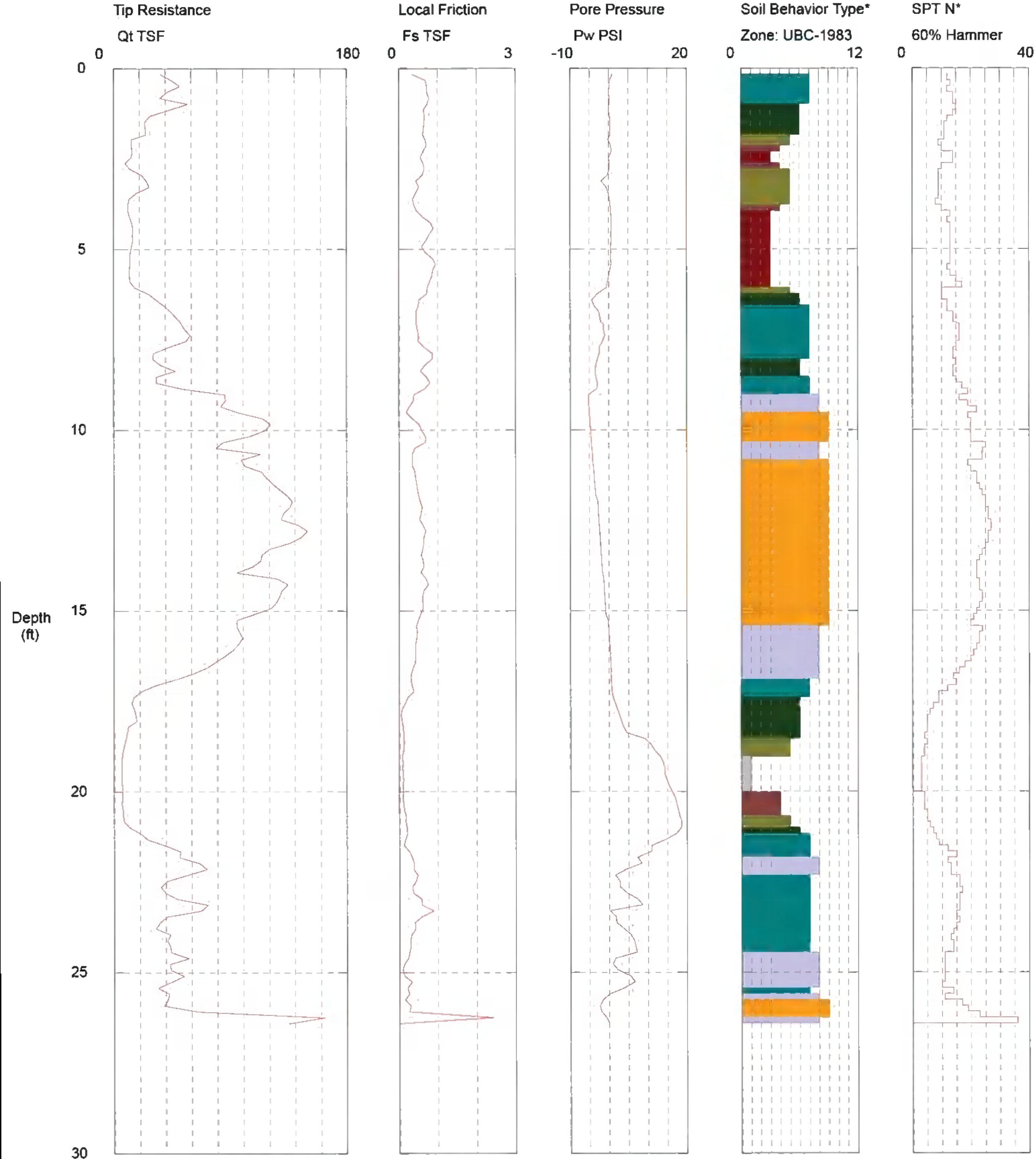
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-21
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 9:44:29 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 26.41 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 (*)

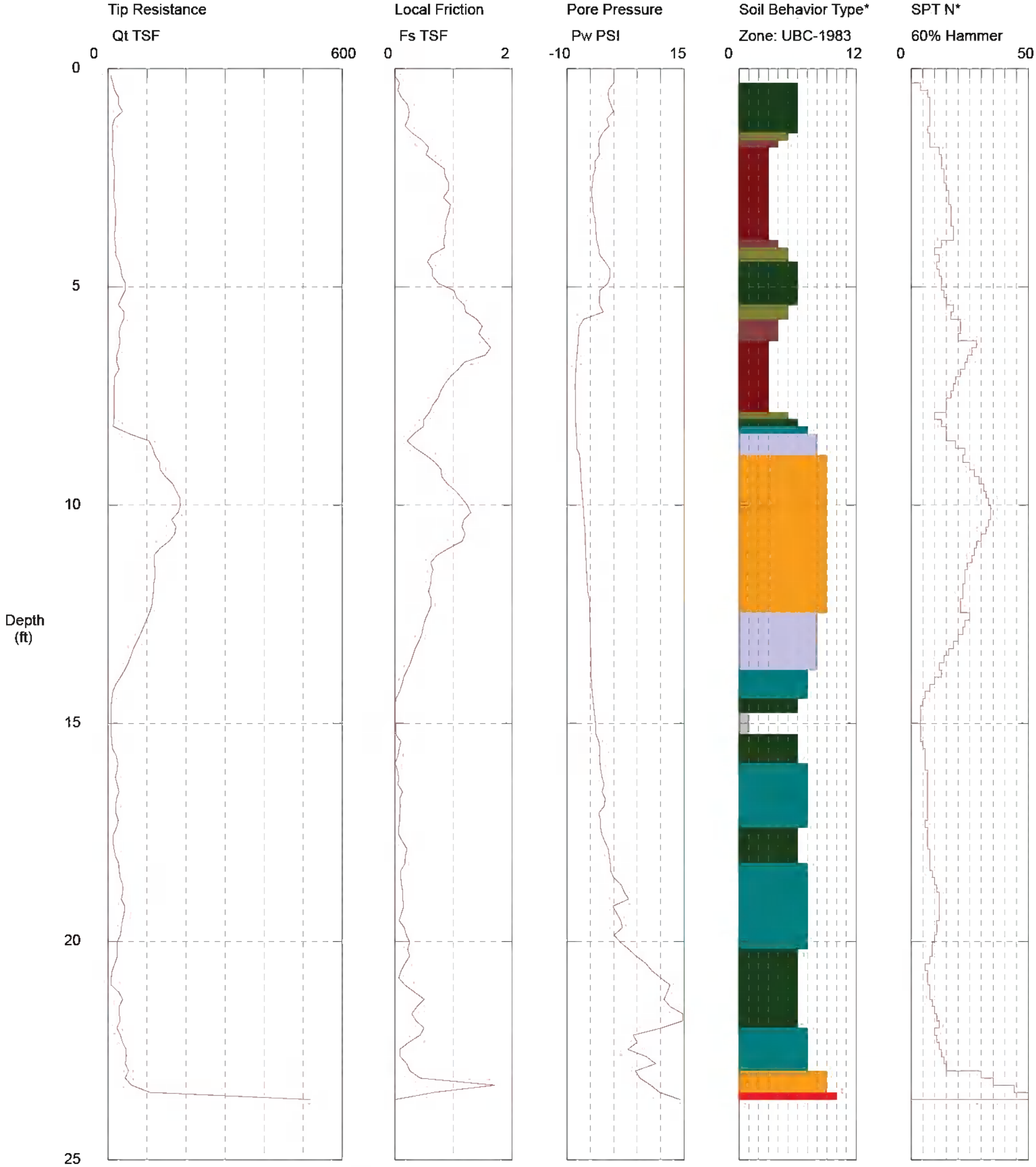
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-22
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 12:00:21 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 23.62 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 (*)

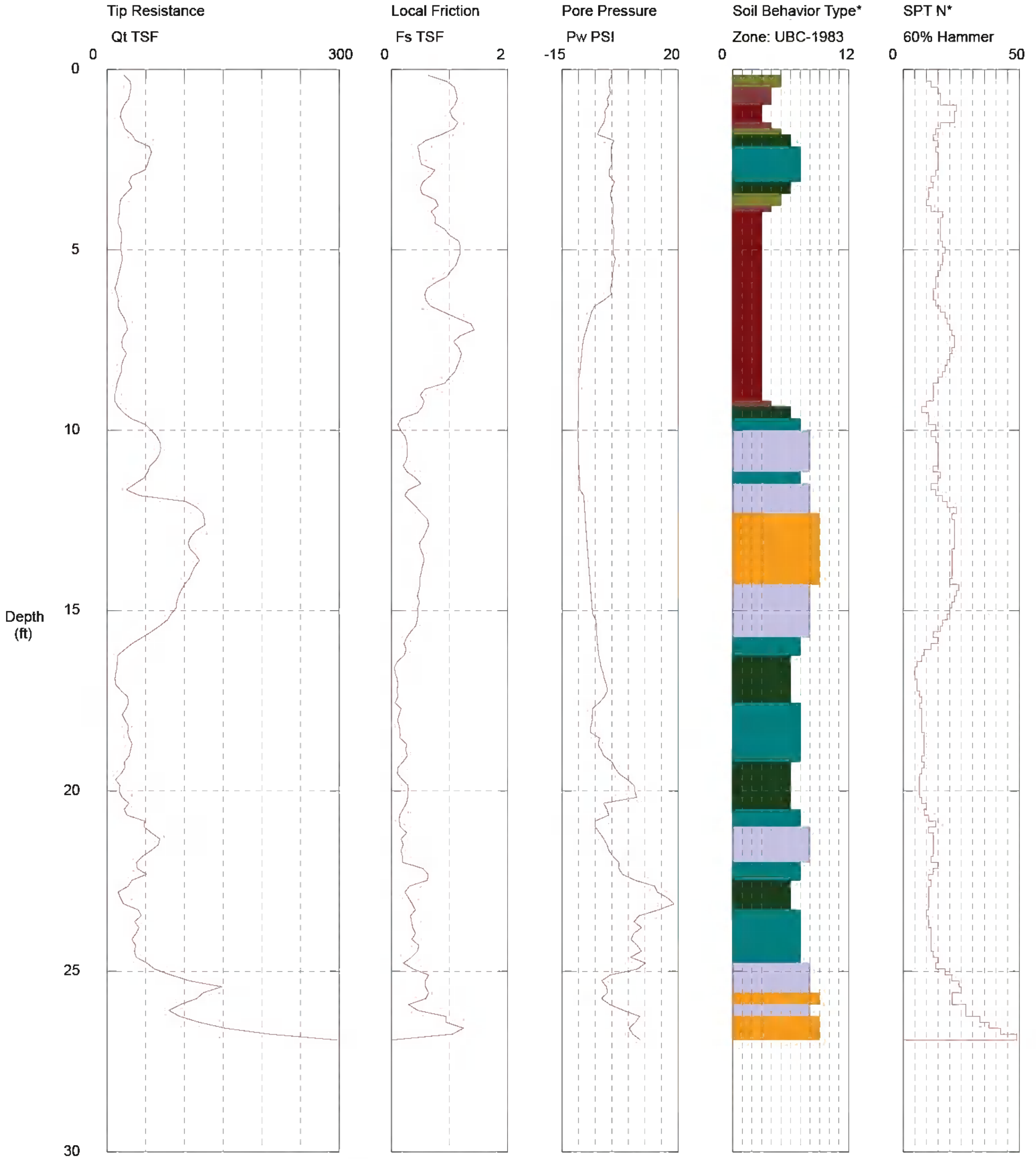
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-23
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 11:35:40 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 26.90 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 (*)

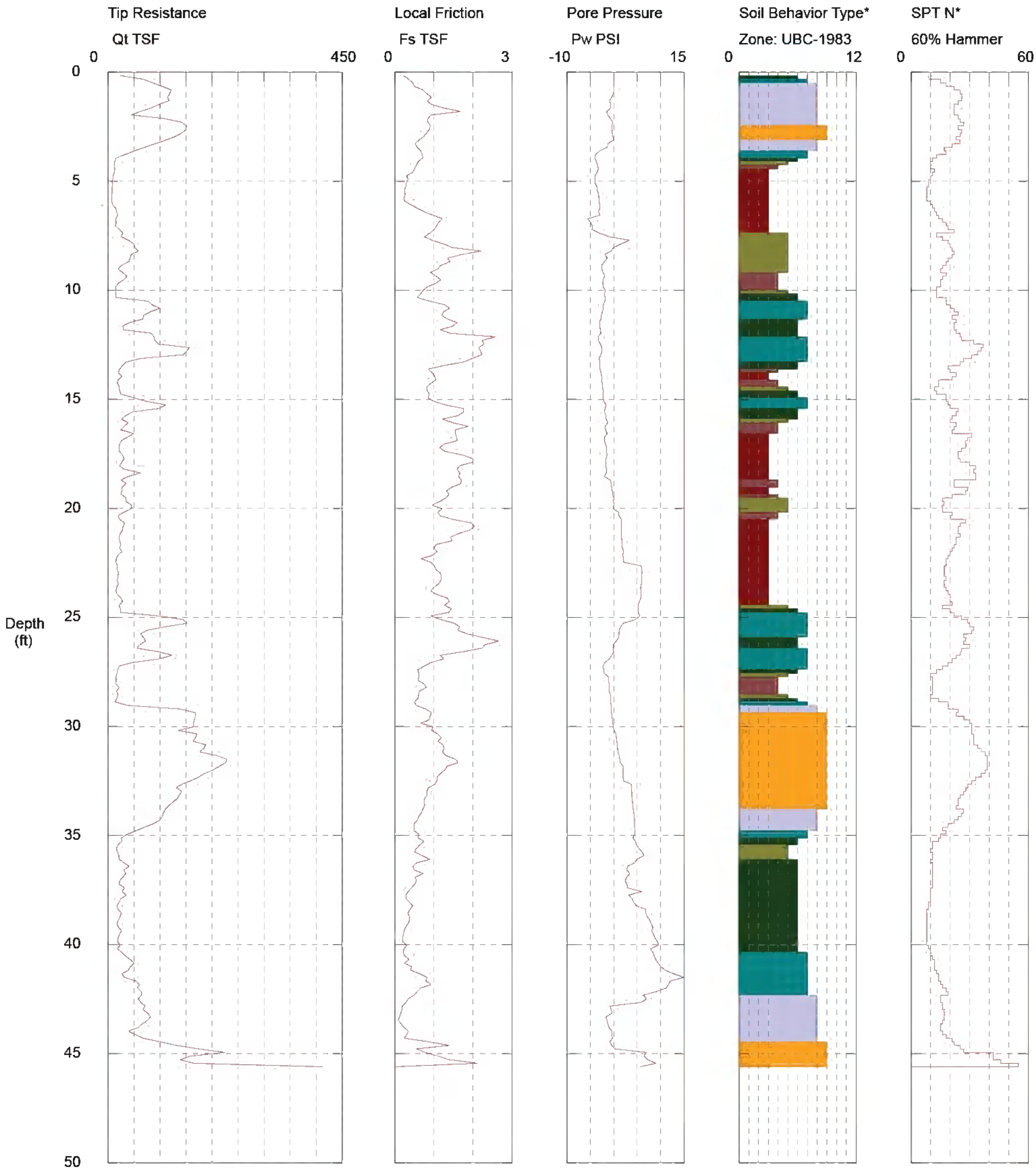
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-24
Cone Used: DSG1156

CPT Date/Time: 2/21/2013 3:32:37 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.60 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 (*)

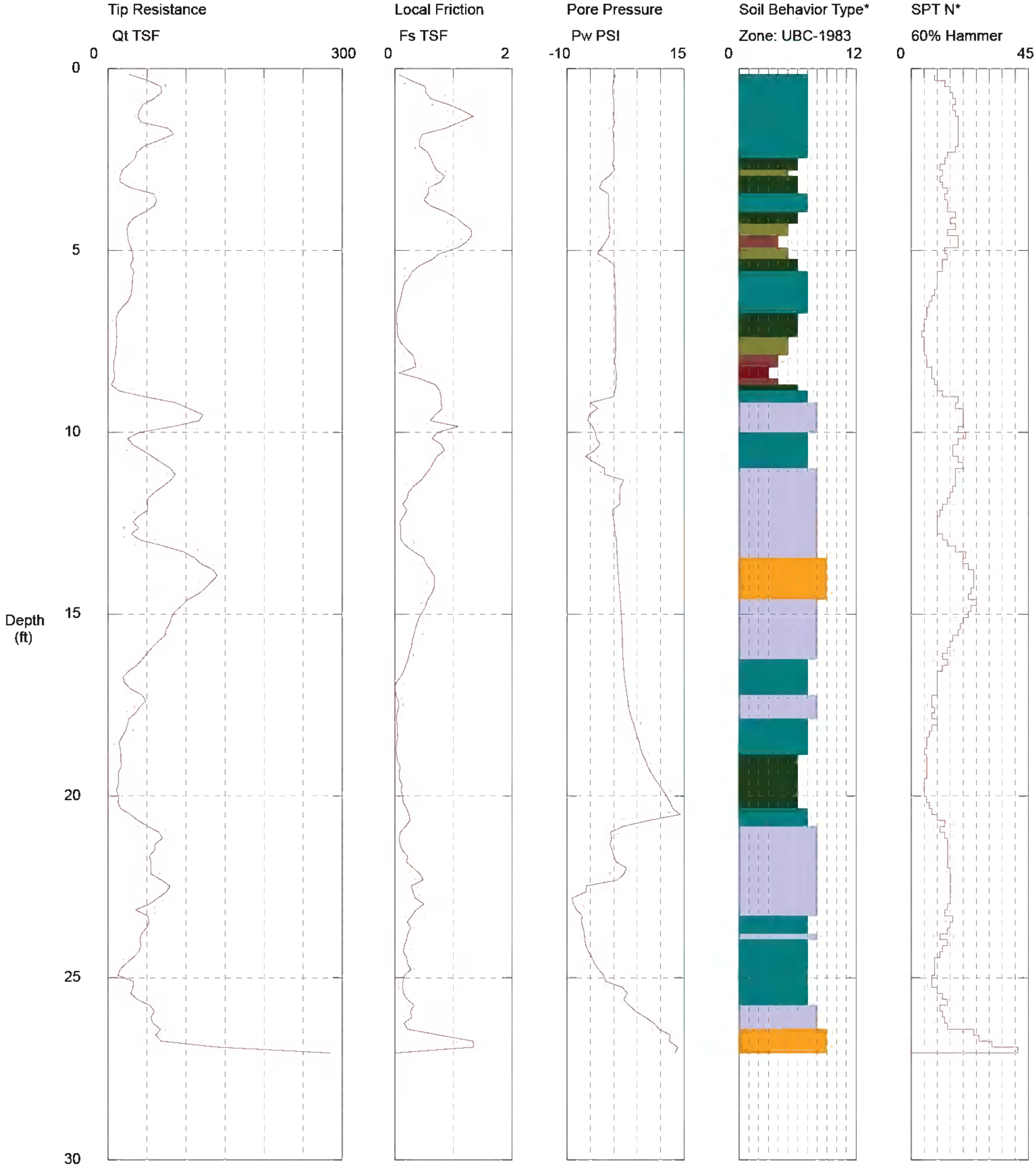
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-25
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 12:21:32 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 27.07 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 (*)

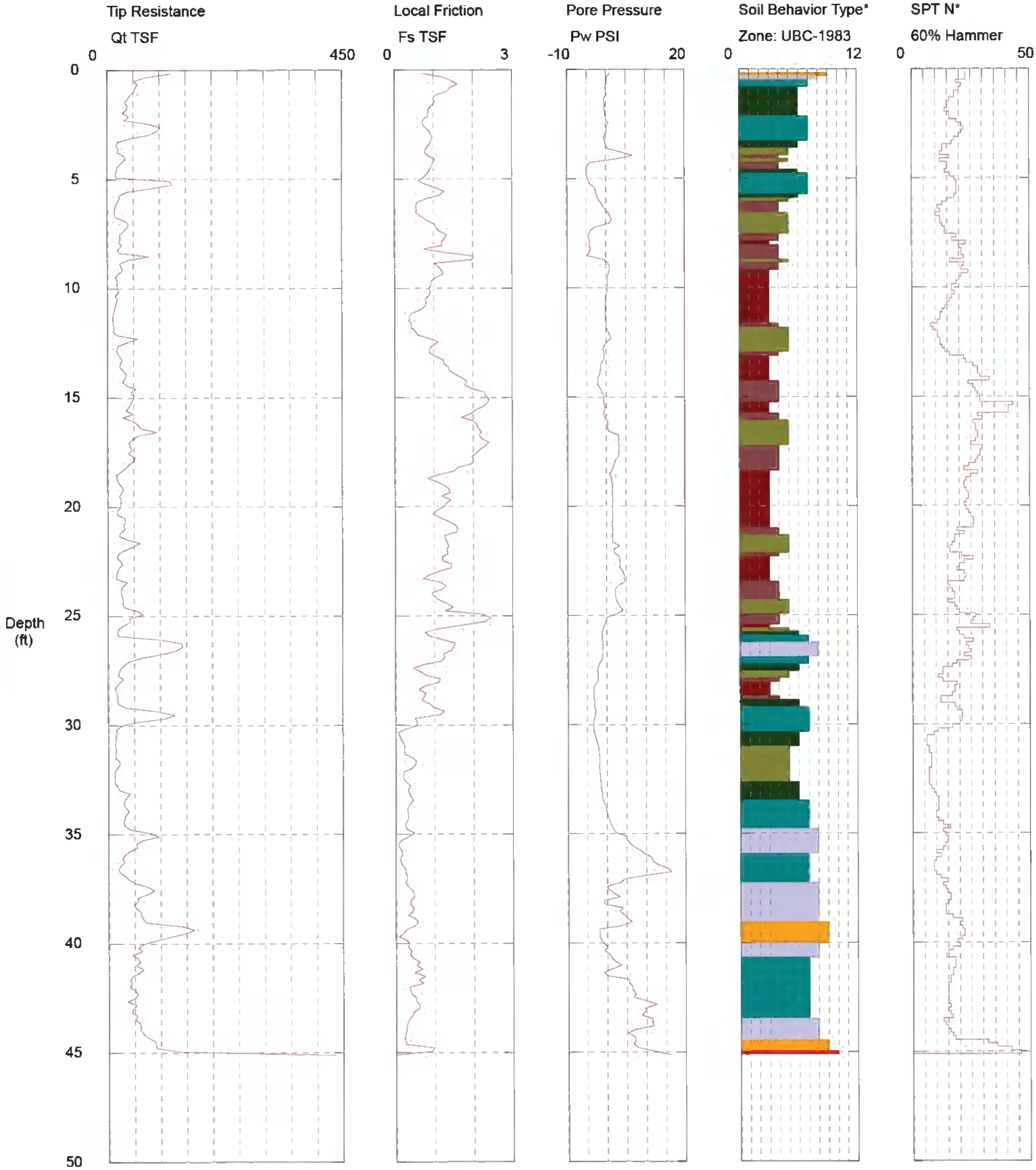
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-26
Cone Used: DSA1123

CPT Date/Time: 2/22/2013 7:52:02 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.11 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 (*)

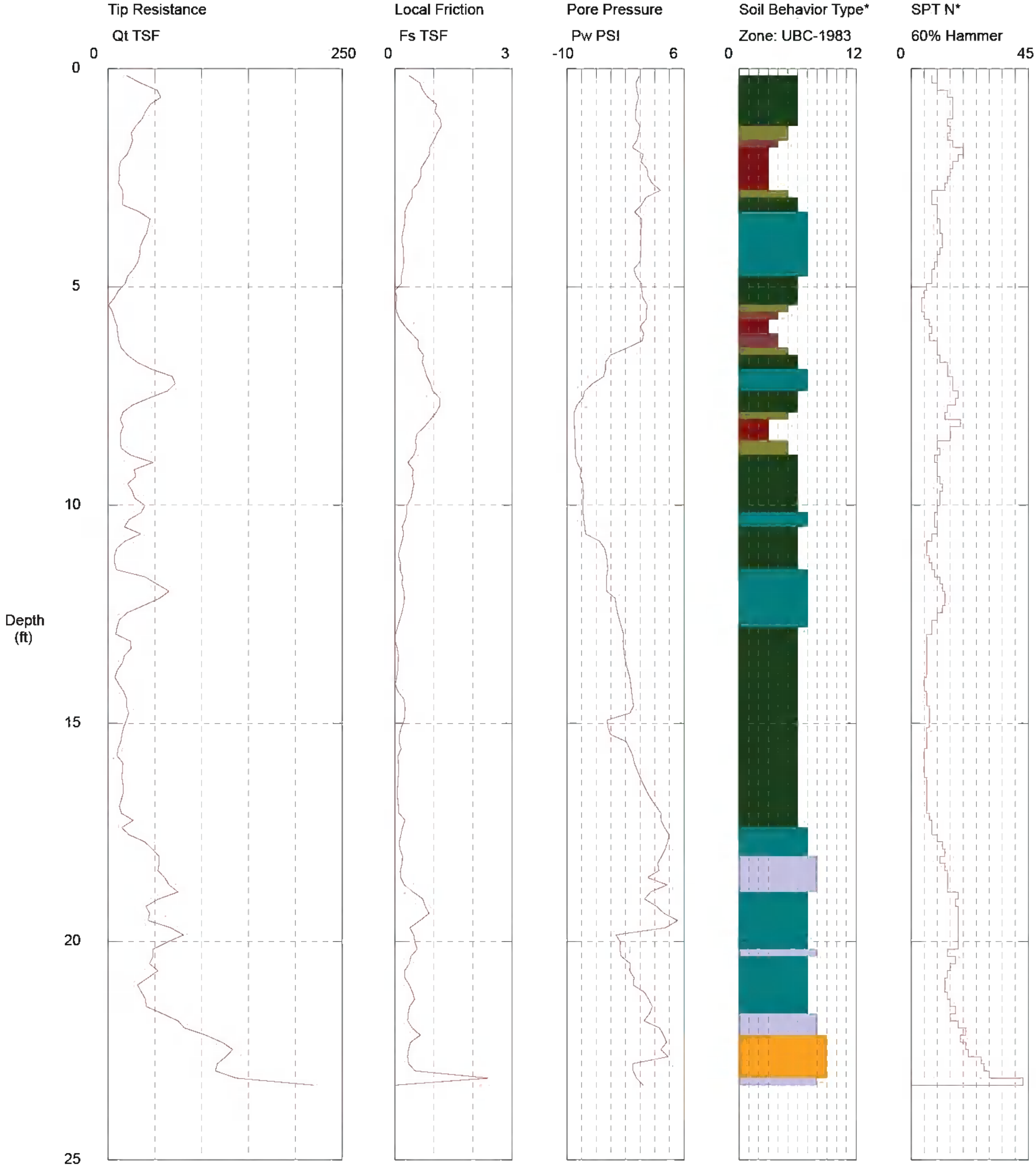
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-27
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 12:43:37 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 23.29 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 (*)

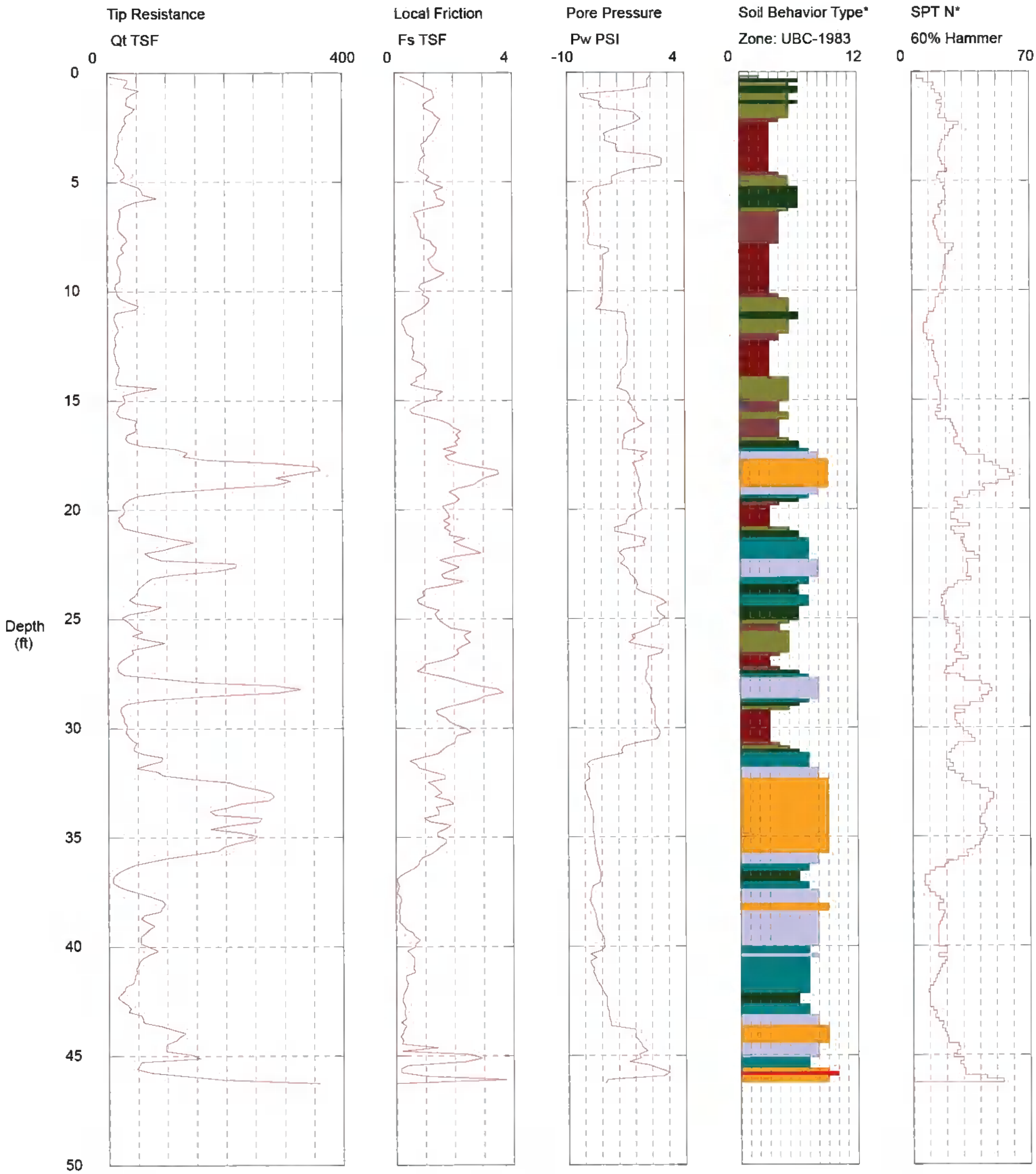
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-28
Cone Used: DSG1156

CPT Date/Time: 3/4/2013 5:27:47 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 46.26 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 (*)

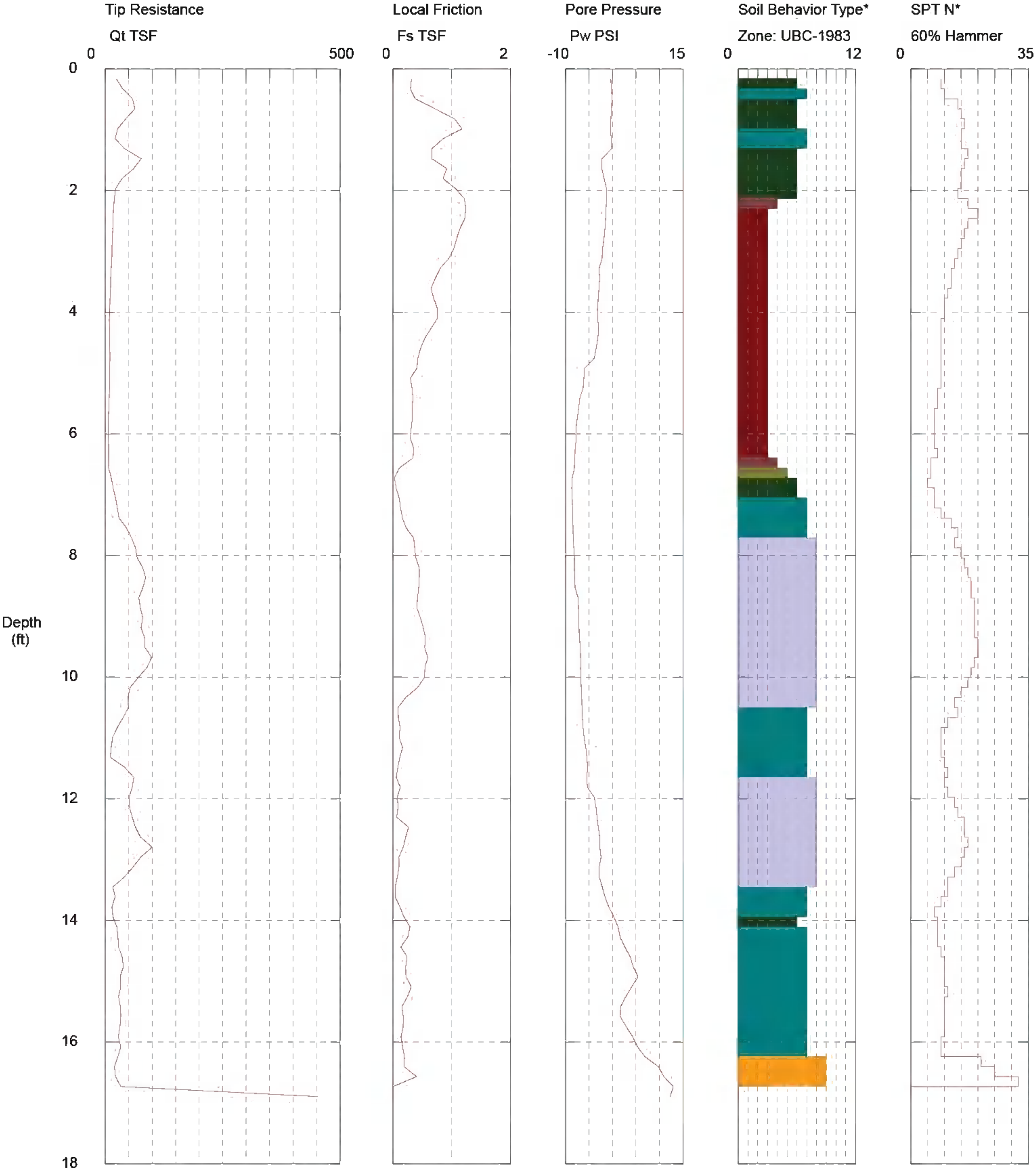
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-29
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 1:05:13 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 16.90 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 (*)

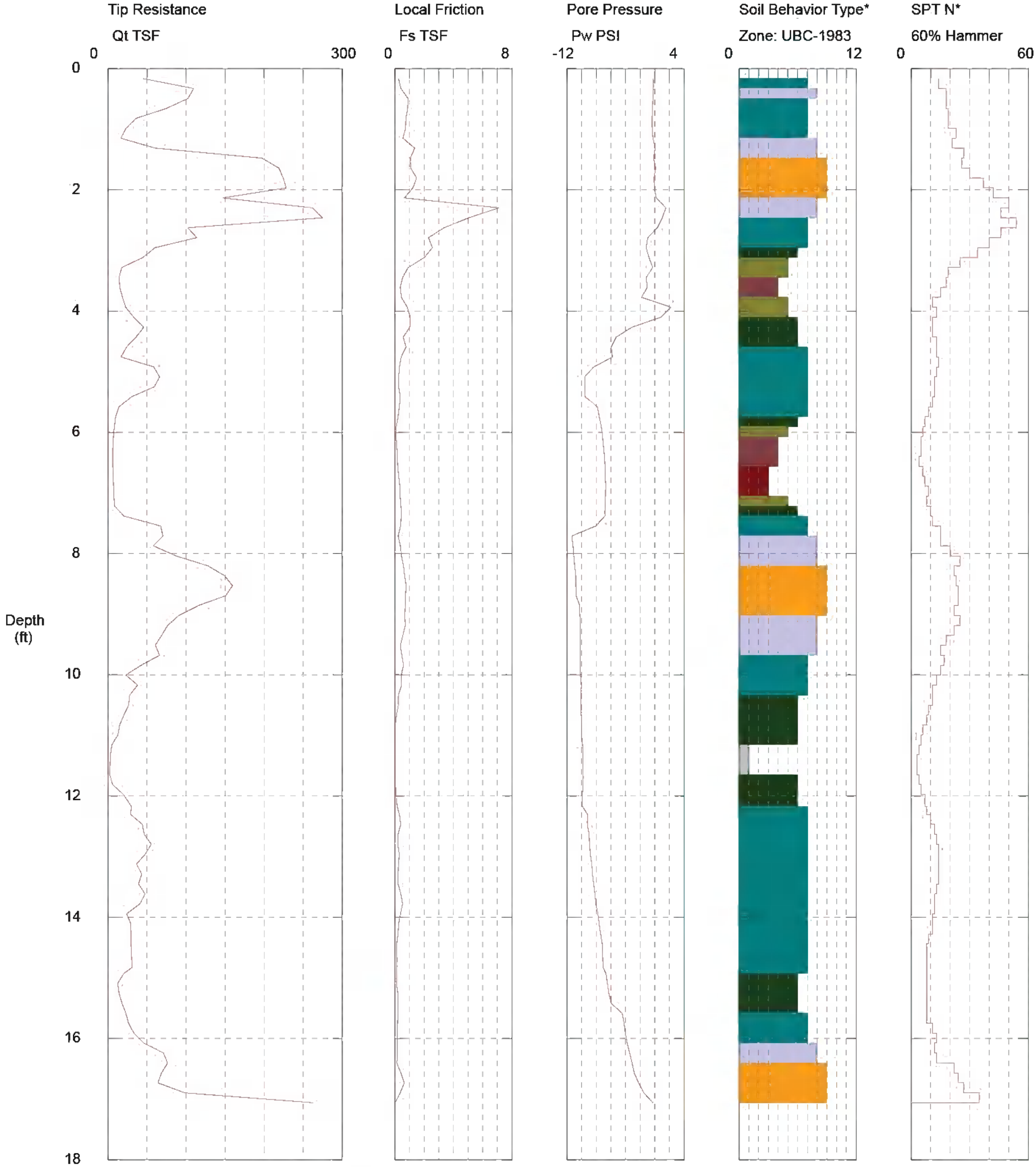
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-30
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 1:23:26 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 17.06 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 (*)

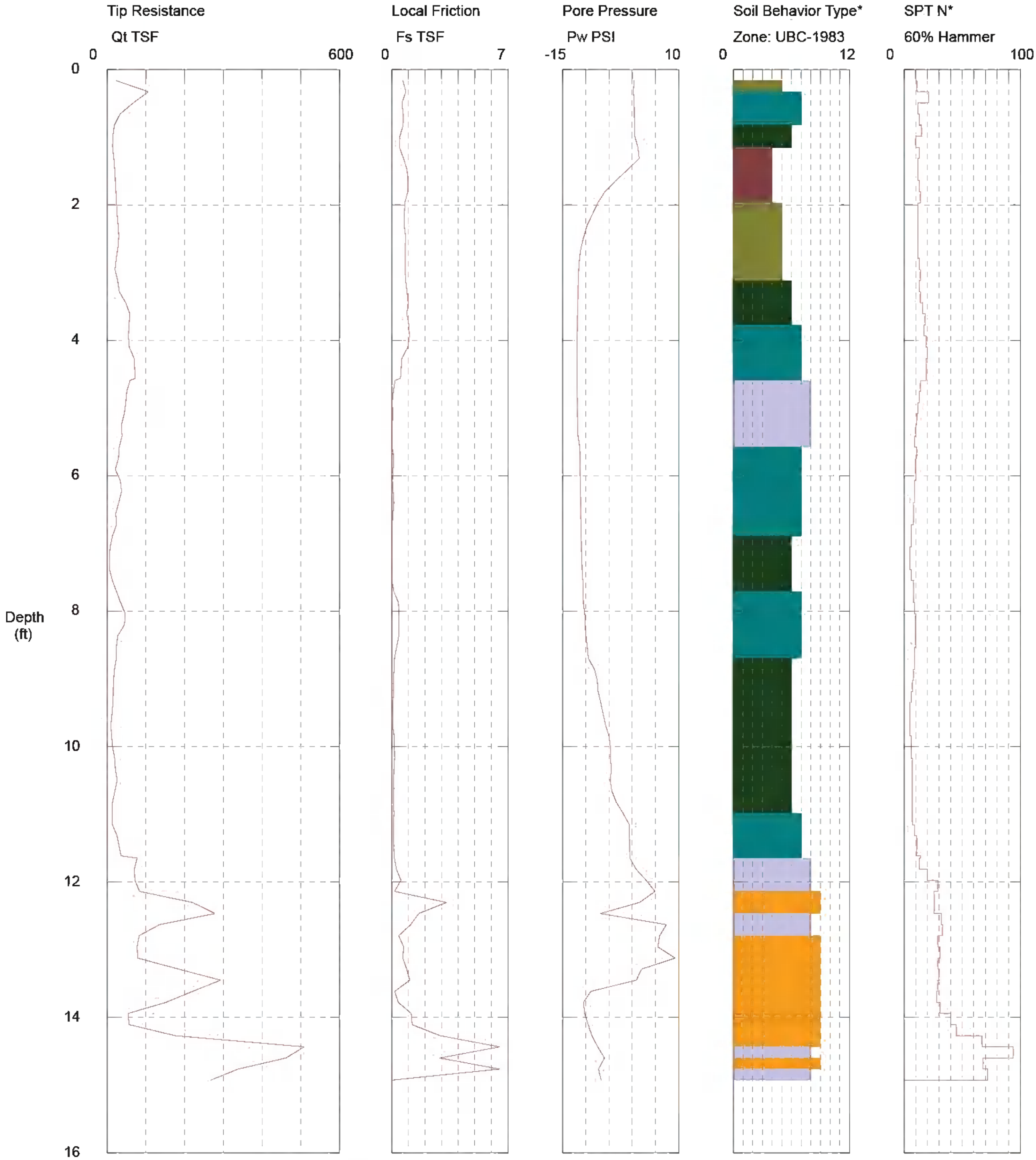
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-31
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 2:32:13 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 14.93 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 (*)

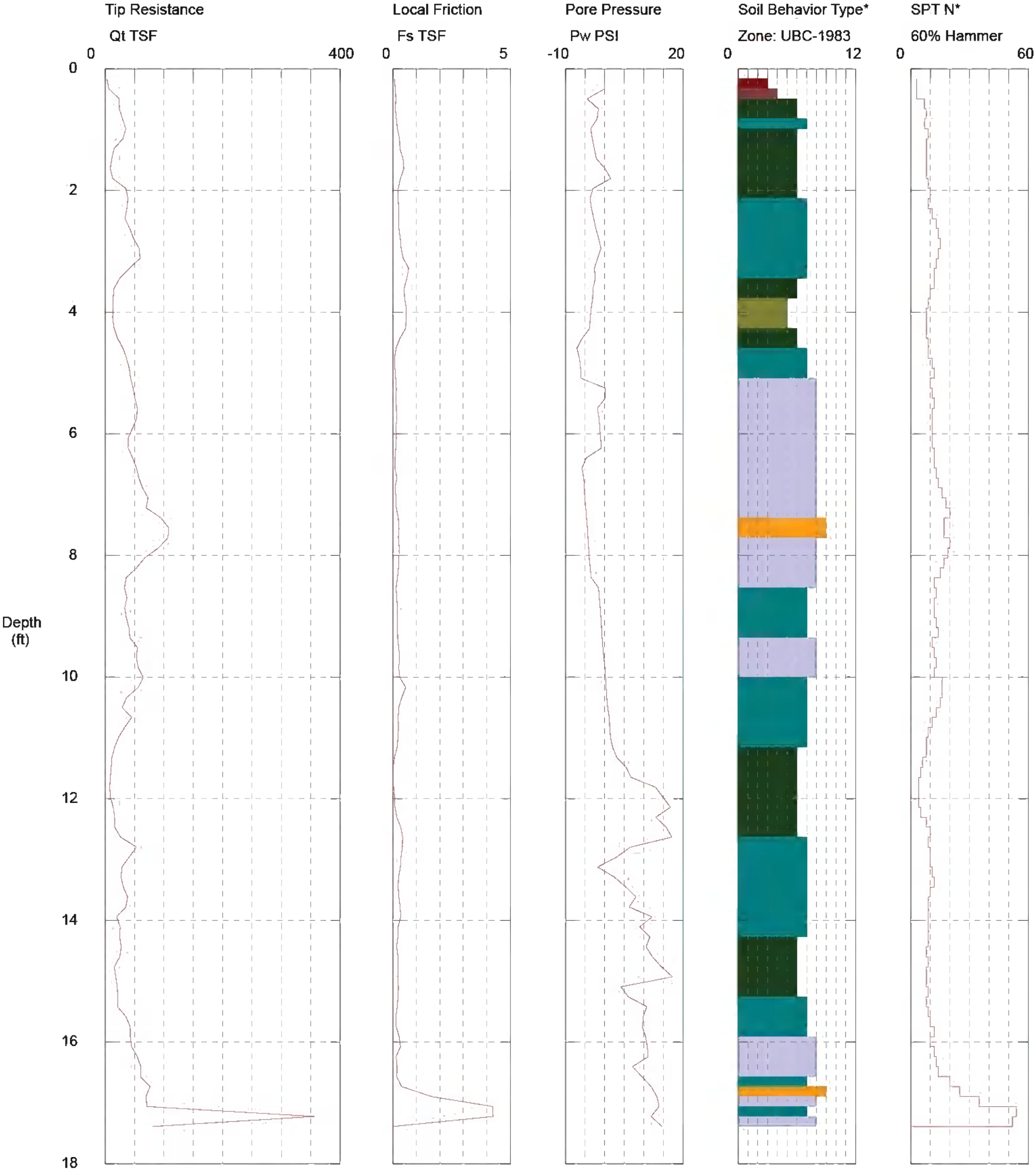
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-32
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 1:59:28 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 17.39 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 (*)

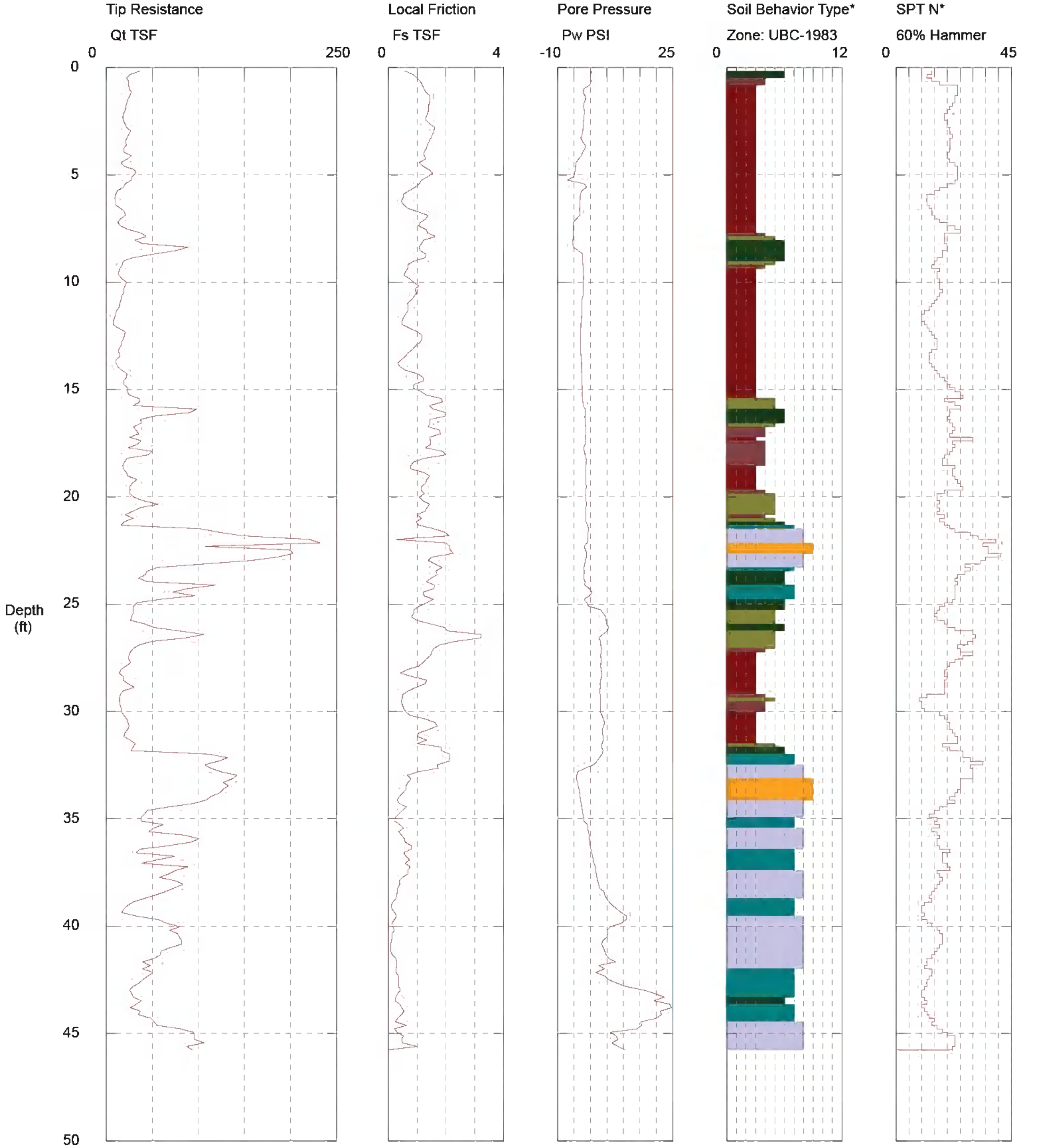
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-33
Cone Used: DSG1156

CPT Date/Time: 2/22/2013 9:15:47 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.77 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 (*)

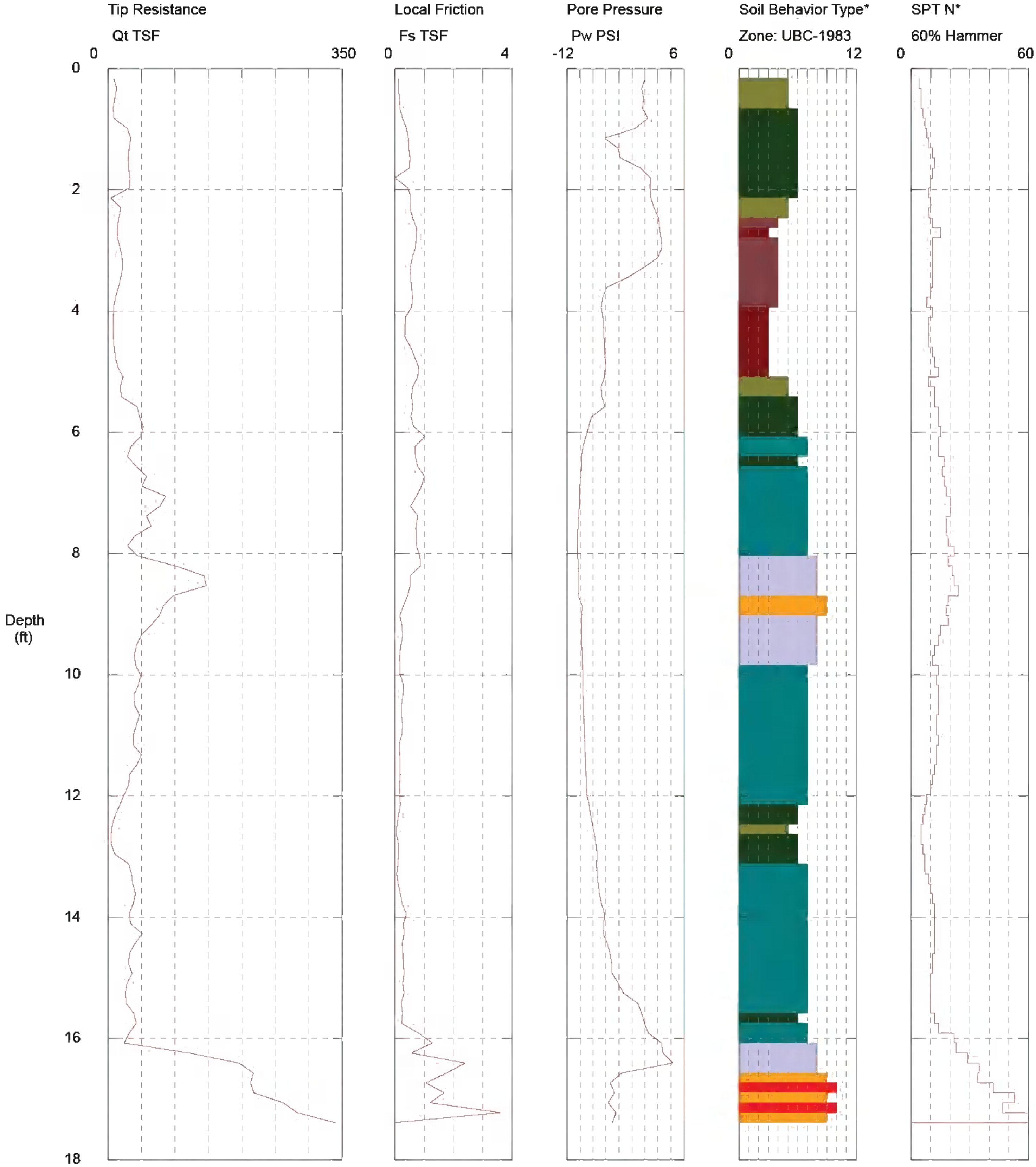
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-34
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 2:56:39 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 17.39 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 (*)

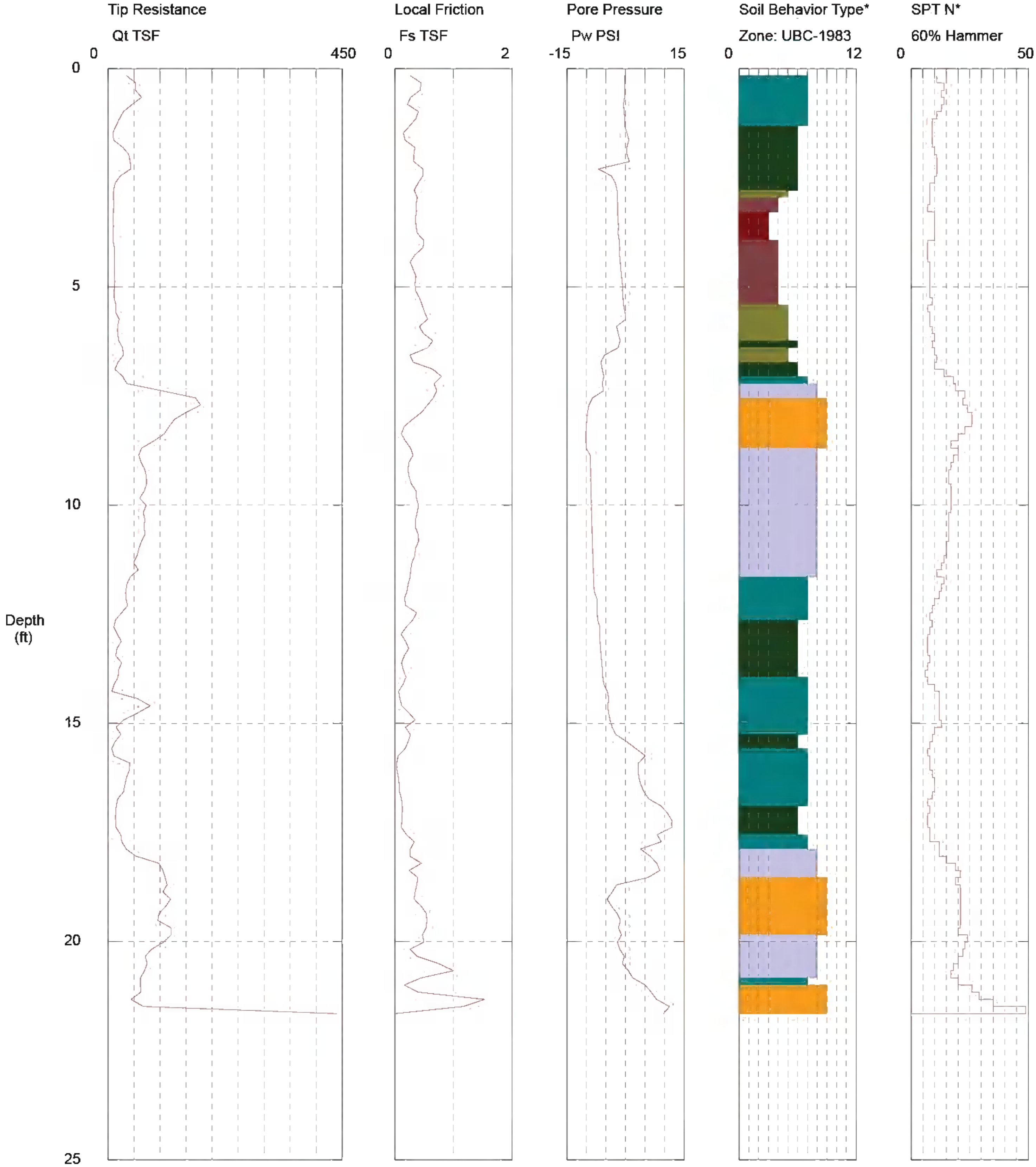
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-35
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 3:15:48 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 21.65 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 (*)

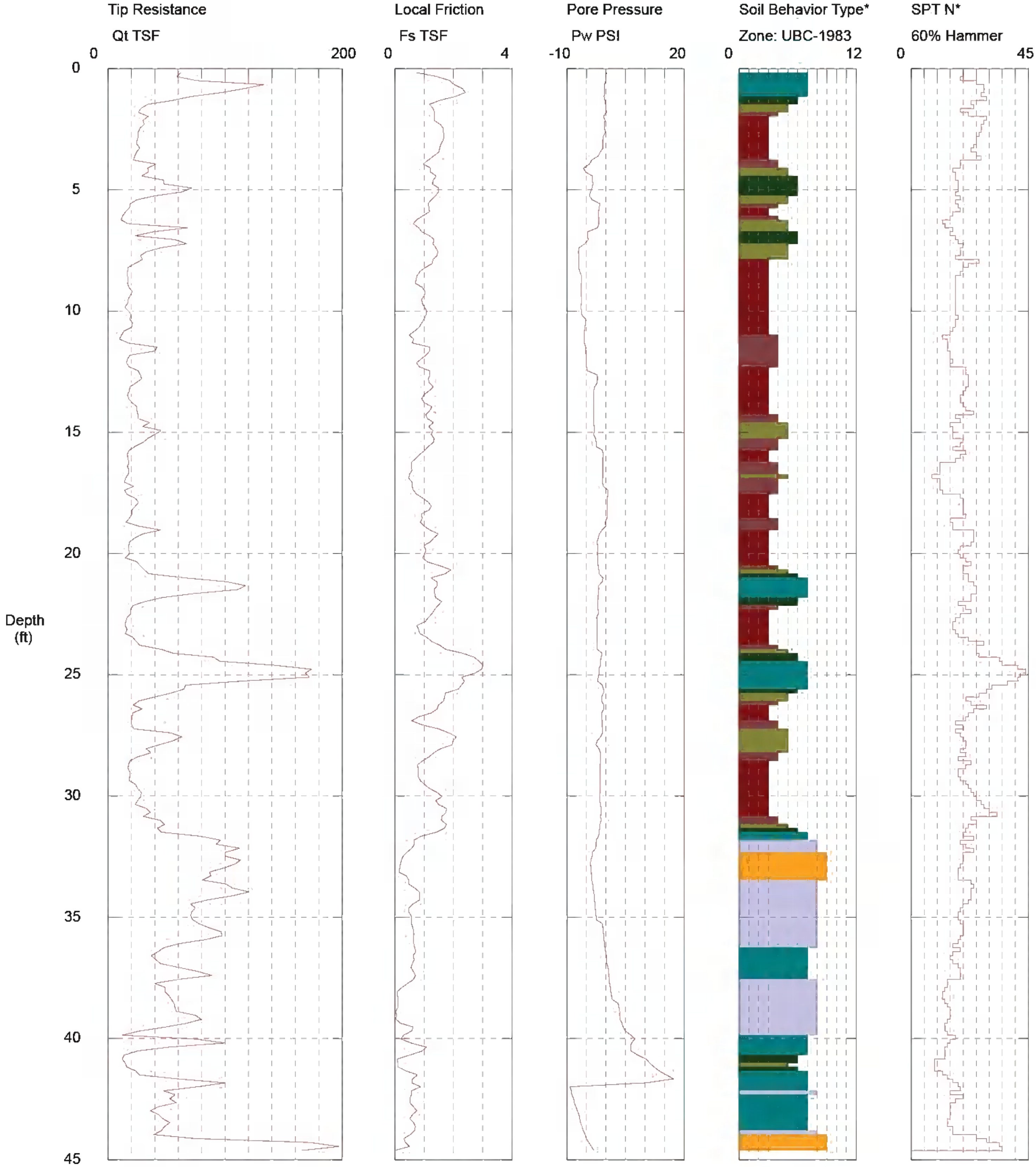
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-36
Cone Used: DSG1156

CPT Date/Time: 2/22/2013 10:20:08 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 44.62 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 (*)

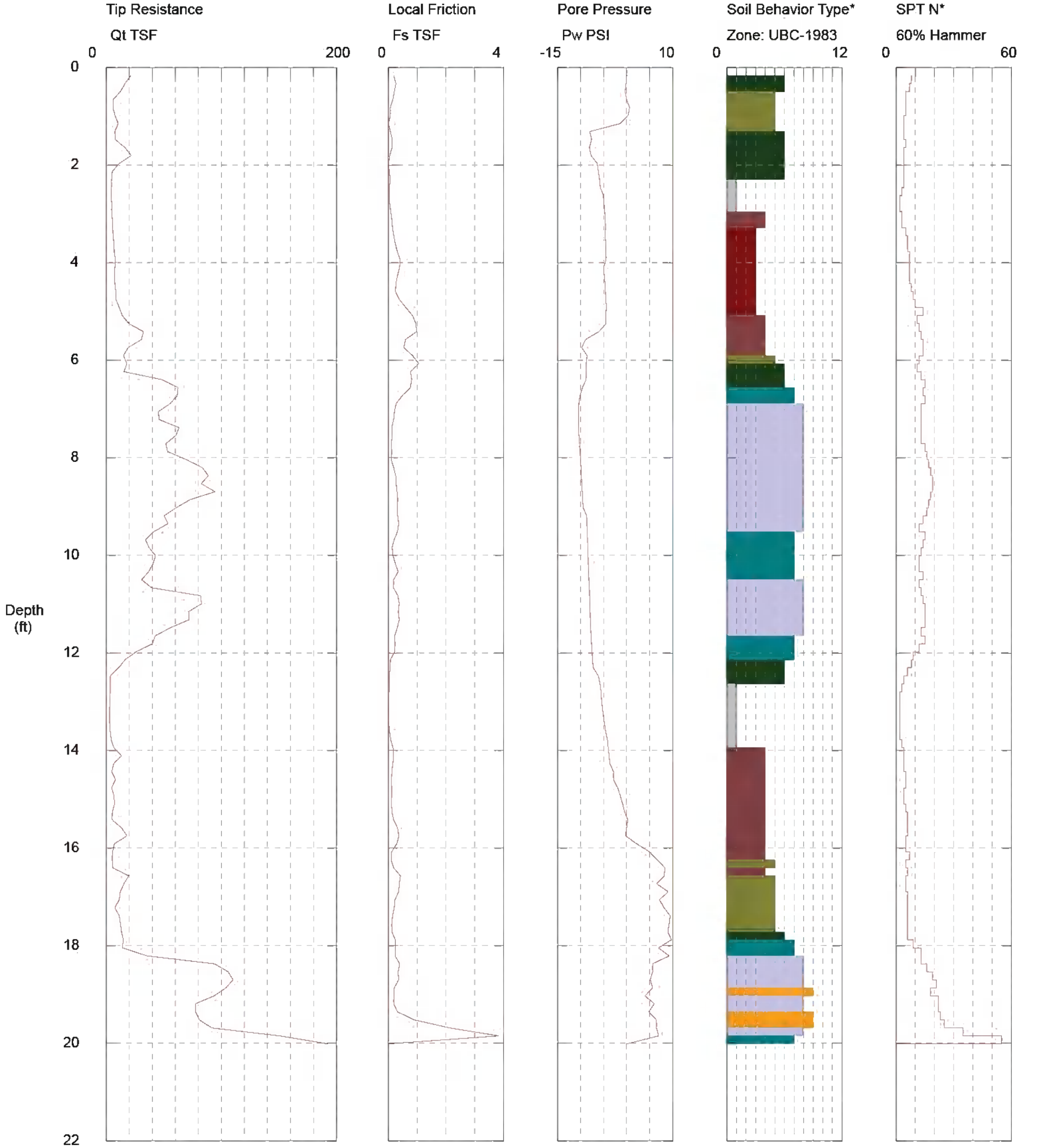
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-37
Cone Used: DSG1156

CPT Date/Time: 3/1/2013 3:39:30 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 20.01 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 (*)

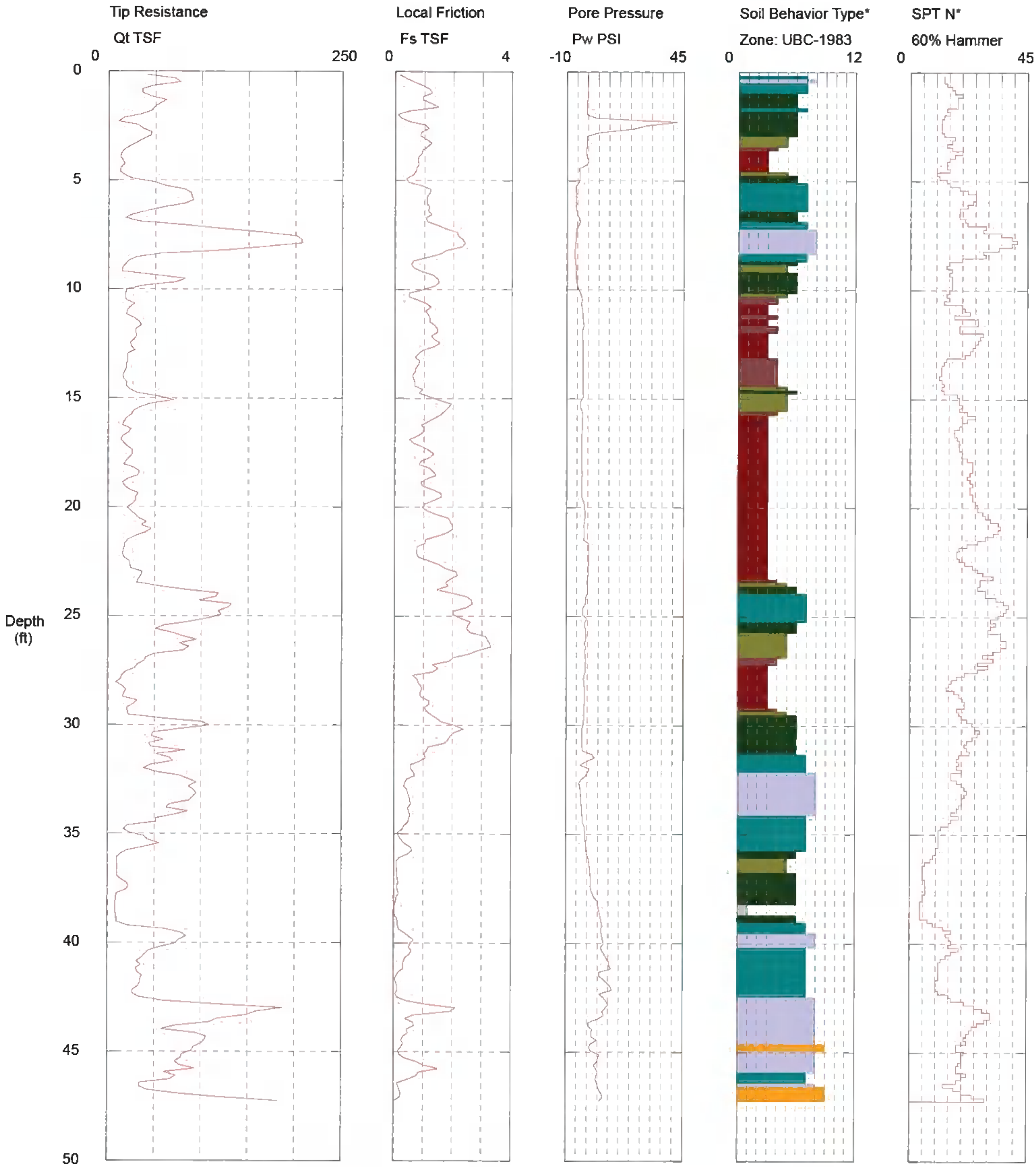
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-38
Cone Used: DSG1156

CPT Date/Time: 2/22/2013 11:19:28 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 47.24 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 (*)

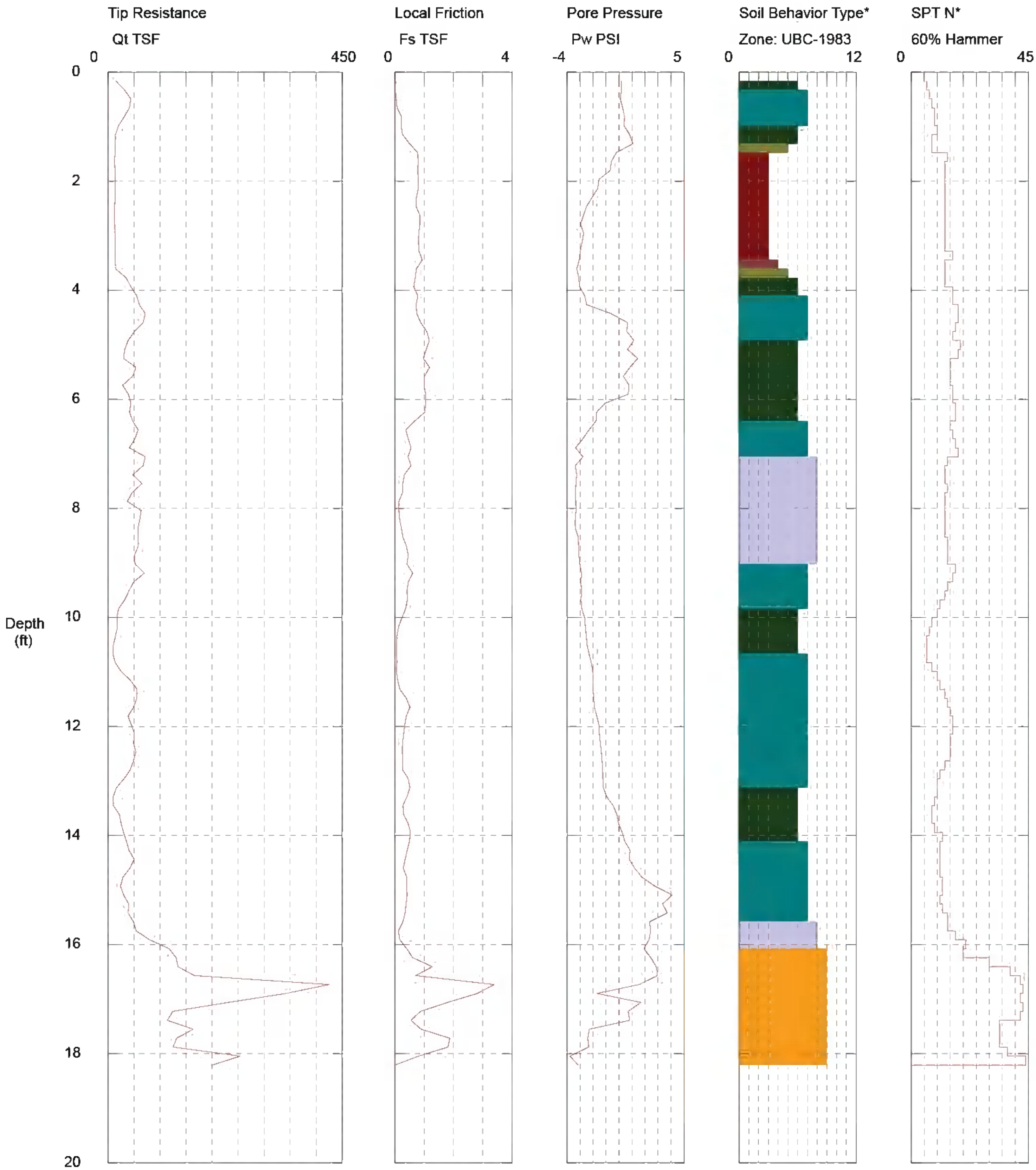
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-39
Cone Used: DSG1156

CPT Date/Time: 3/4/2013 10:40:11 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 18.21 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 (*)

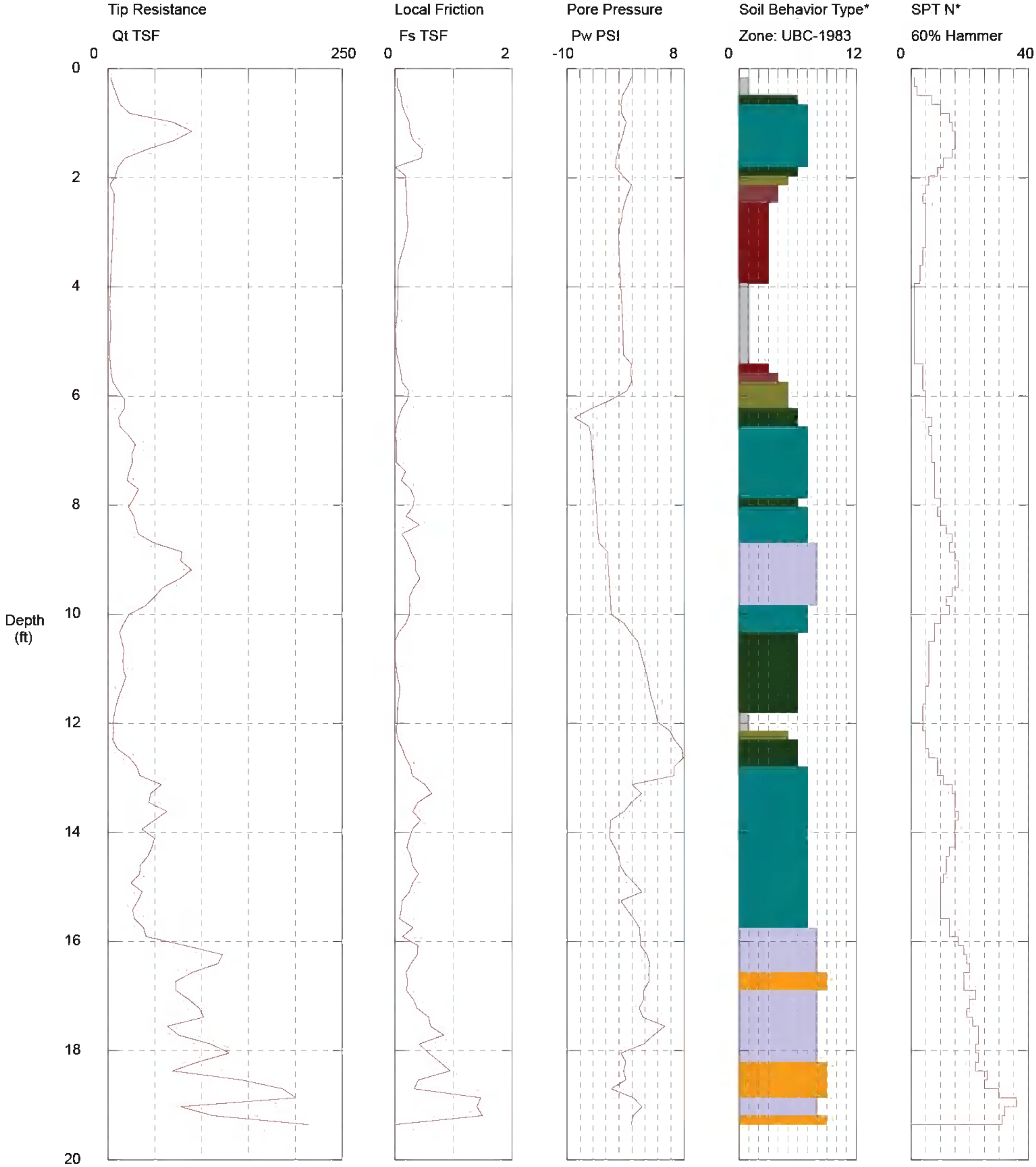
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-40
Cone Used: DSG1156

CPT Date/Time: 3/4/2013 9:53:04 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 19.36 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 (*)

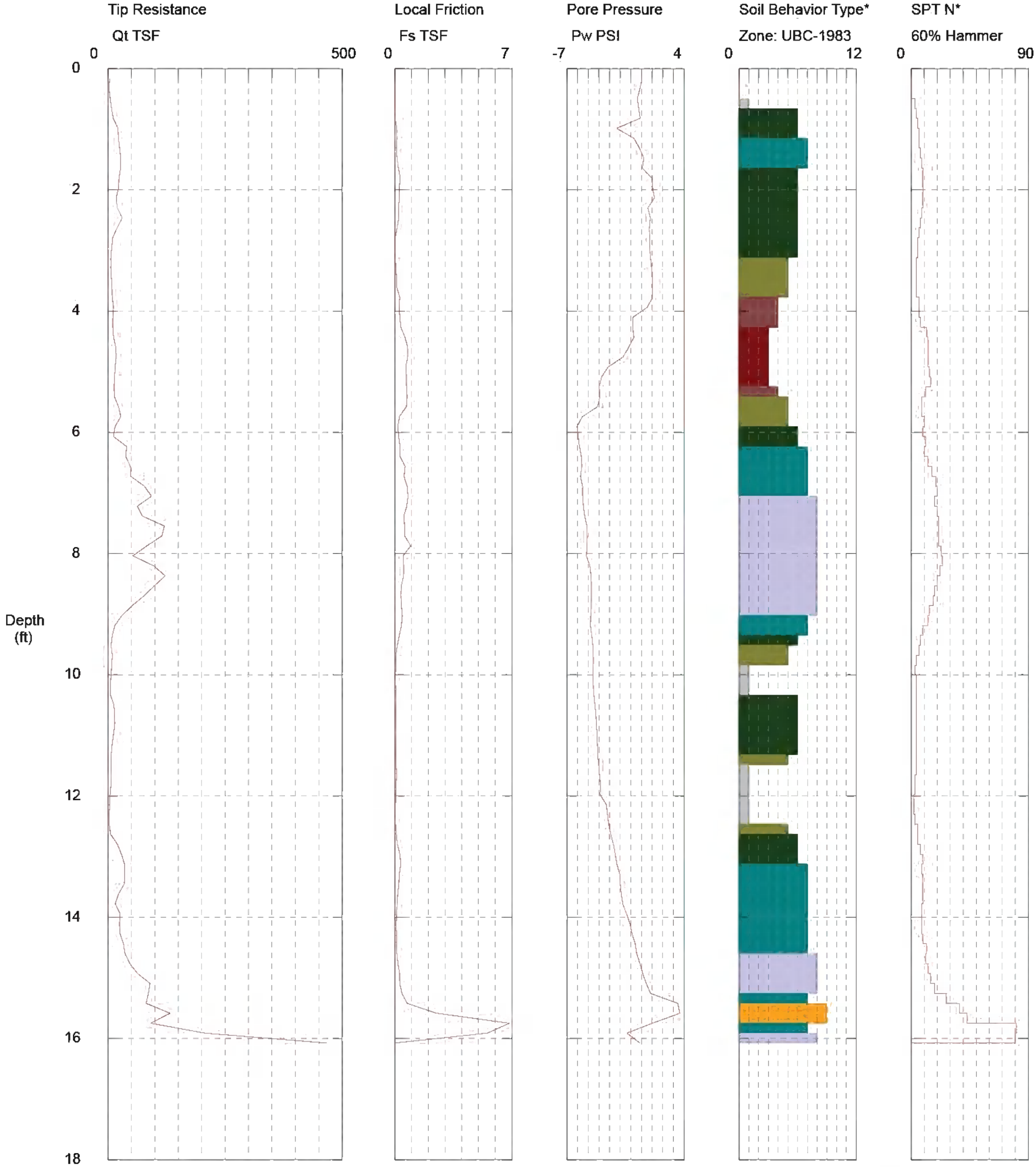
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-41
Cone Used: DSG1156

CPT Date/Time: 3/4/2013 11:07:58 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 16.08 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 (*)

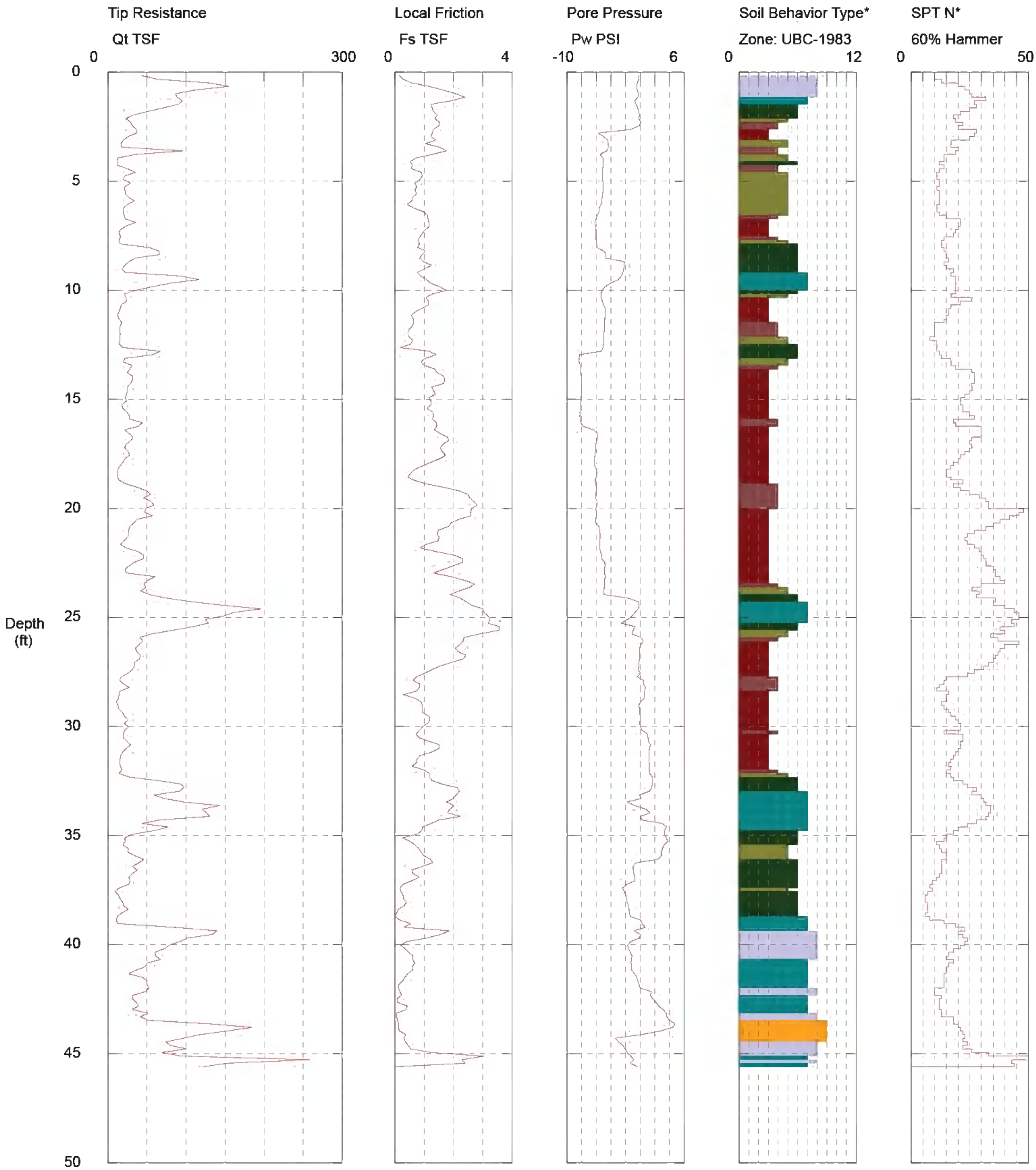
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-42
Cone Used: DSG1156

CPT Date/Time: 2/22/2013 1:53:24 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 45.60 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 (*)

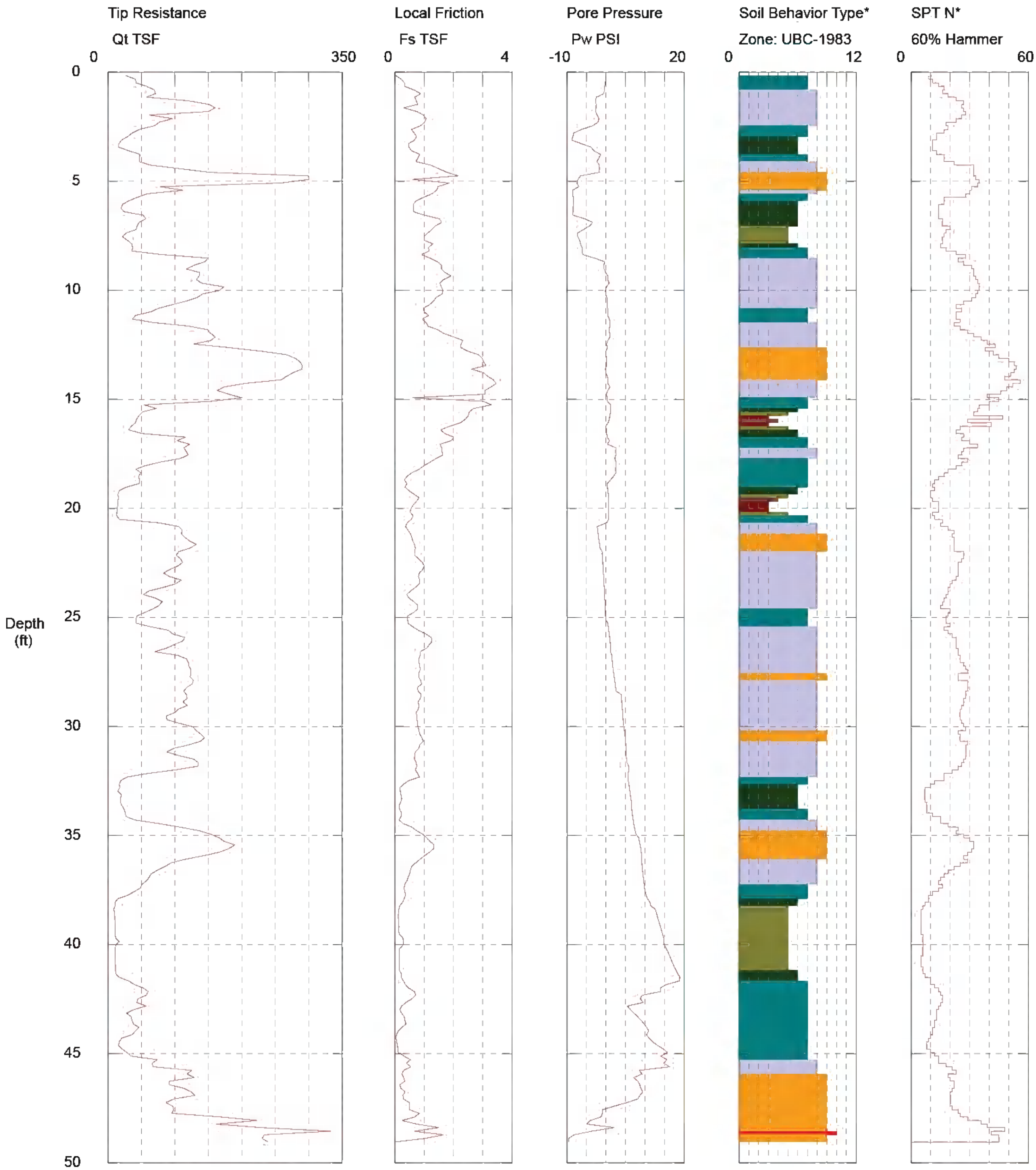
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-62
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 5:05:19 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 49.05 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 (*)

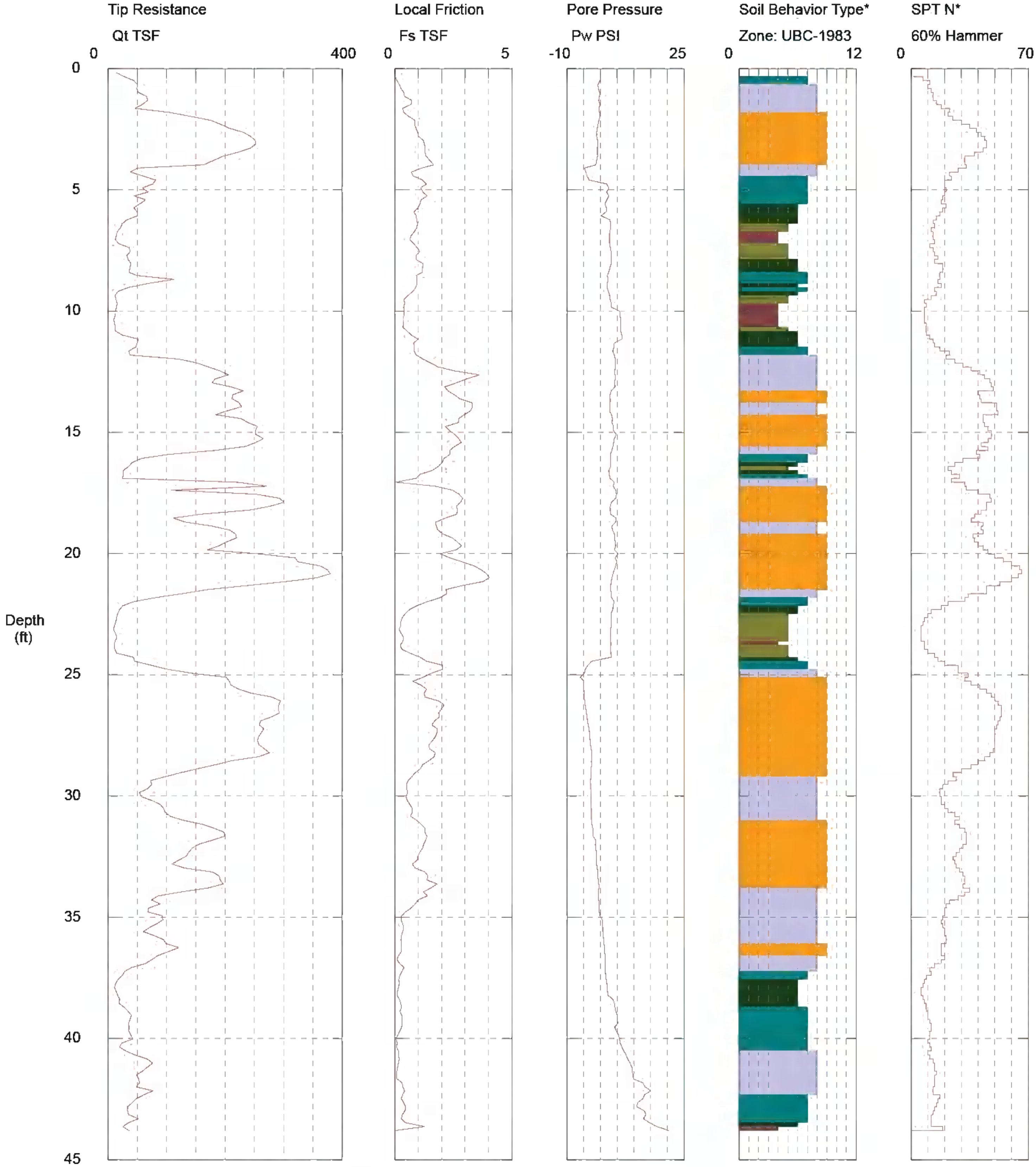
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-63
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 4:20:25 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 43.80 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 (*)

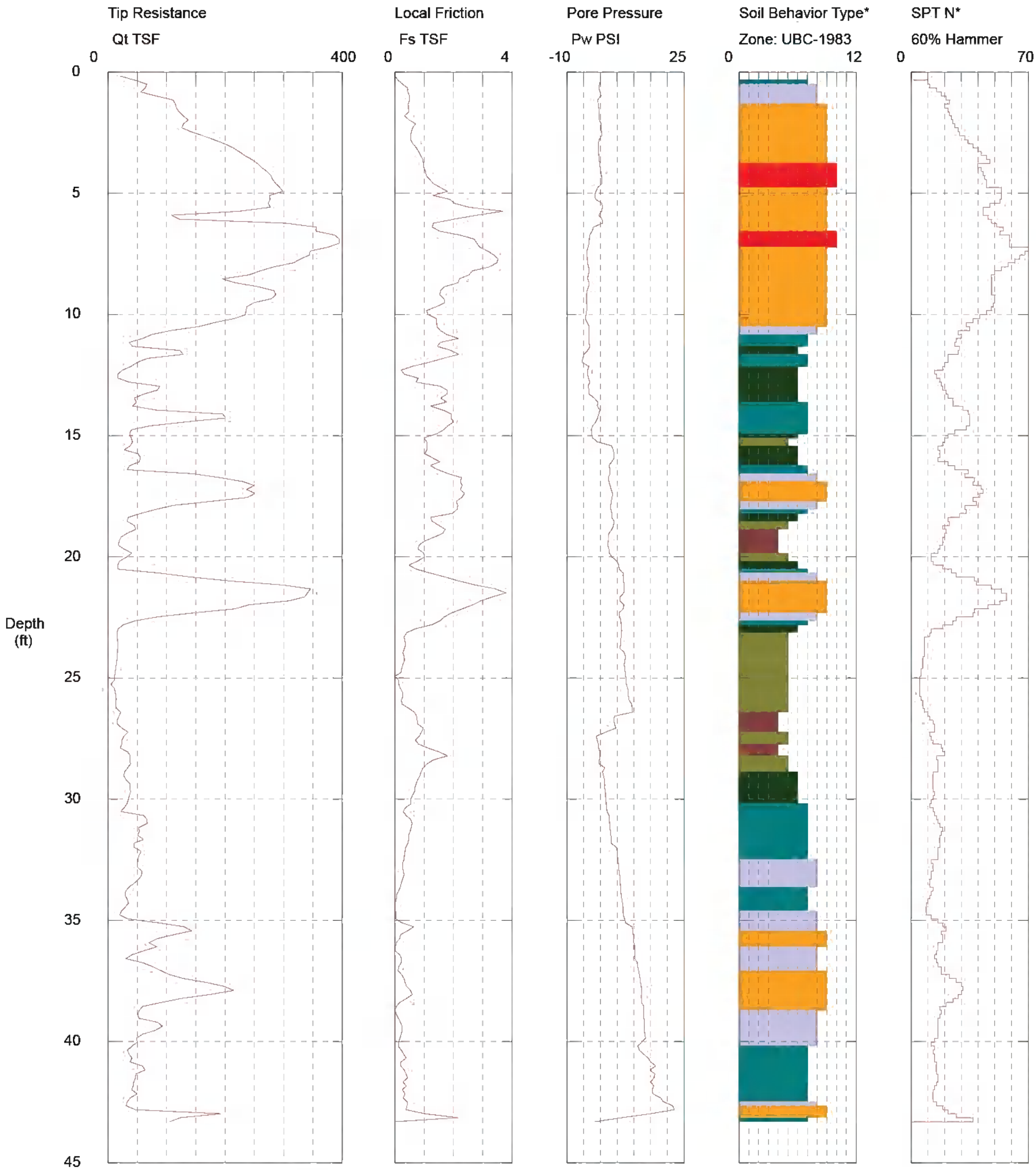
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-64
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 3:34:41 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 43.31 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 (*)

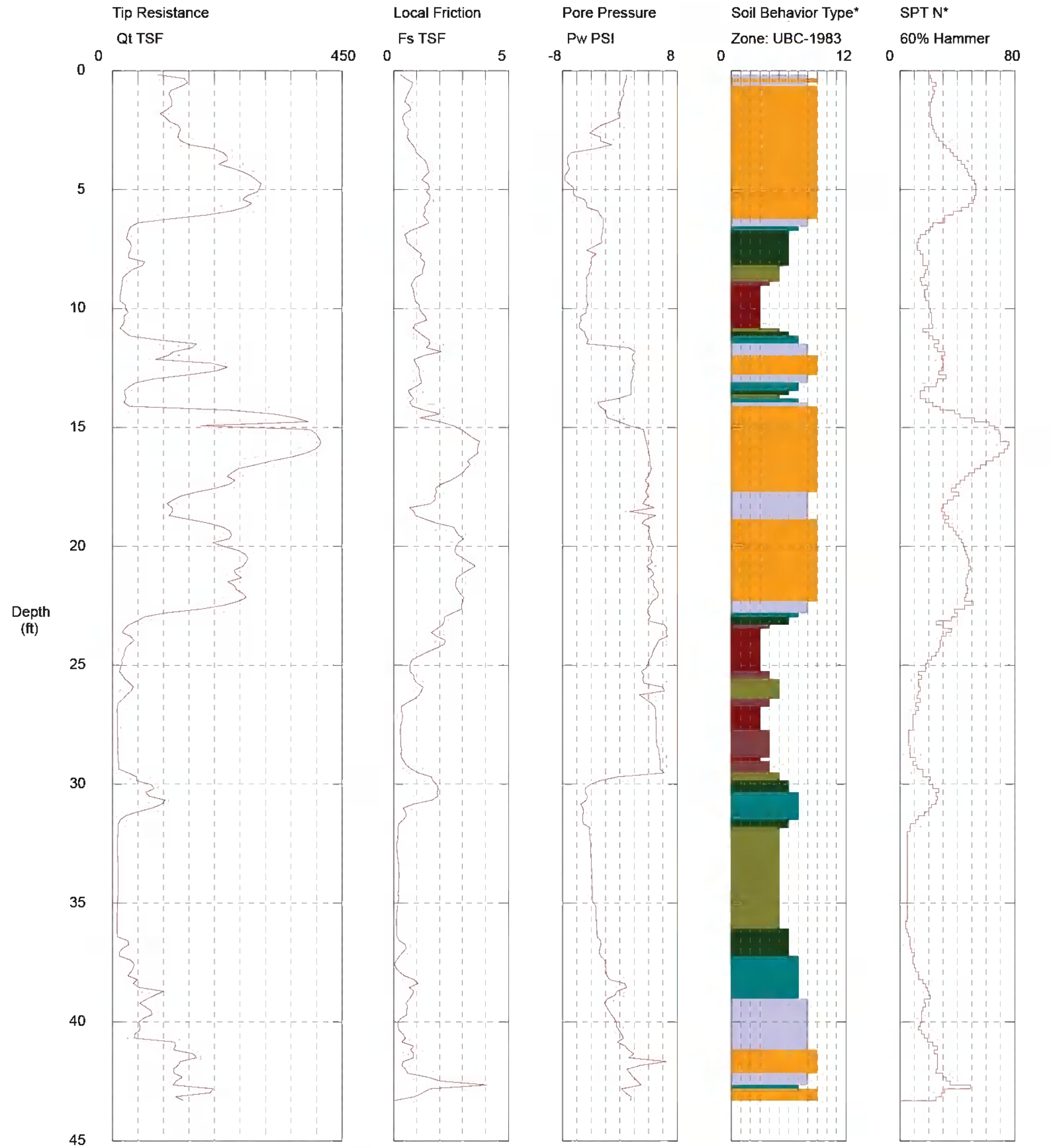
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-65
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 9:58:40 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 43.31 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 (*)

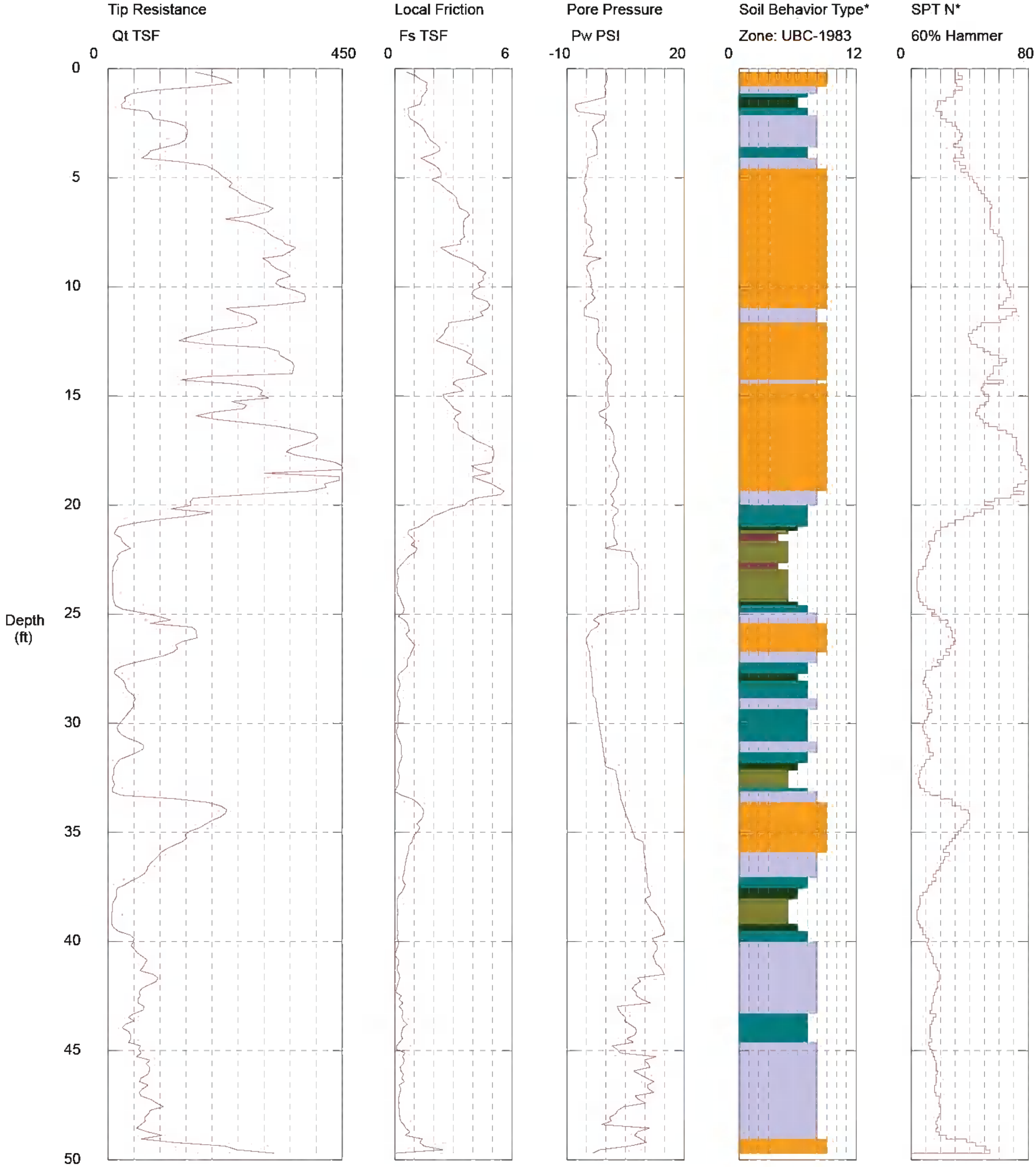
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-69
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 8:59:24 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 49.70 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 (*)

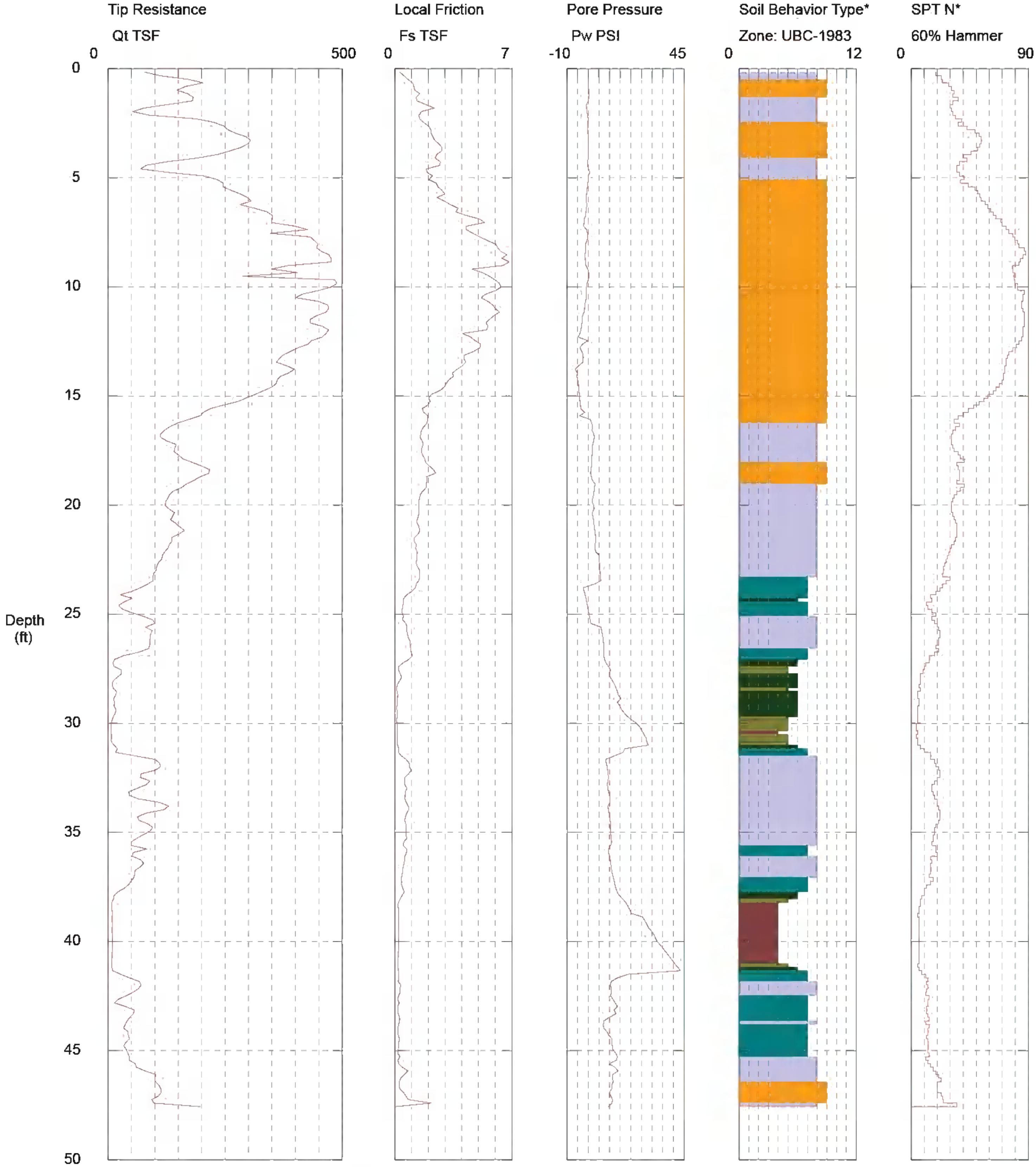
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPT-70
Cone Used: DSG1156

CPT Date/Time: 2/27/2013 7:52:59 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Maximum Depth = 47.57 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 (*)

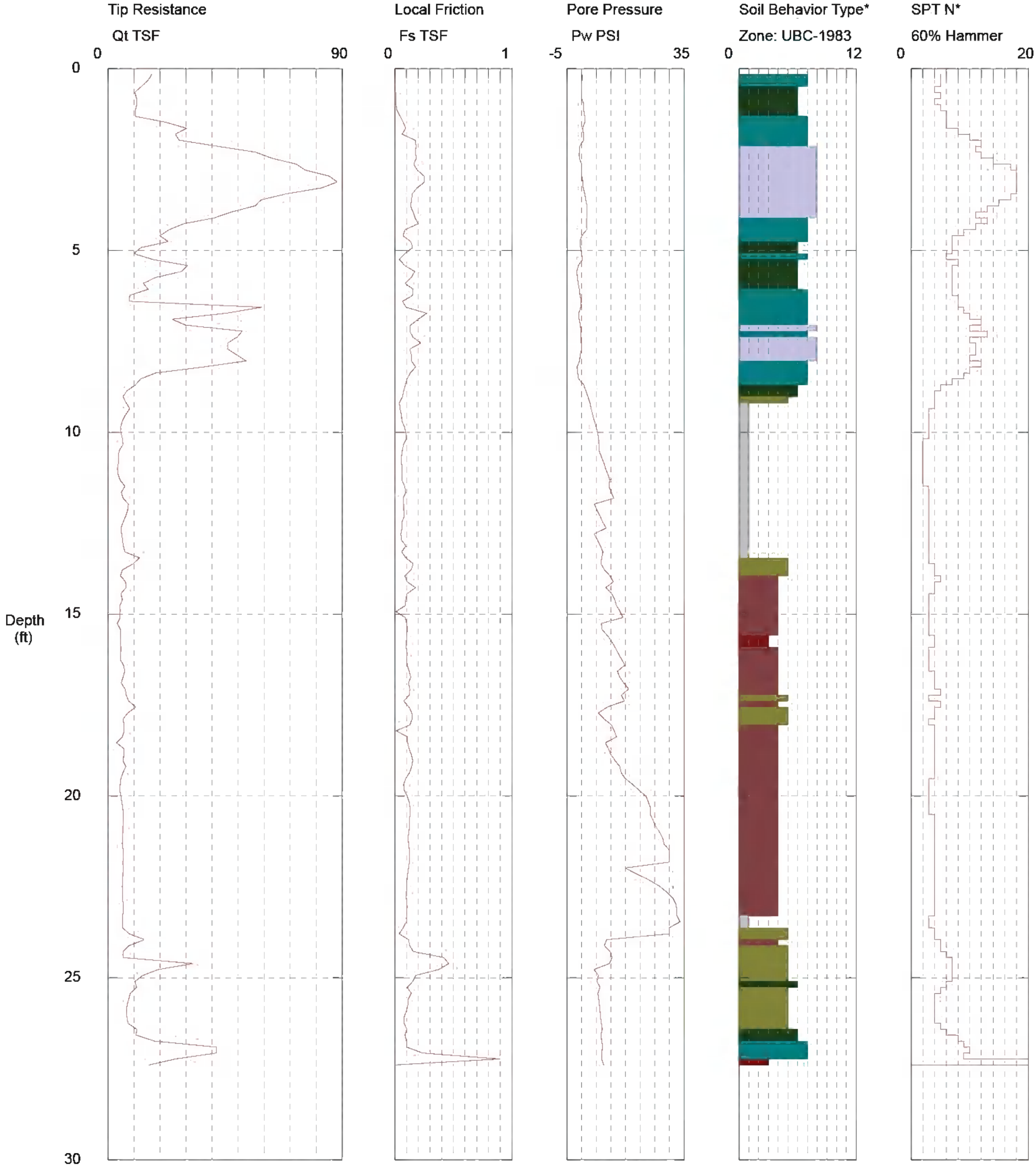
Geosyntec Consultants

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPTO-86
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 12:15:17 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 27.40 feet

Depth Increment = 0.164 feet

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

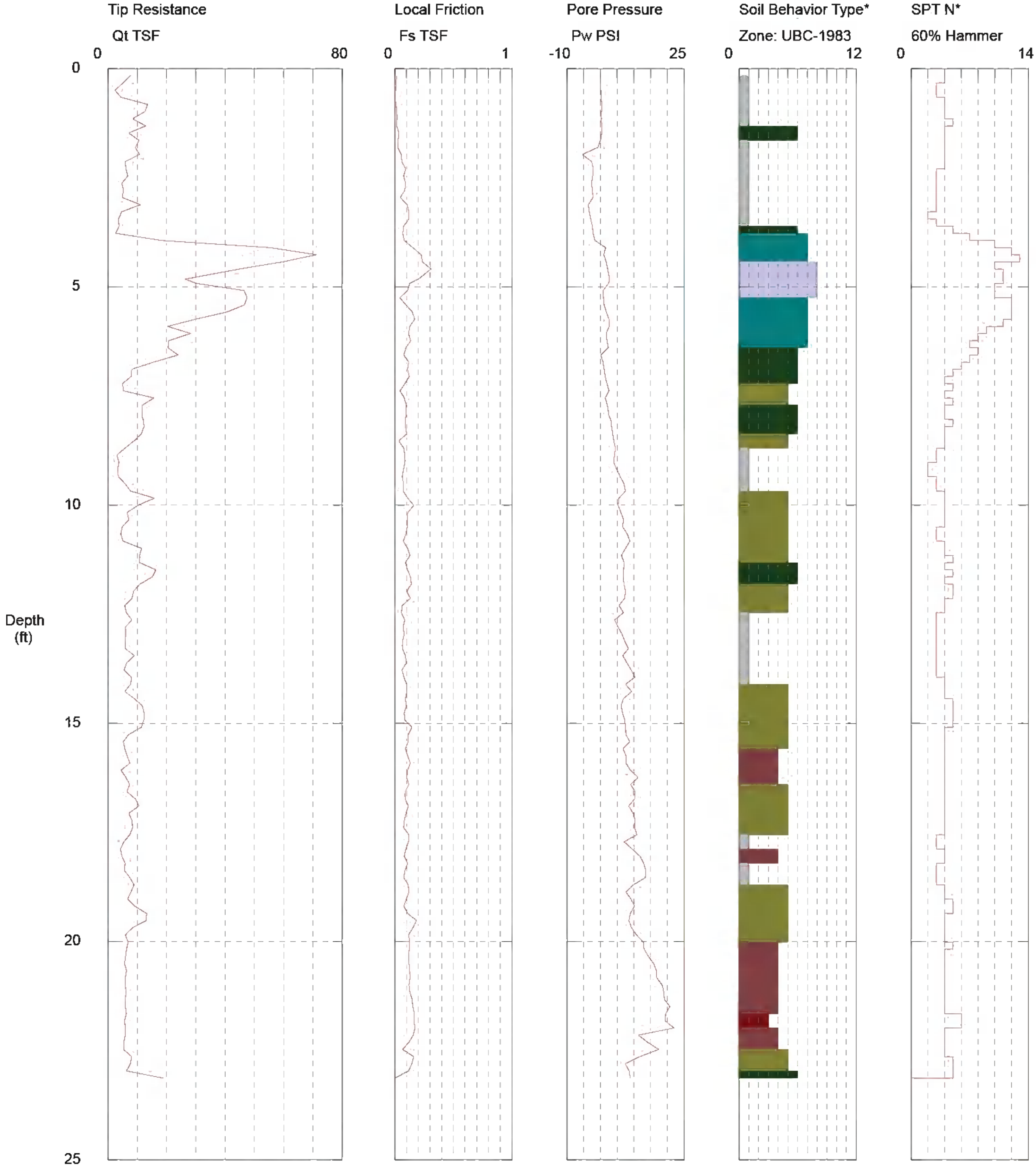
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPTO-87
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 11:36:32 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 23.13 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 (*)

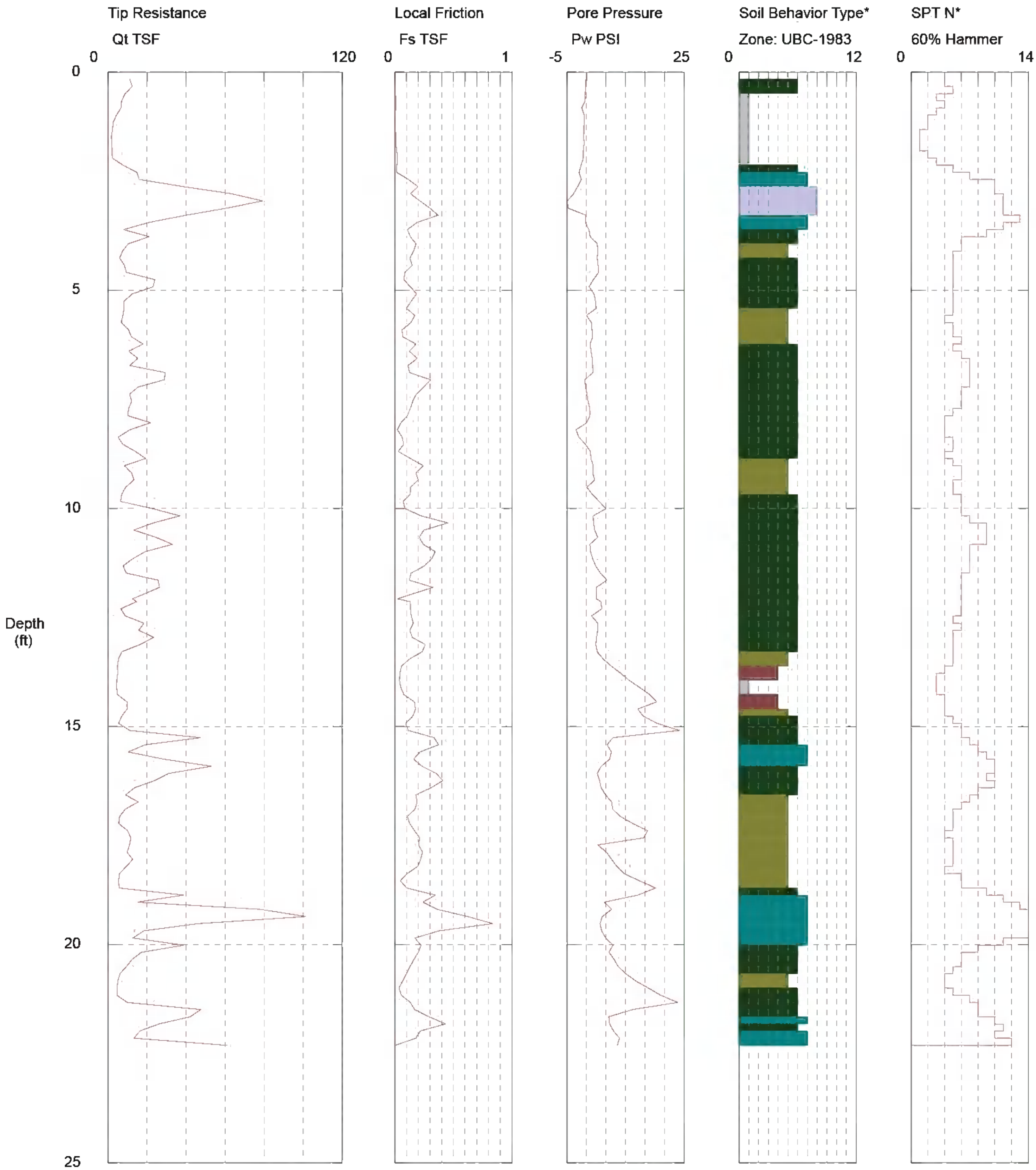
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPTO-88
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 9:06:33 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 22.31 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 (*)

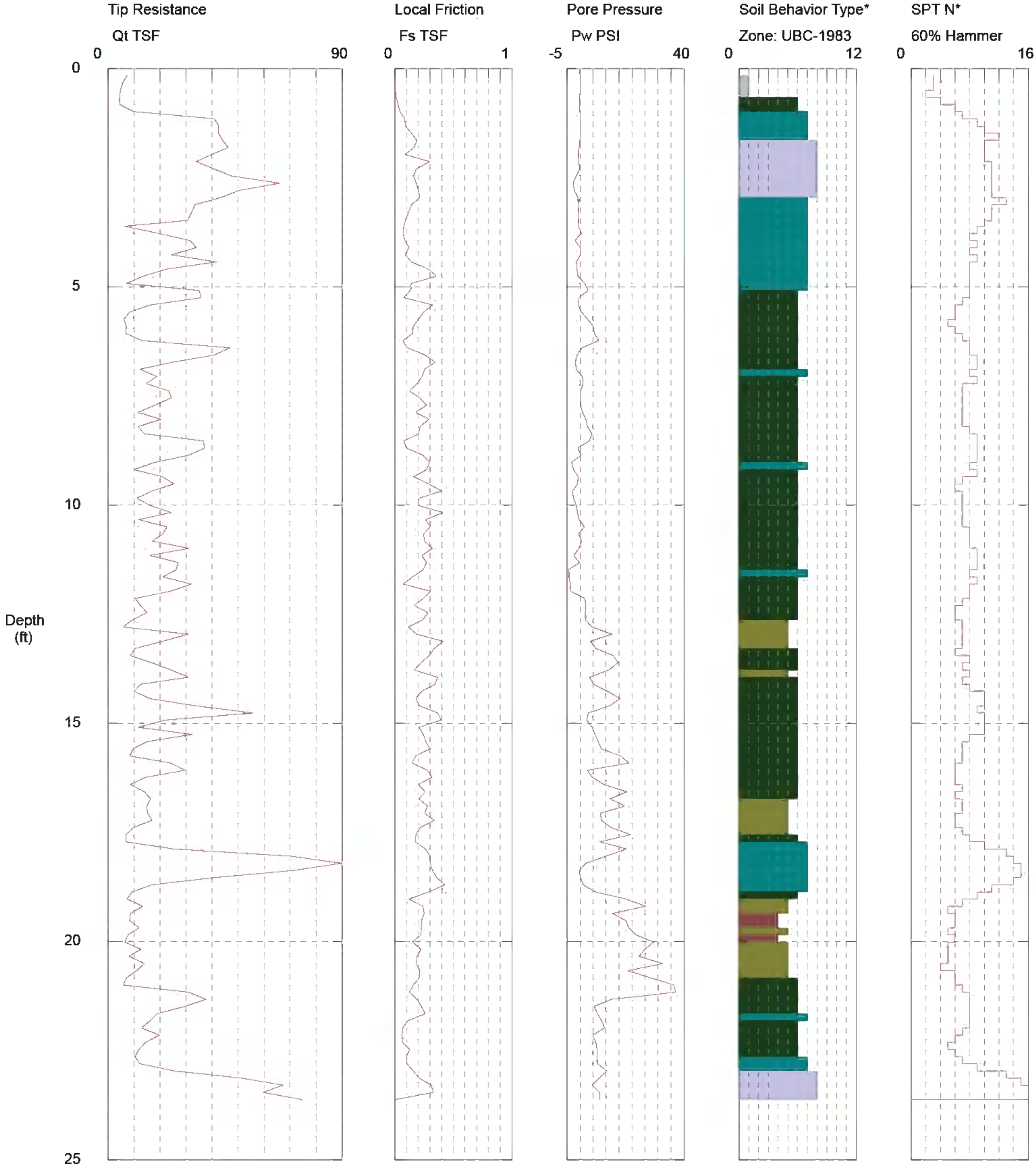
Geosyntec

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

Geosyntec Consultants

Operator: Cory Robison
Sounding: CPTO-89
Cone Used: DSG1156

CPT Date/Time: 2/28/2013 8:22:17 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Maximum Depth = 23.62 feet

Depth Increment = 0.164 feet

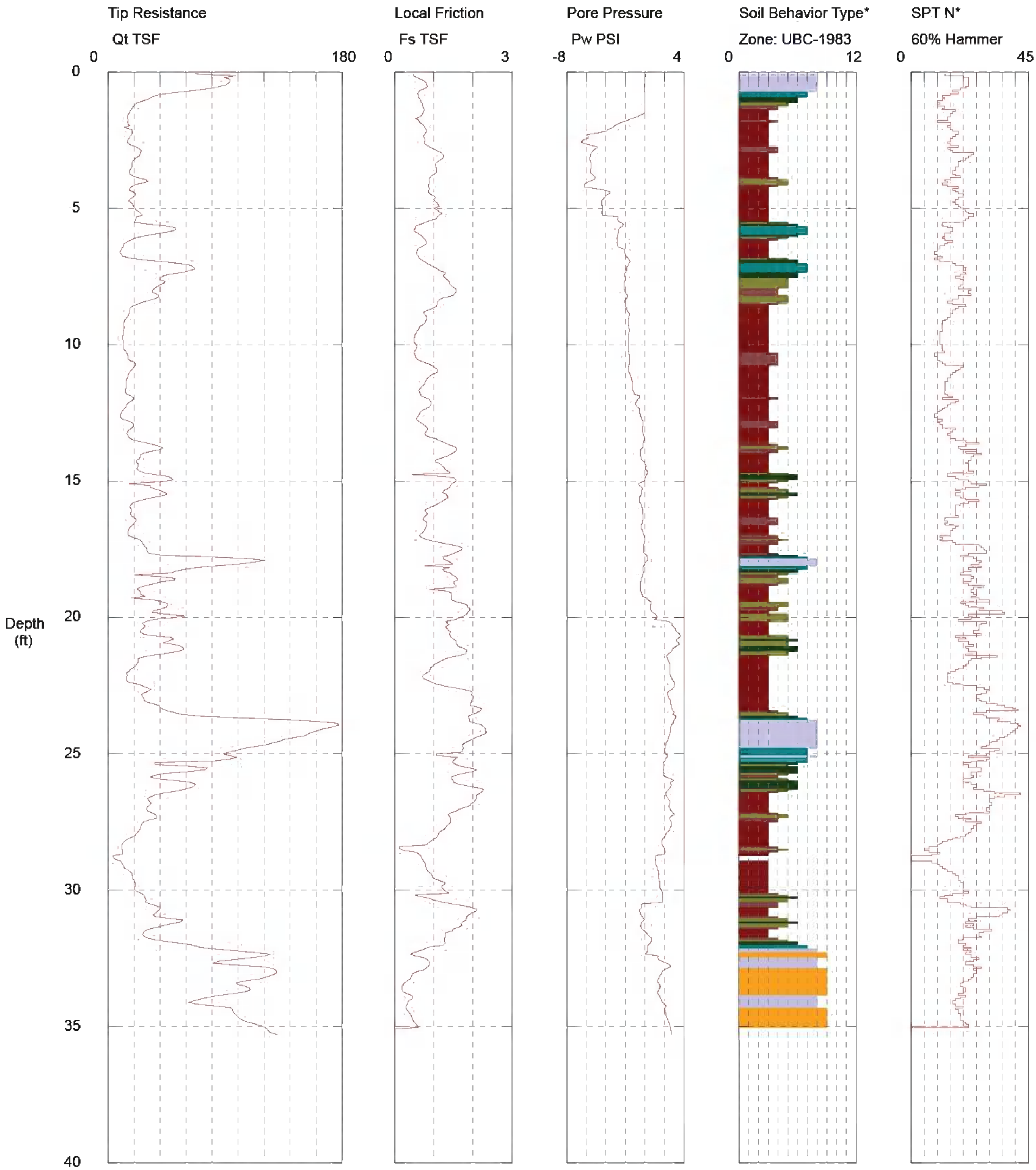
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

Geosyntec

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

Operator: Cory Robison
Sounding: CPT-116
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 3:36:37 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 35.30 feet

Depth Increment = 0.066 feet

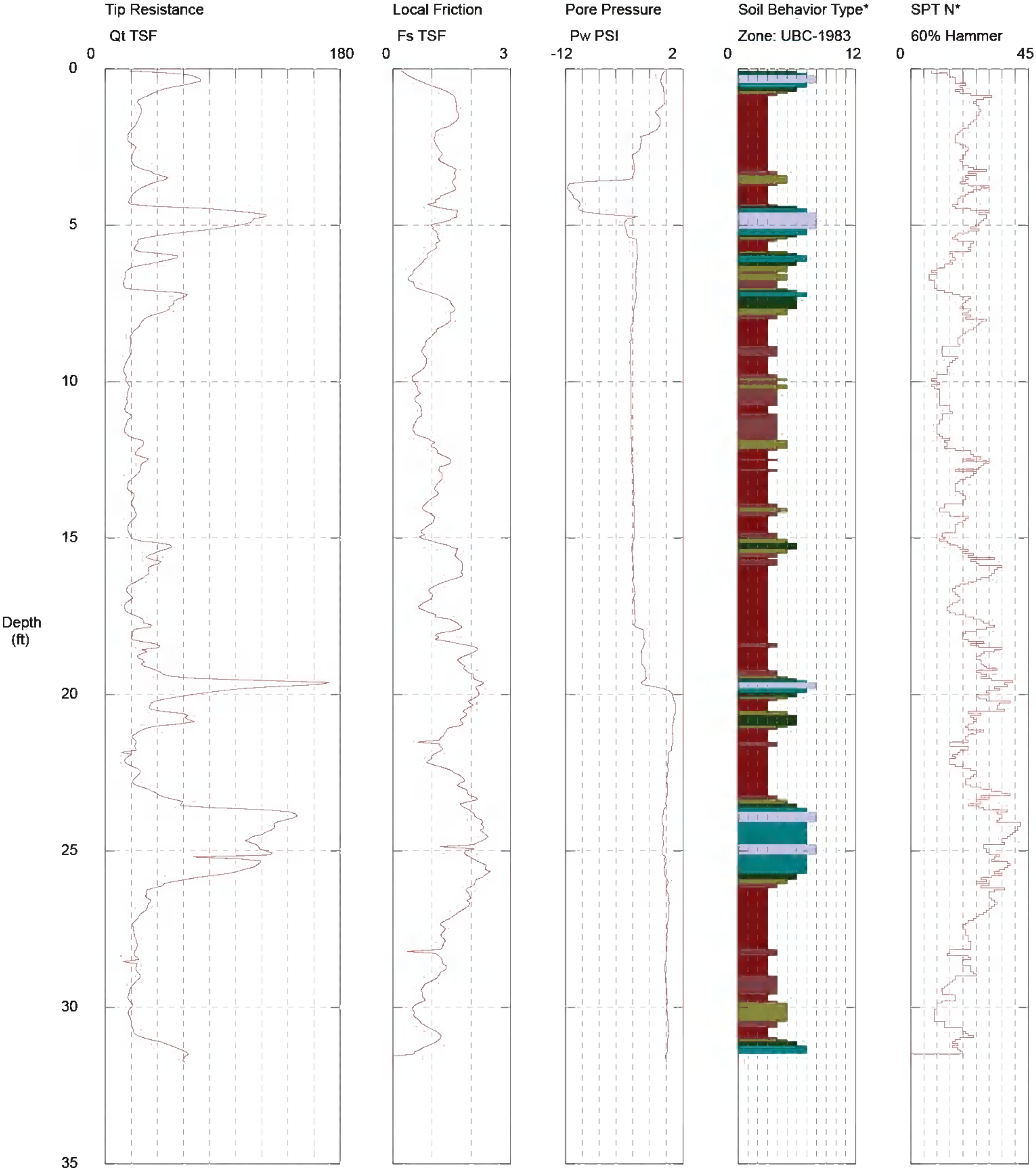
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Geosyntec consultants
engineers | scientists | innovators

Operator: Cory Robisor
Sounding: CPT-116A
Cone Used: DSG0867

CPT Date/Time: 10/8/2013 10:42:30 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 31.76 feet

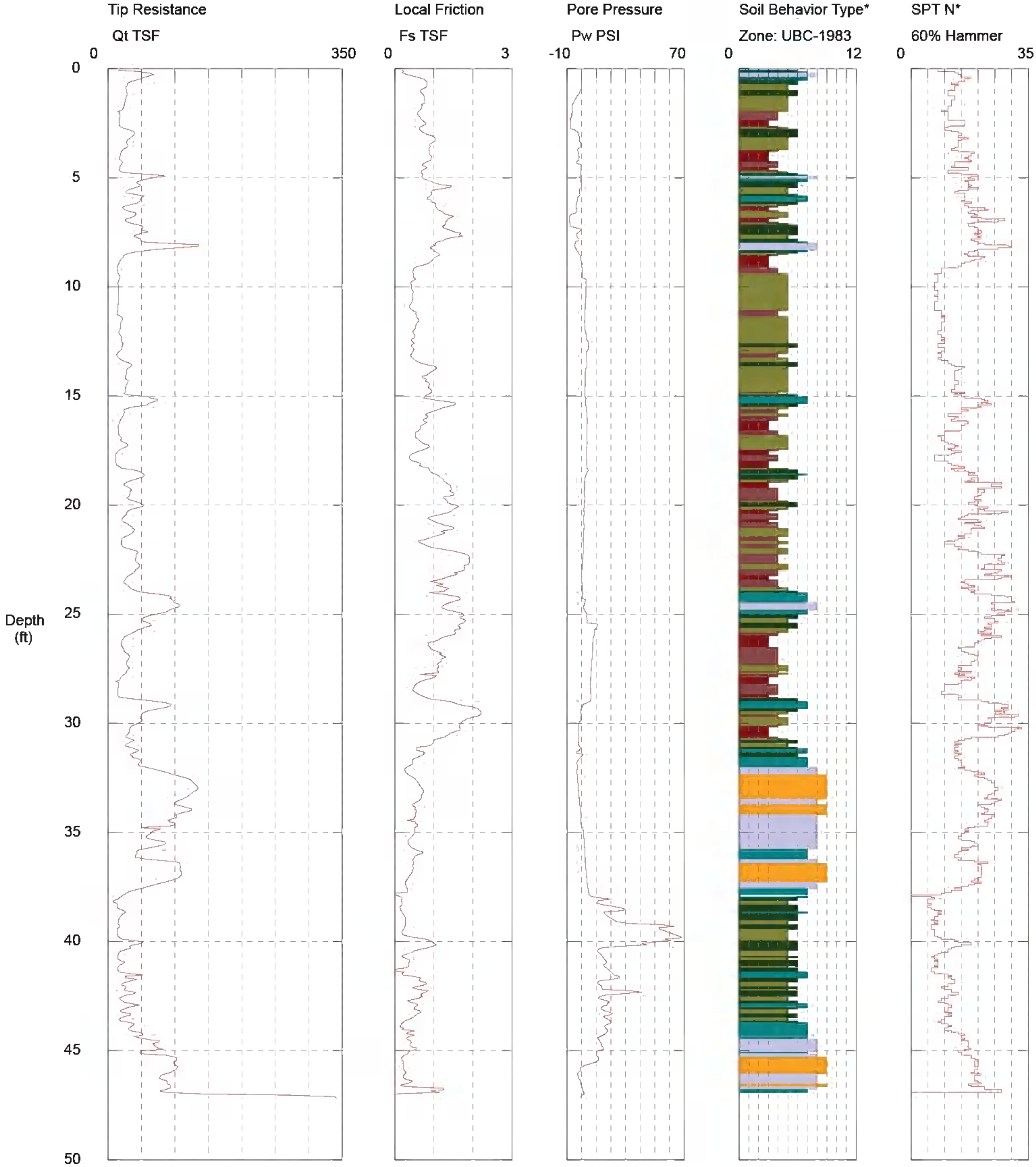
Depth Increment = 0.066 feet

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 (*)

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

Operator: Cory Robison
Sounding: CPT-117
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 2:14:33 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 47.18 feet

Depth Increment = 0.066 feet

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

Footer 1

Geosyntec

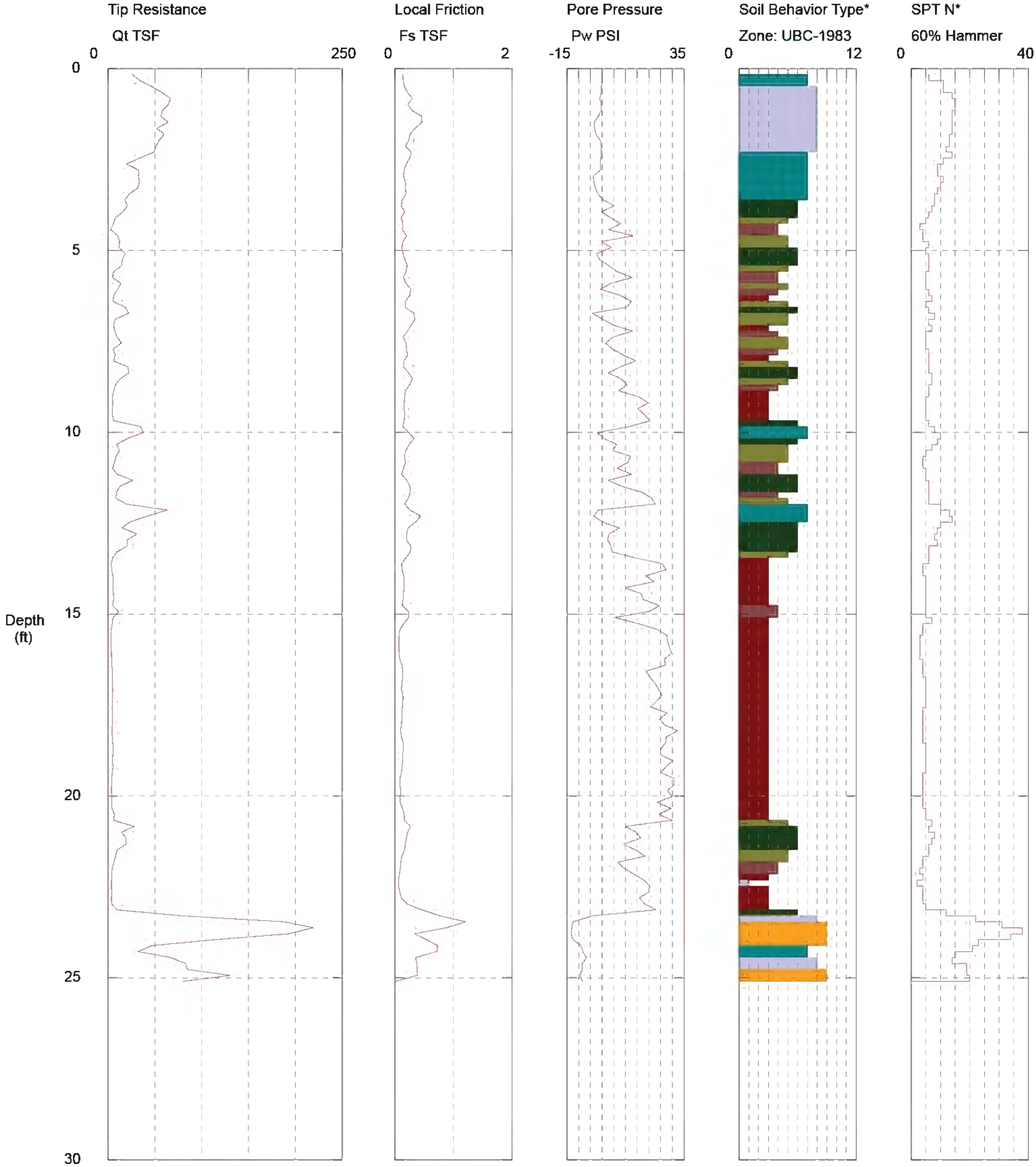
*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants

engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-119
Cone Used: DDG1195

CPT Date/Time: 9/27/2013 11:16:08 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 25.10 feet

Depth Increment = 0.164 feet

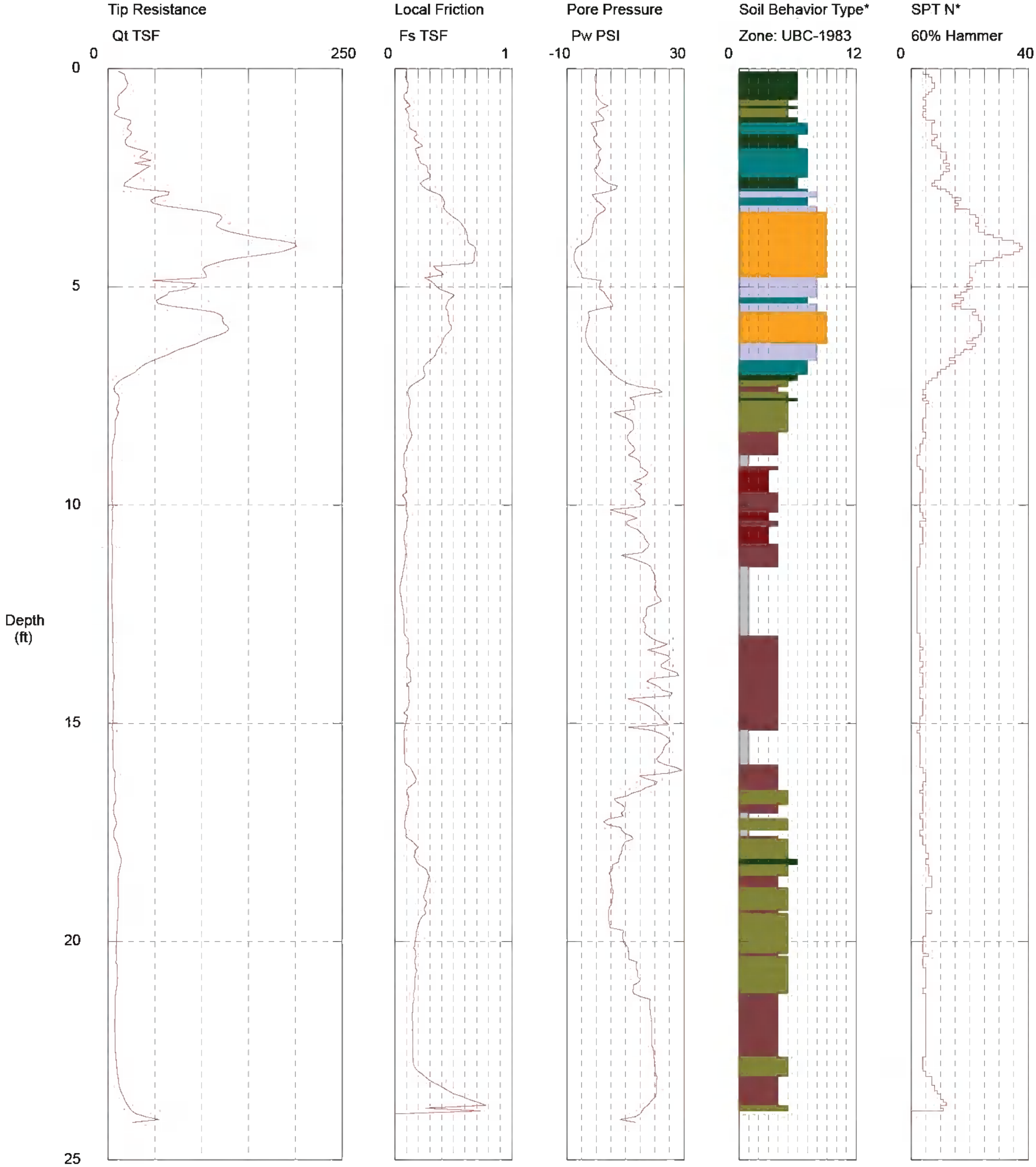
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Geosyntec

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

Operator: Cory Robison
Sounding: CPT-120
Cone Used: DDG1195

CPT Date/Time: 10/3/2013 12:04:18 PM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 24.15 feet

Depth Increment = 0.066 feet

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

Geosyntec

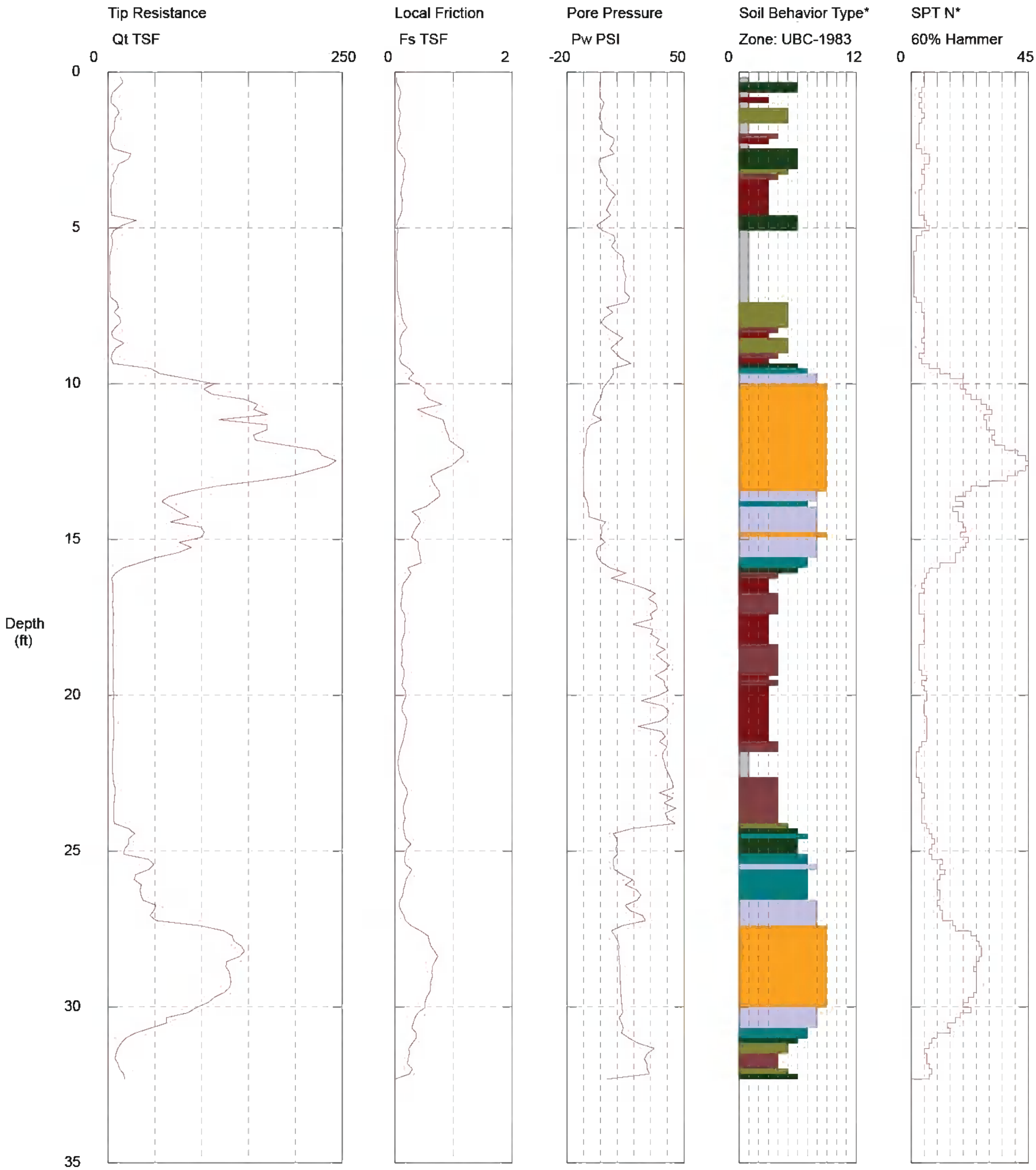
*Soil behavior type and SPT based on data from UBC-1983

Geosyntec consultants

engineers | scientists | innovators

Operator: Cory Robison
Sounding: CPT-121
Cone Used: DDG1195

CPT Date/Time: 9/27/2013 11:57:43 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Depth = 32.32 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 (*)

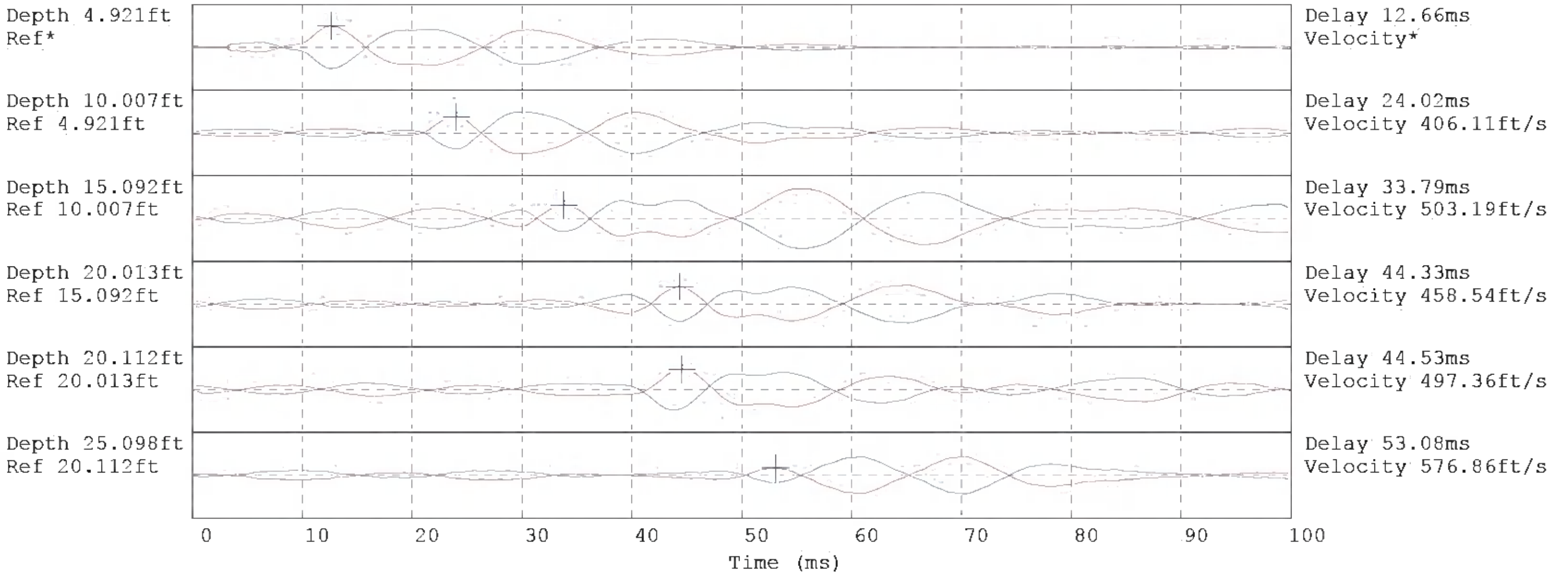
Geosyntec

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

ATTACHMENT 3-B

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

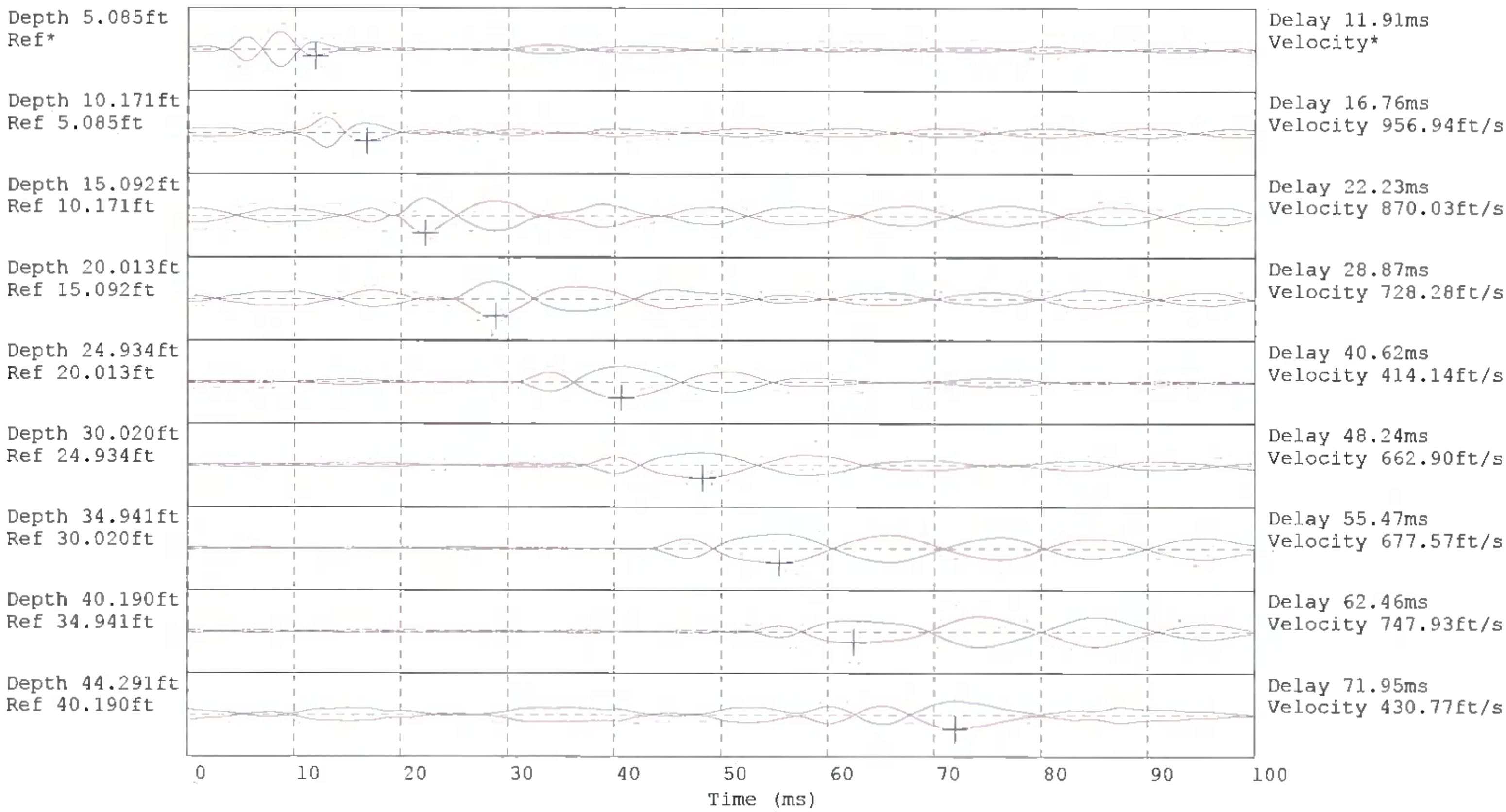
Geosyntec



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

CPT-5

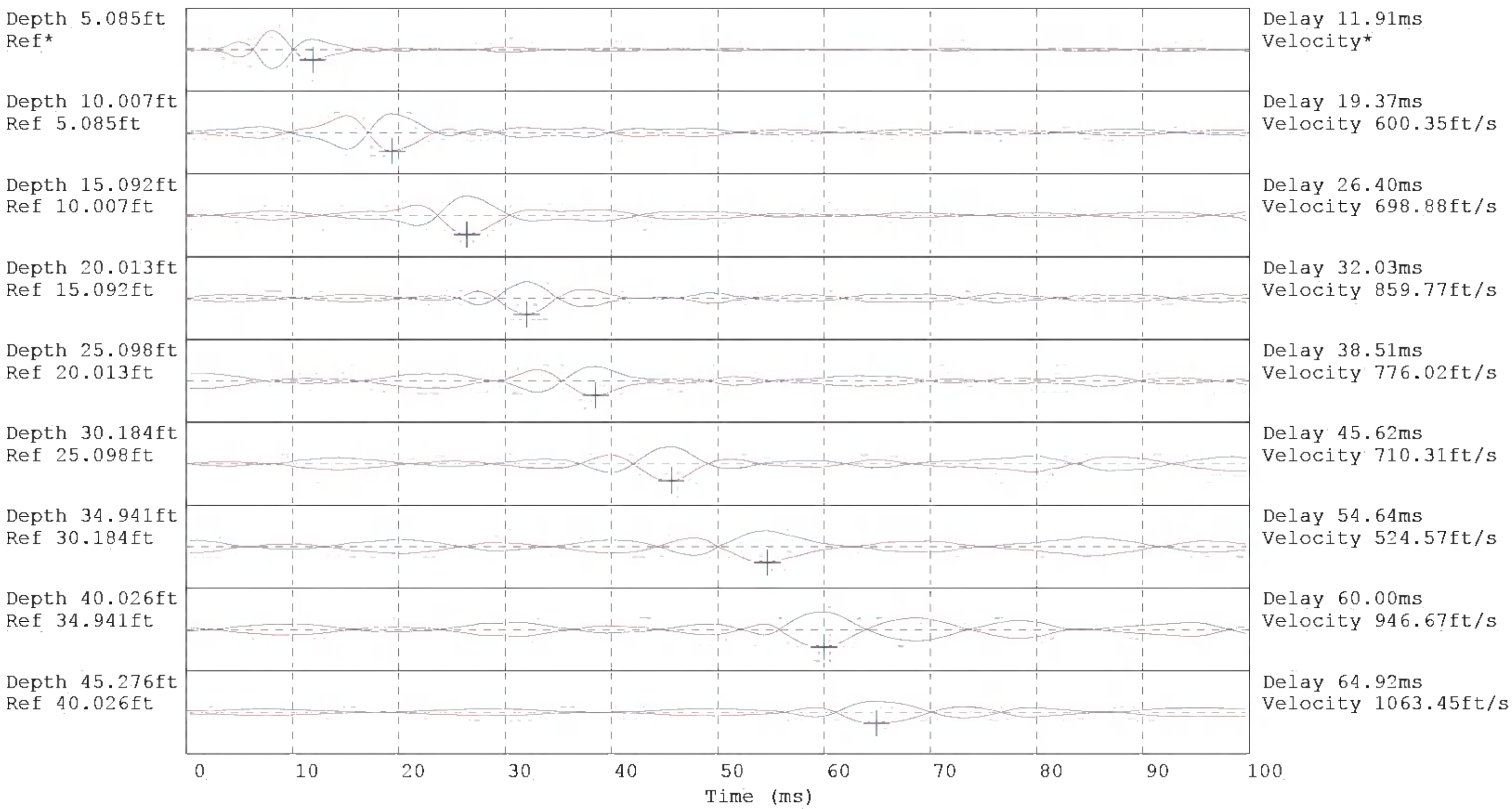
Geosyntec Consultants



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

CPT-17a

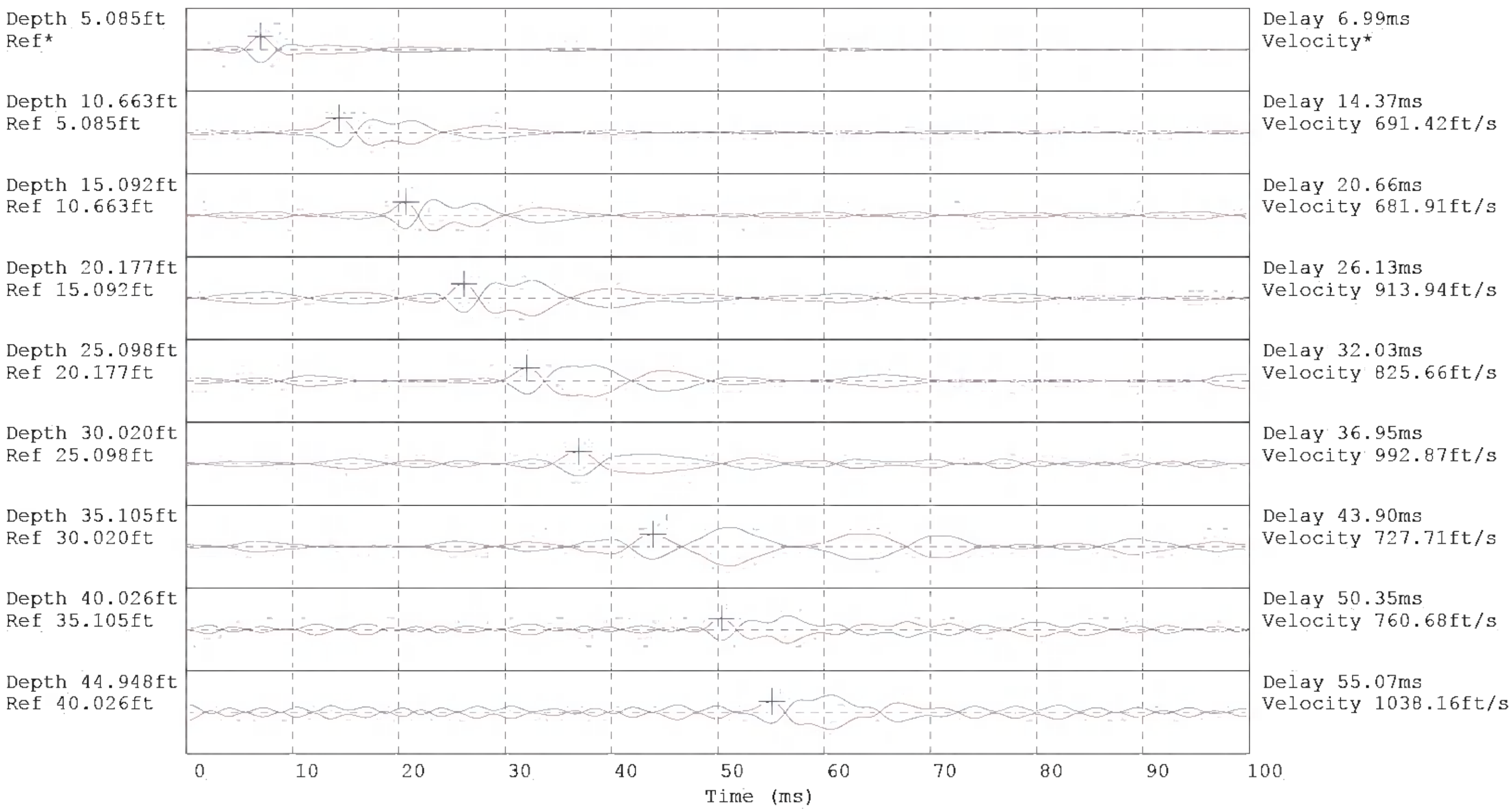
Geosyntec Consultants



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

CPT-24

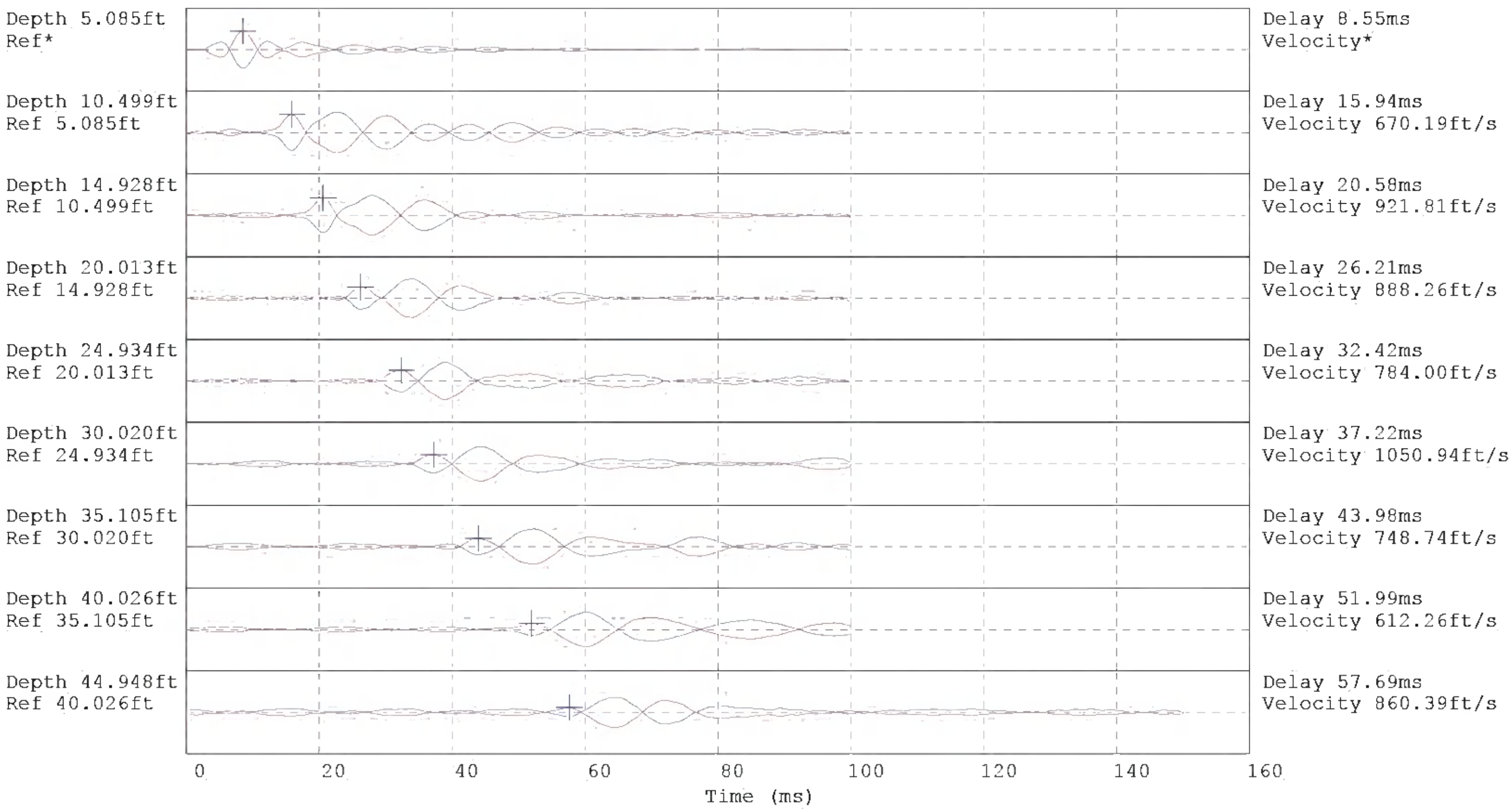
Geosyntec



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

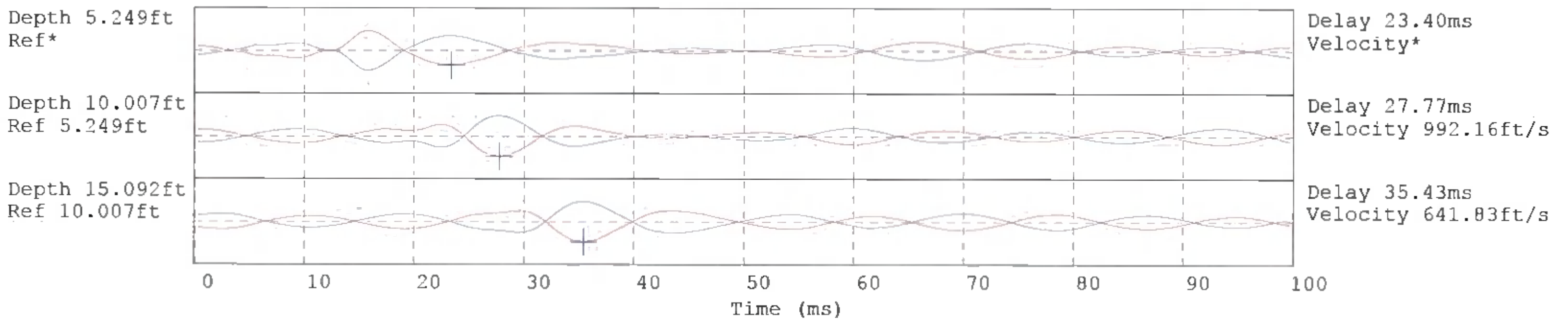
CPT-33

Geosyntec



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

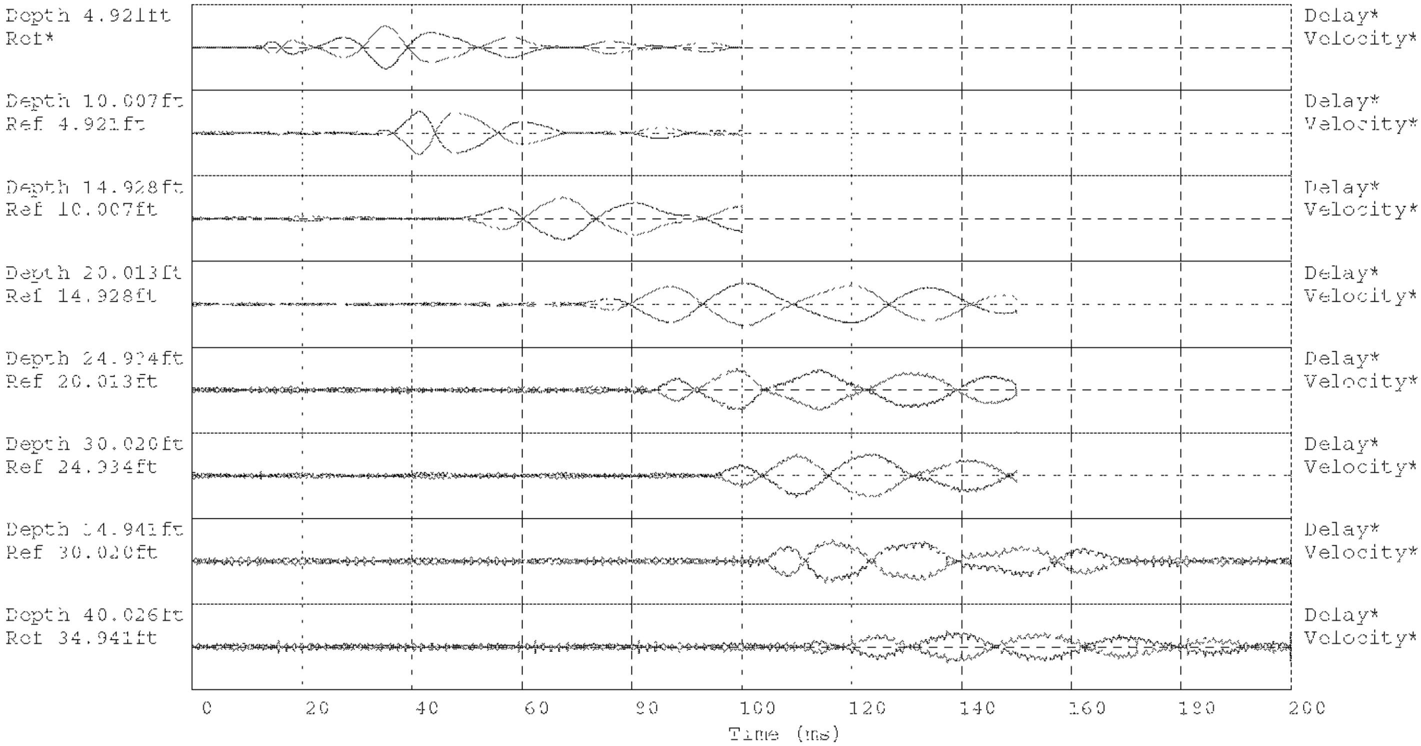
CPT-38



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

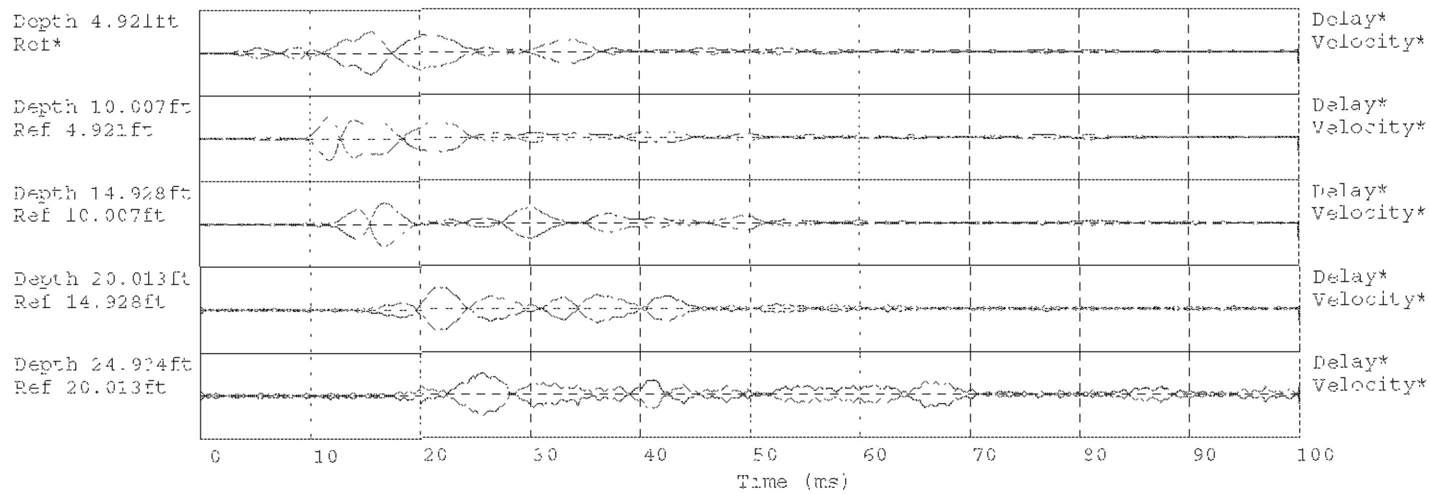
CPT-40

Mid-Atlantic Drilling, Inc.
CPT-118



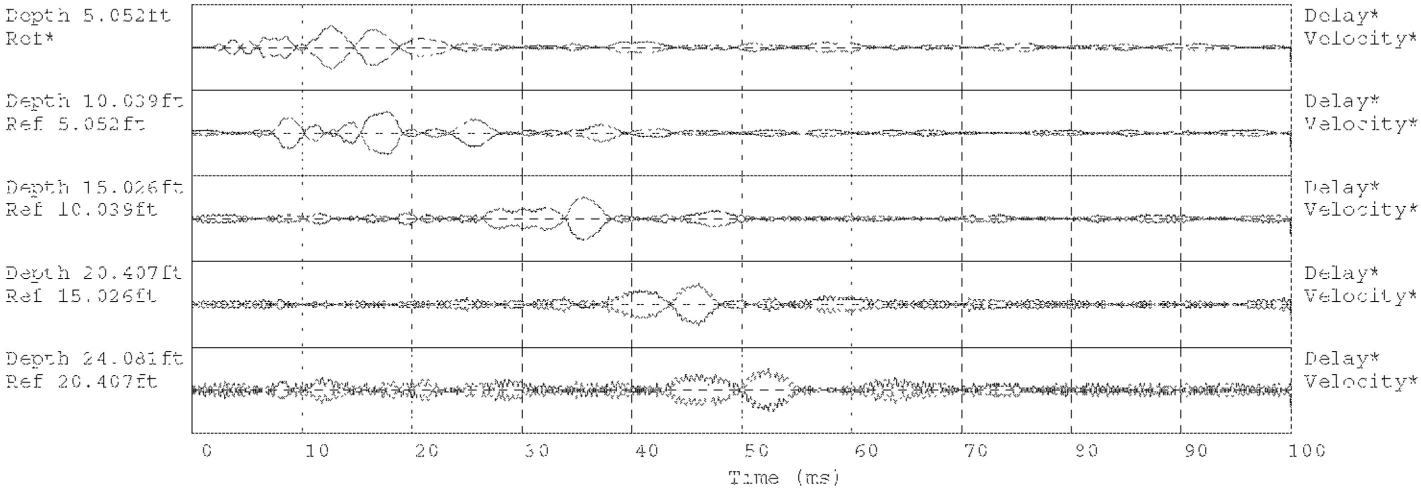
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling, Inc.
CPT-119



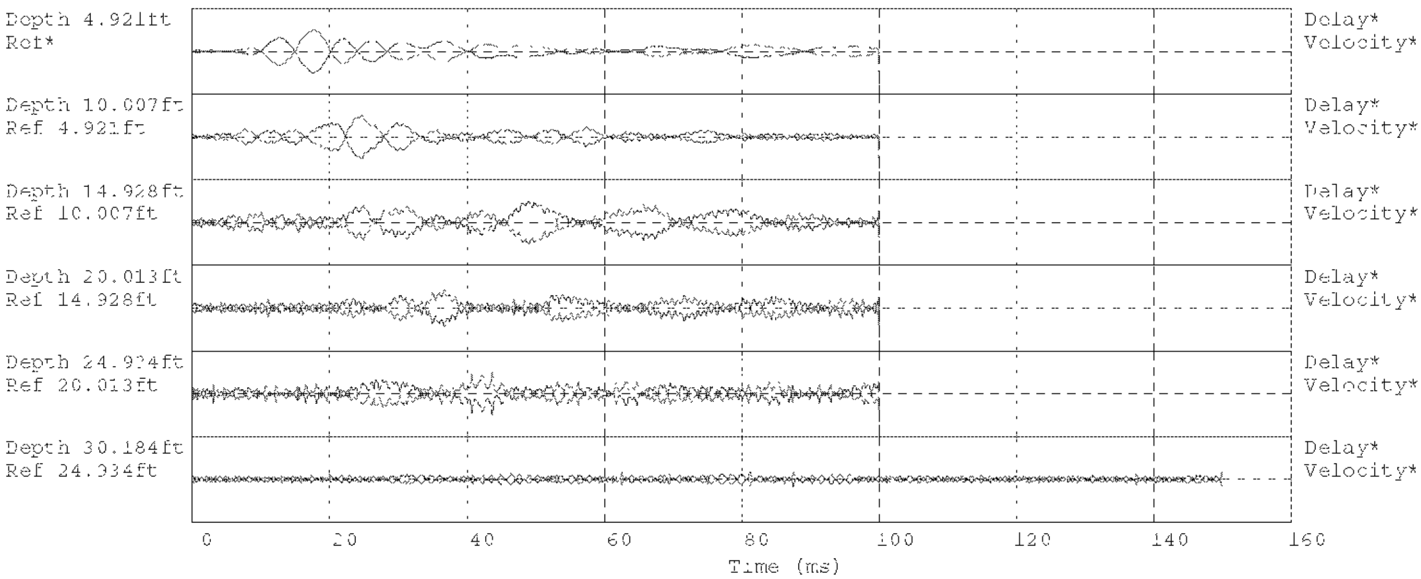
Hammer to Rod String Distance 1 (m)
* = Not Determined

Mid-Atlantic Drilling, Inc.
CPT-120



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

Mid-Atlantic Drilling, Inc.
CPT-121



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

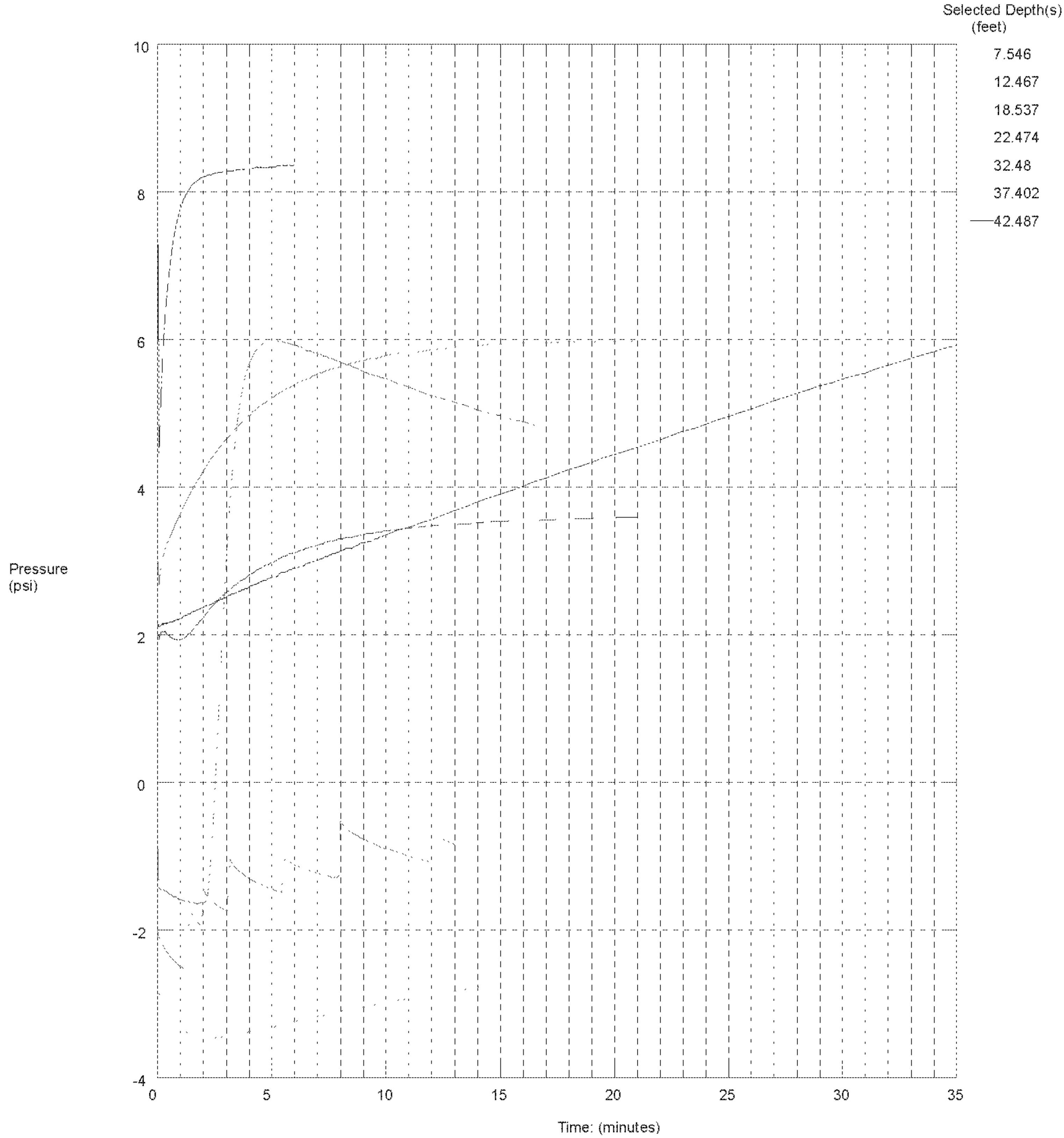
ATTACHMENT 3-C

Dissipation Test Data
(Provided by Mid-Atlantic Drilling)

Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-24
Cone Used: DSG1156

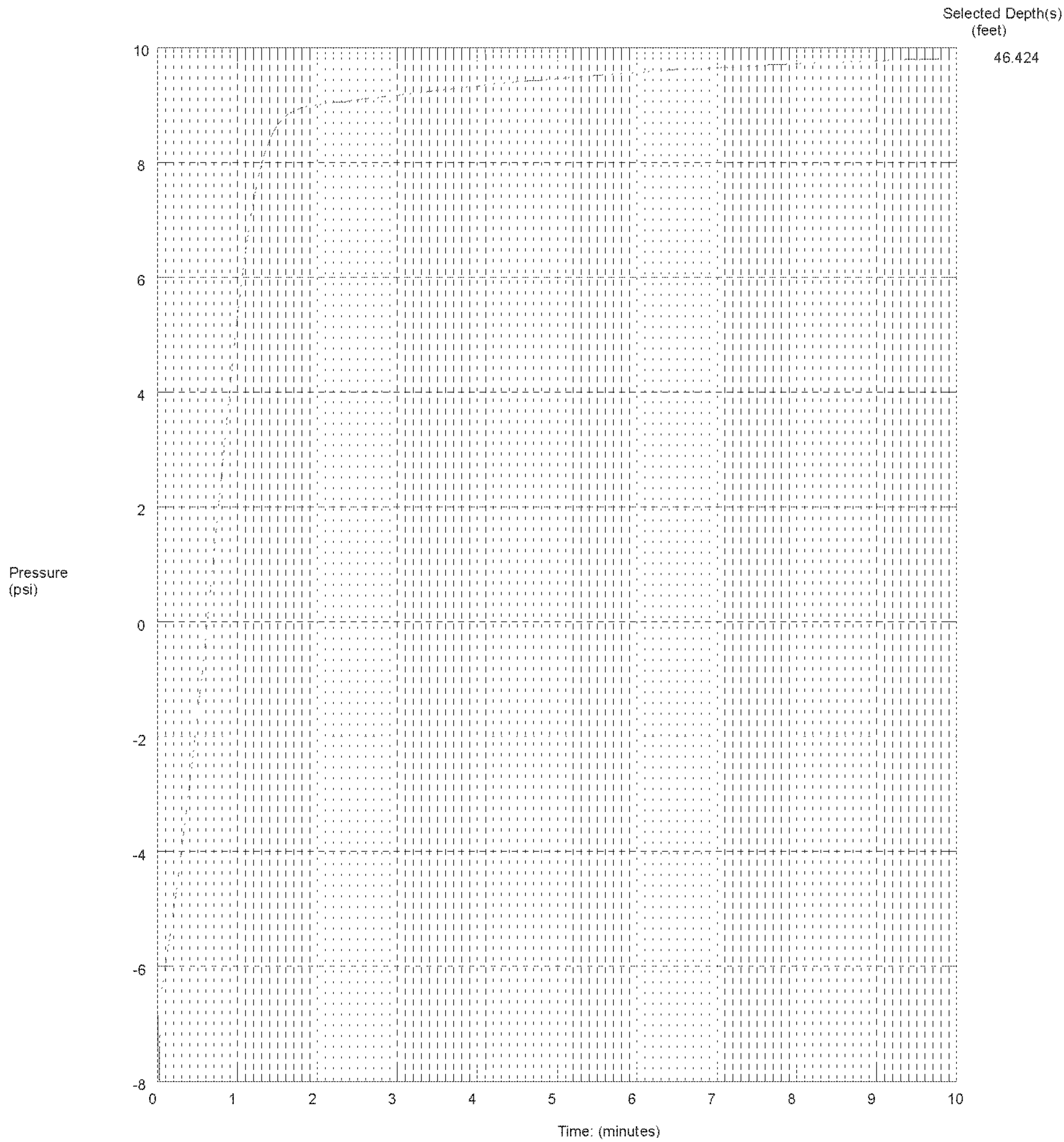
CPT Date/Time: 2/21/2013 3:32:37 PM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-28
Cone Used: DSG1156

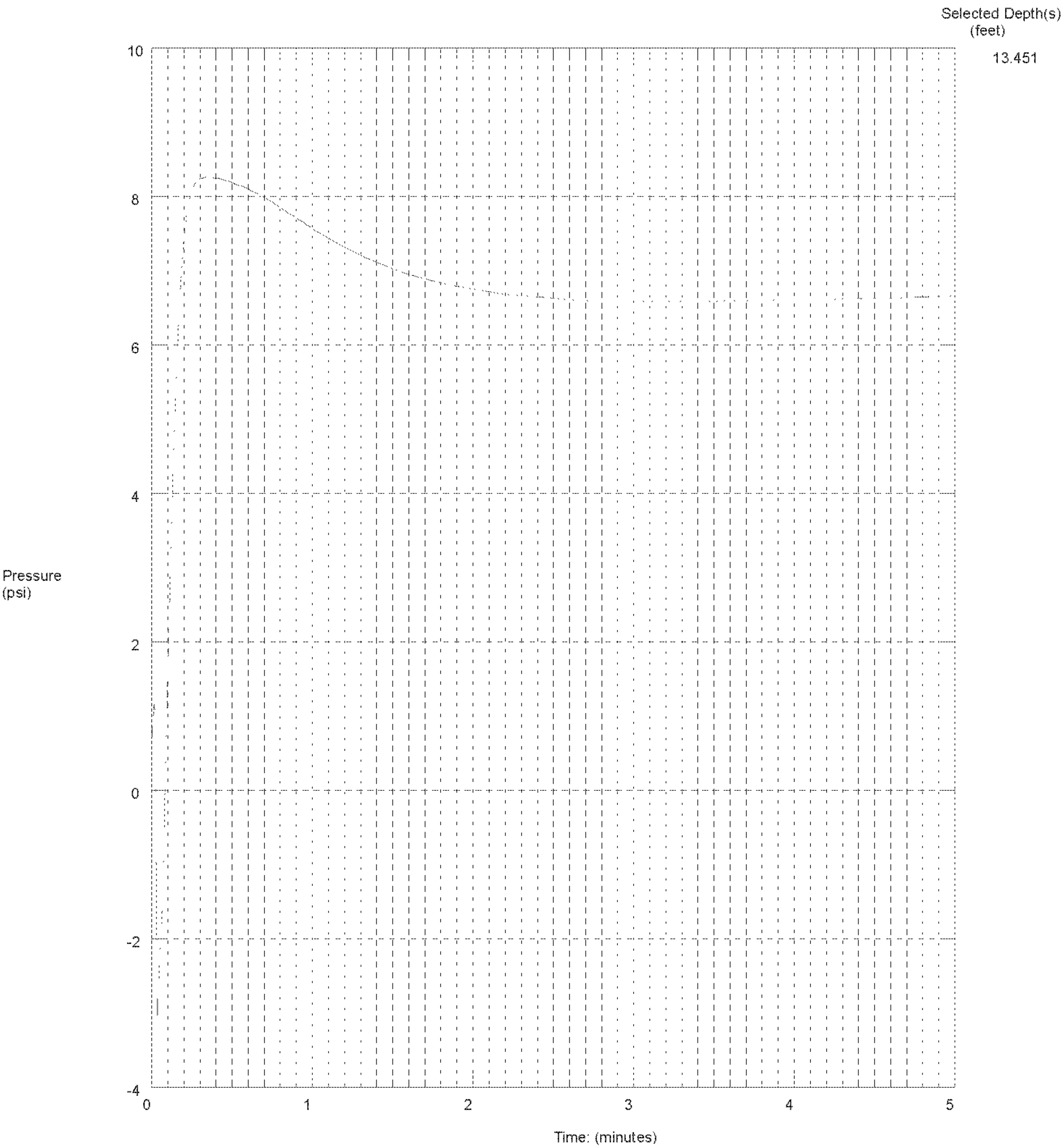
CPT Date/Time: 3/4/2013 5:27:47 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-31
Cone Used: DSG1156

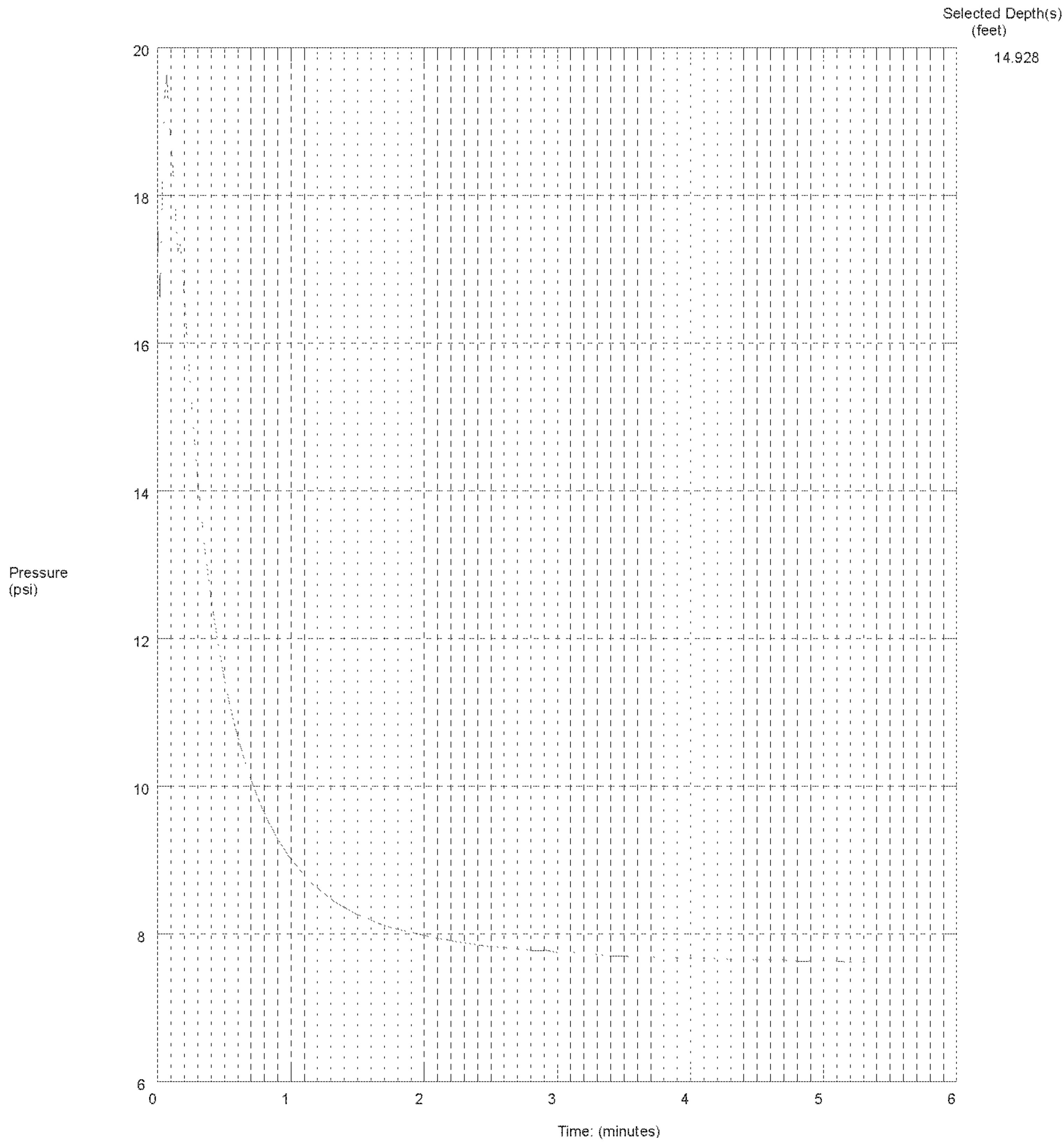
CPT Date/Time: 3/1/2013 2:32:13 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-32
Cone Used: DSG1156

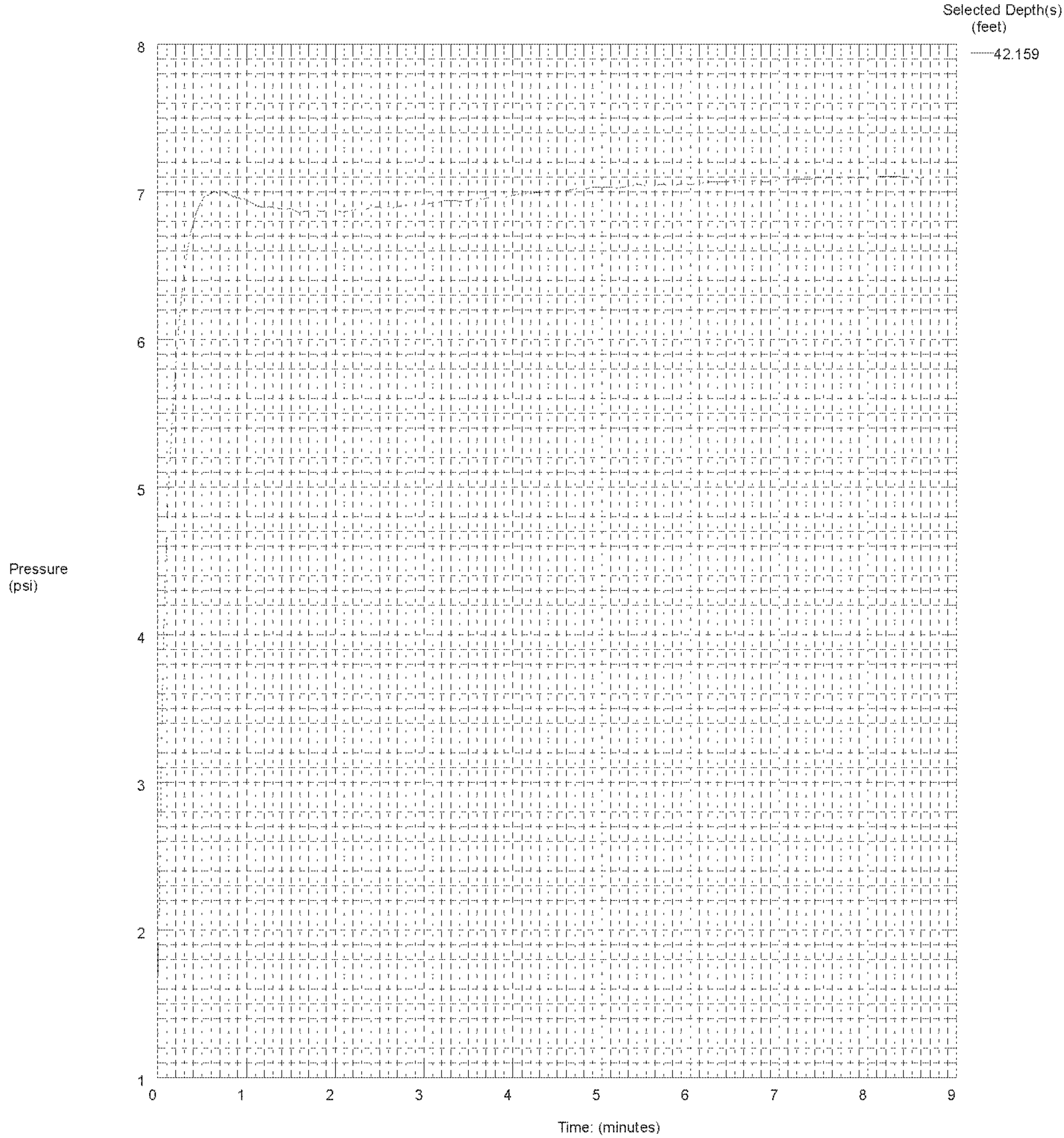
CPT Date/Time: 3/1/2013 1:59:28 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-33
Cone Used: DSG1156

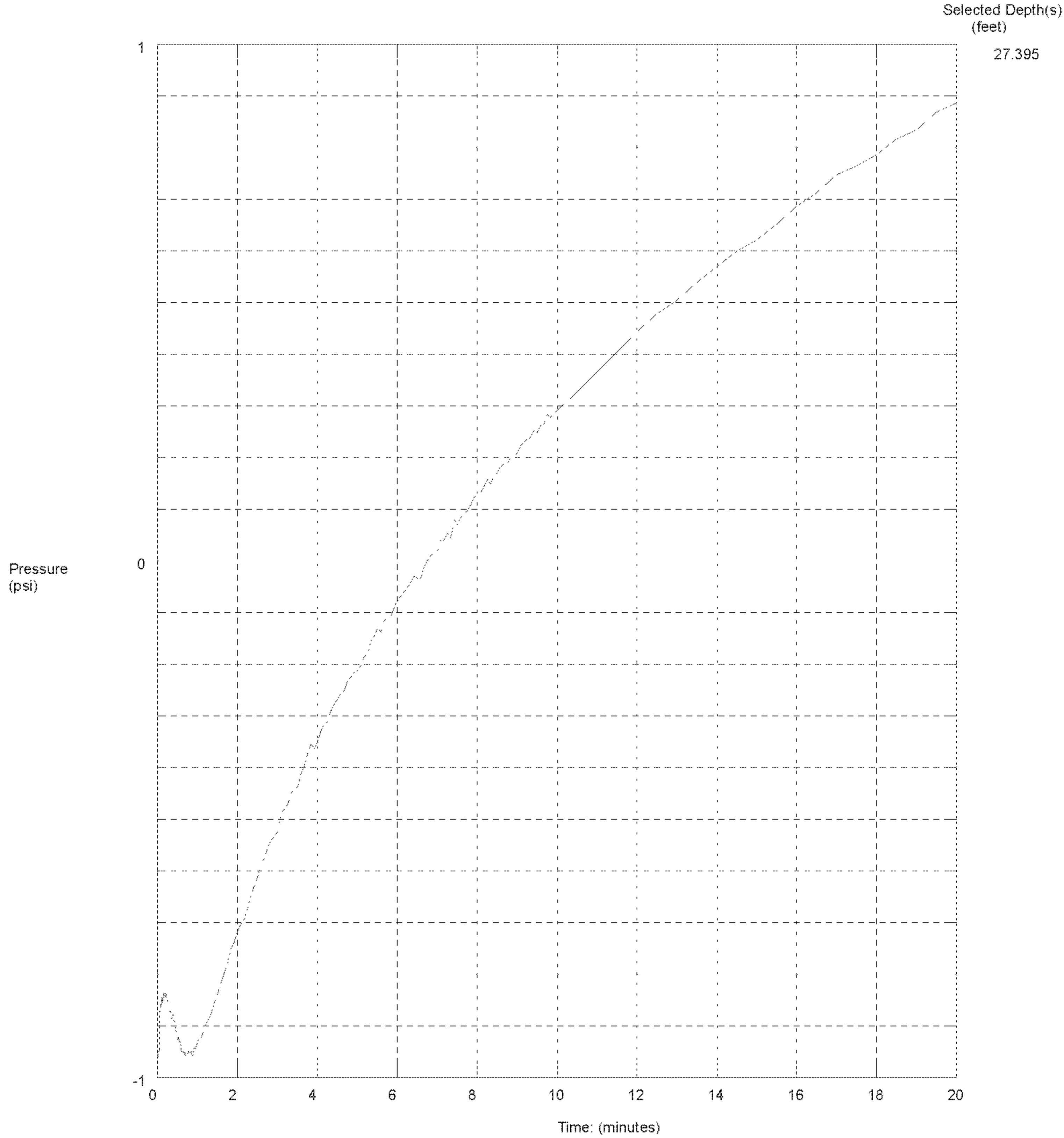
CPT Date/Time: 2/22/2013 9:15:47 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-38
Cone Used: DSG1156

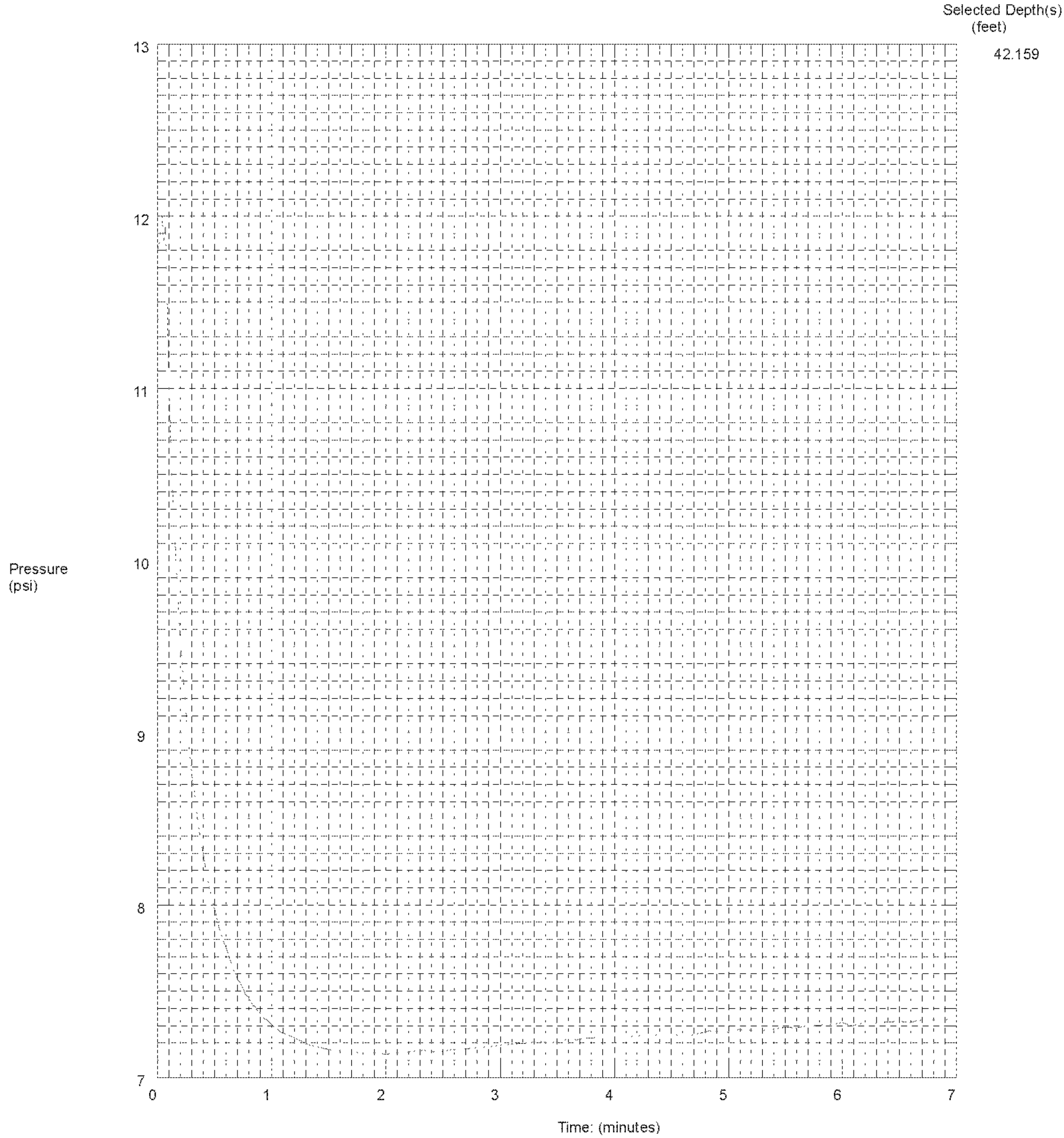
CPT Date/Time: 2/22/2013 11:19:28 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-38
Cone Used: DSG1156

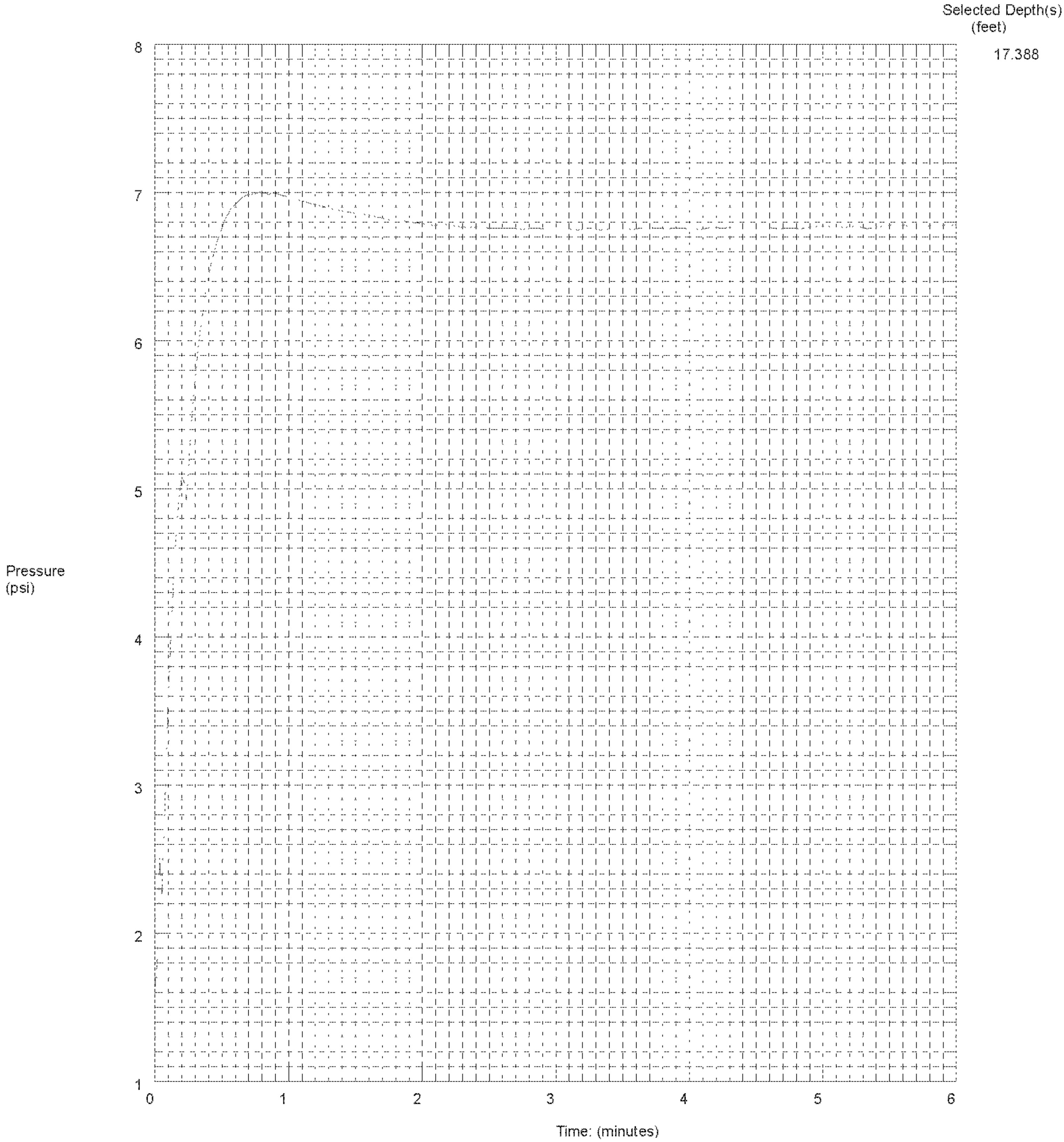
CPT Date/Time: 2/22/2013 11:19:28 AM
Location: Winyah Generating Station S.C.
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPT-40
Cone Used: DSG1156

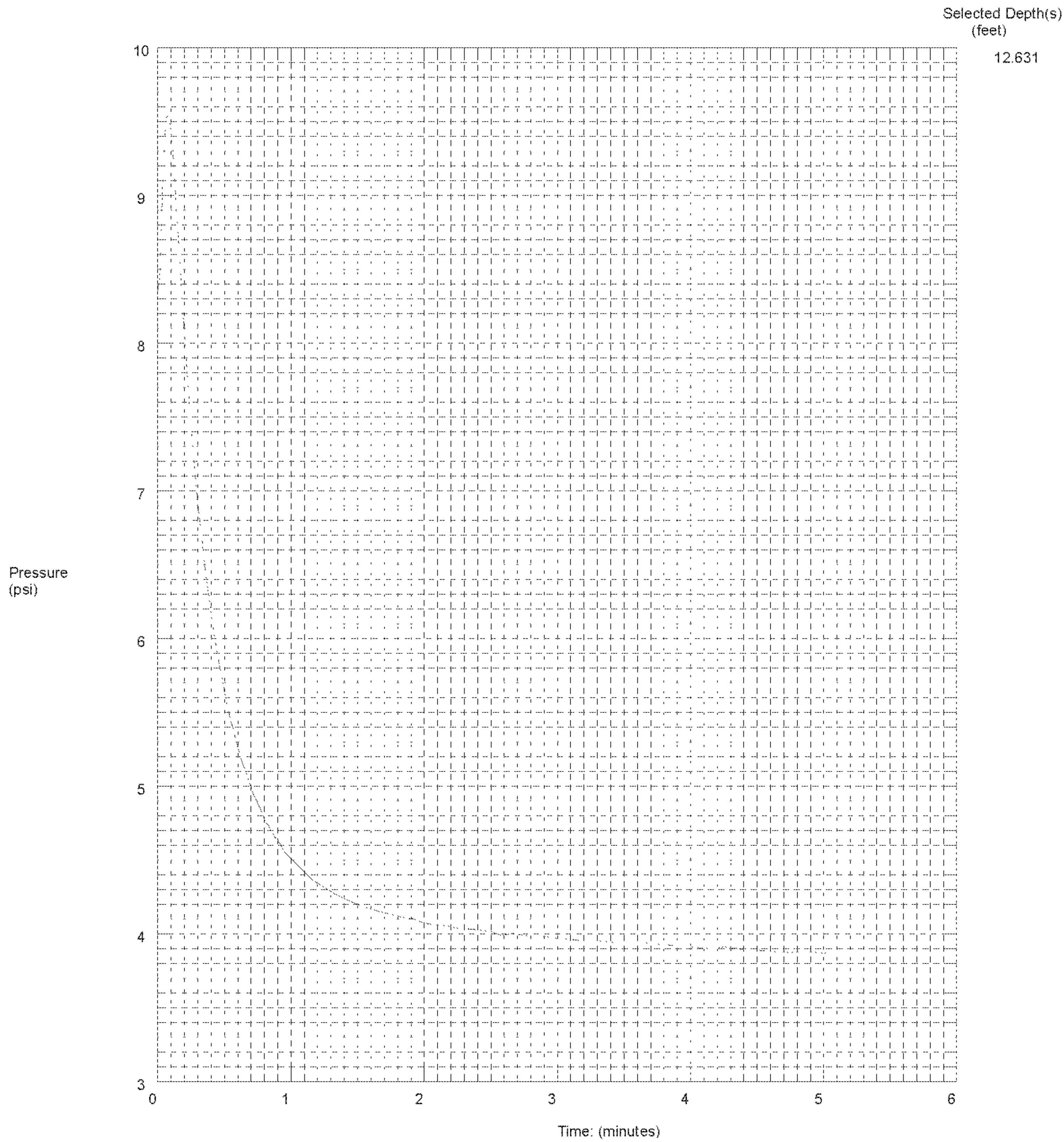
CPT Date/Time: 3/4/2013 9:53:04 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-86
Cone Used: DSG1156

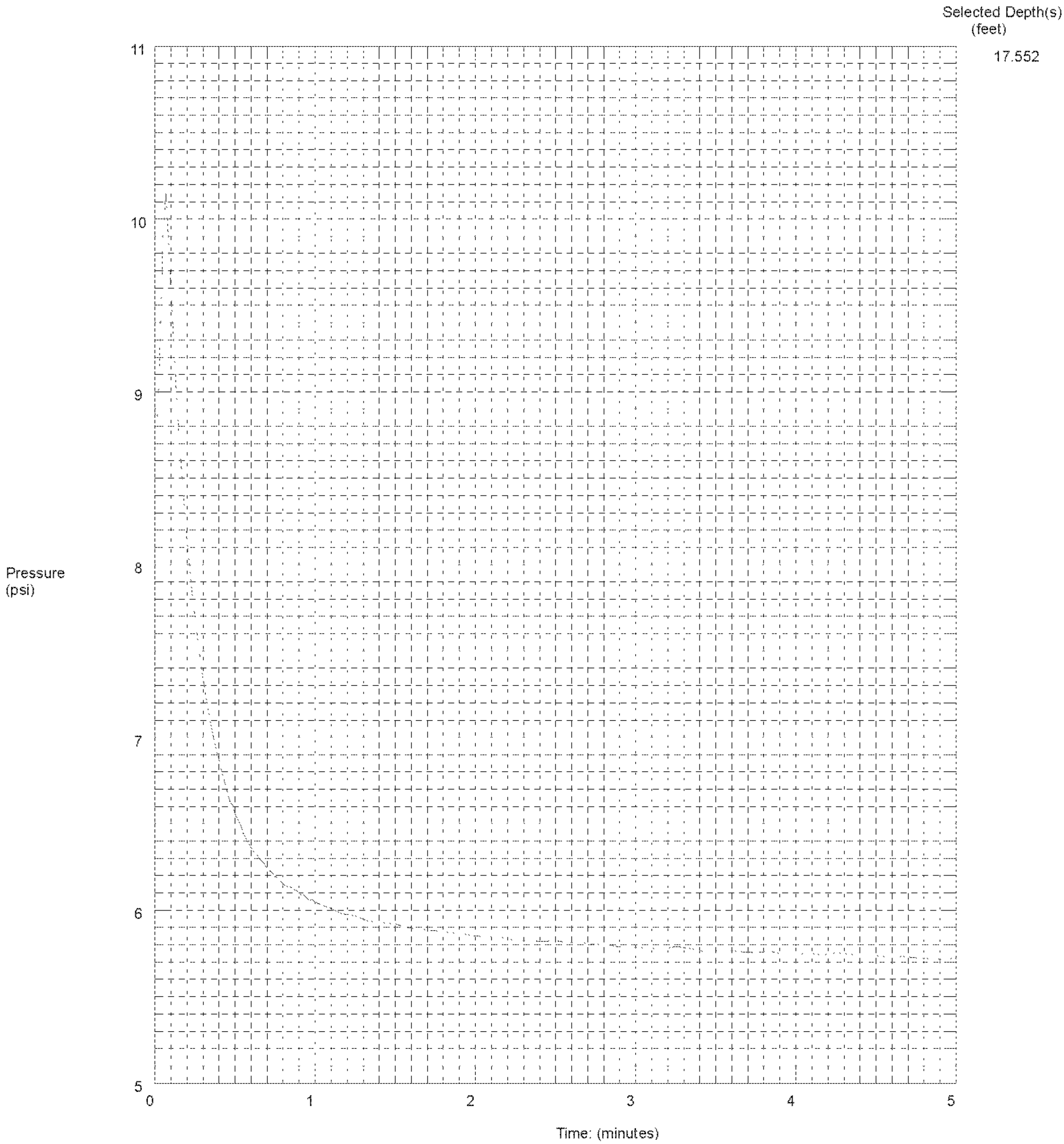
CPT Date/Time: 2/28/2013 12:15:17 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-86
Cone Used: DSG1156

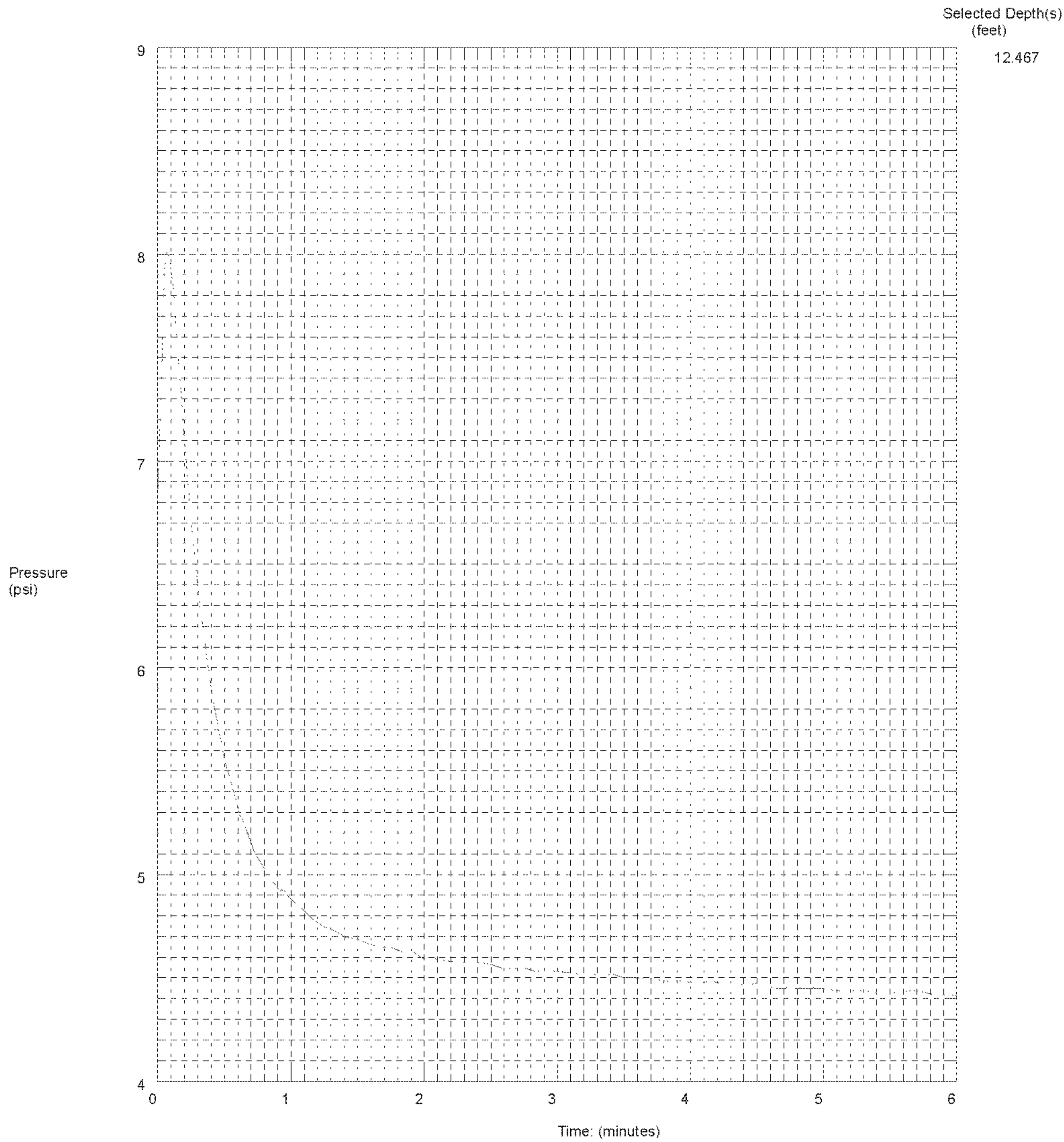
CPT Date/Time: 2/28/2013 12:15:17 PM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-87
Cone Used: DSG1156

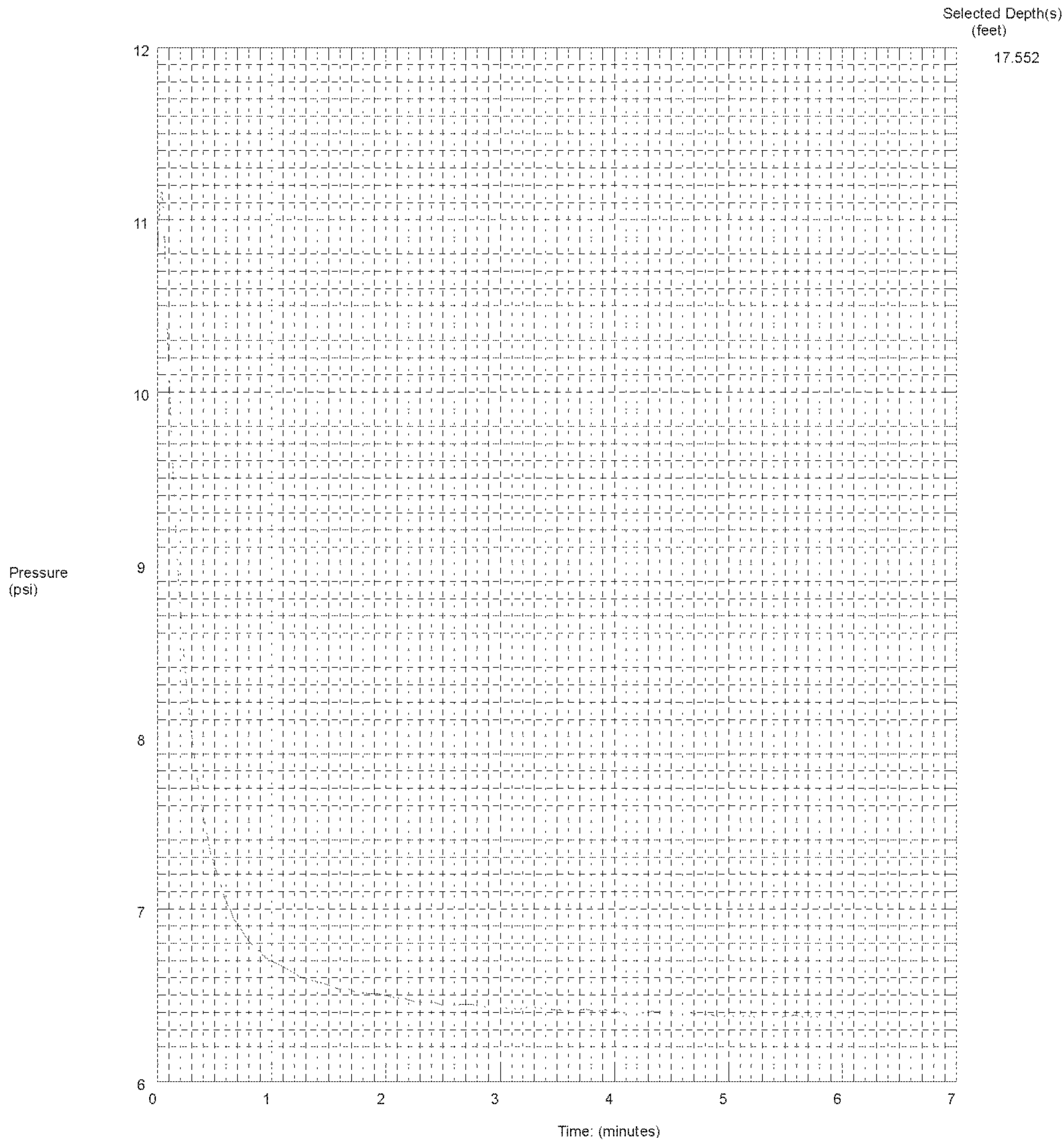
CPT Date/Time: 2/28/2013 11:36:32 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-87
Cone Used: DSG1156

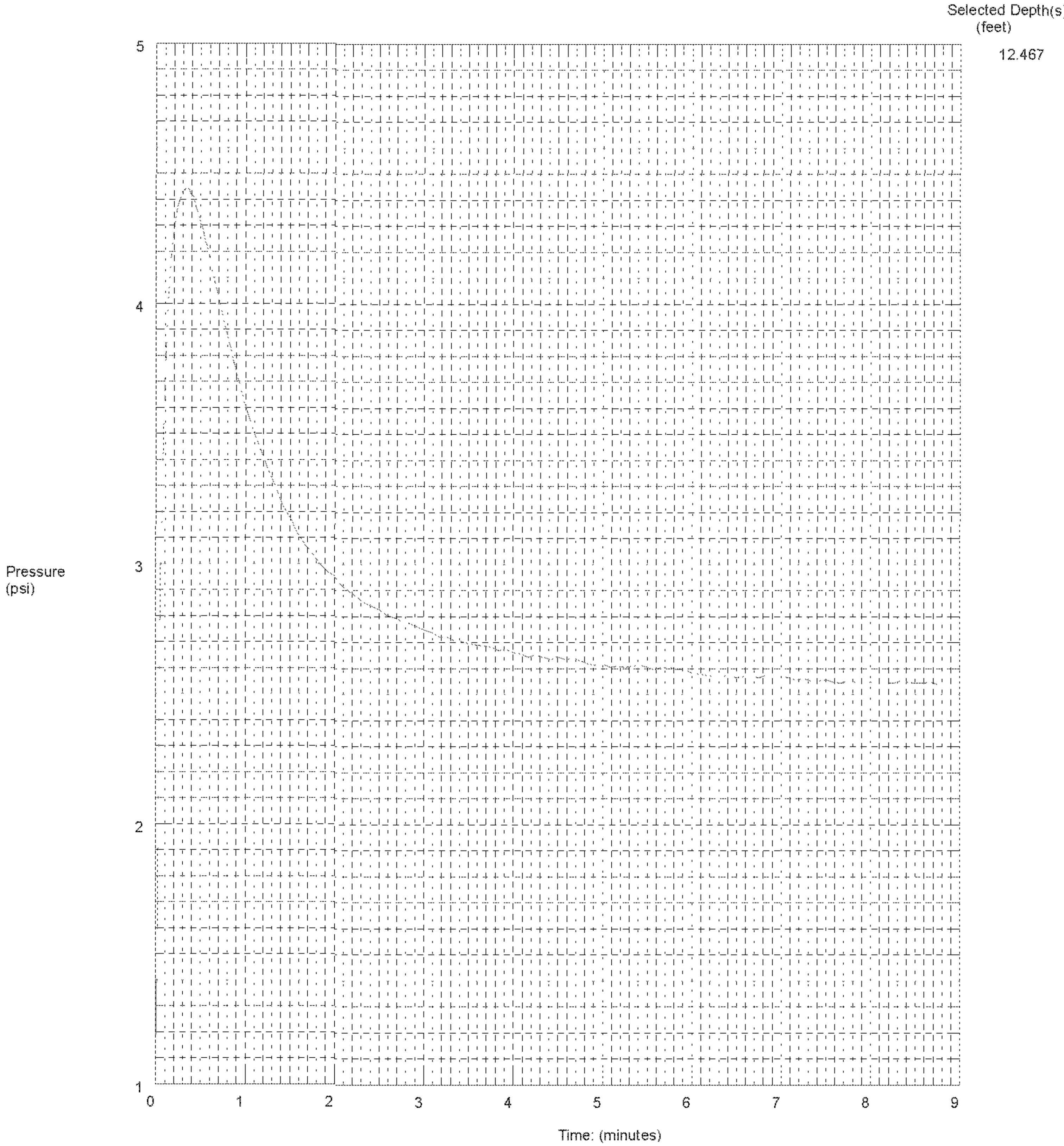
CPT Date/Time: 2/28/2013 11:36:32 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-88
Cone Used: DSG1156

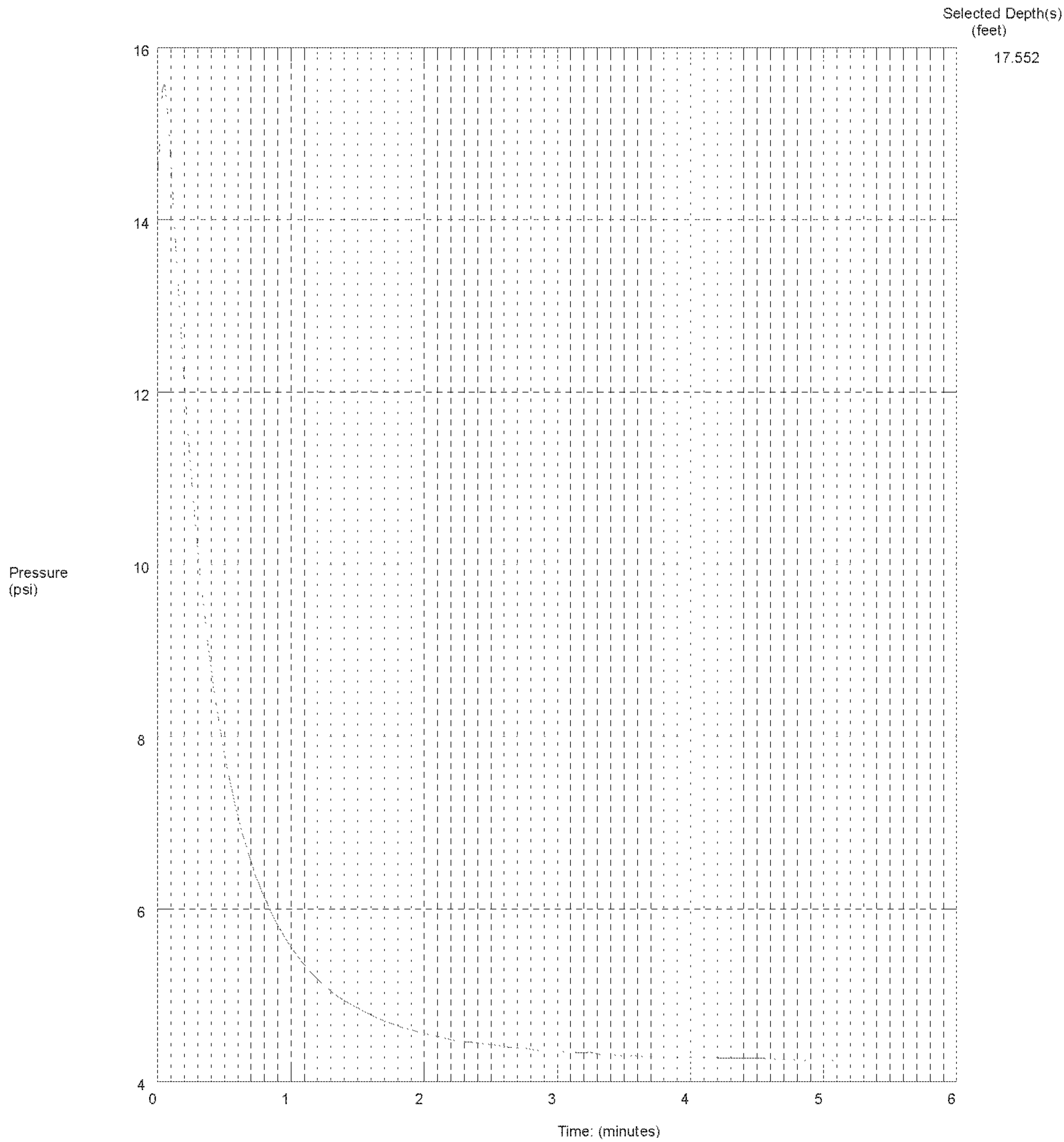
CPT Date/Time: 2/28/2013 9:06:33 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-88
Cone Used: DSG1156

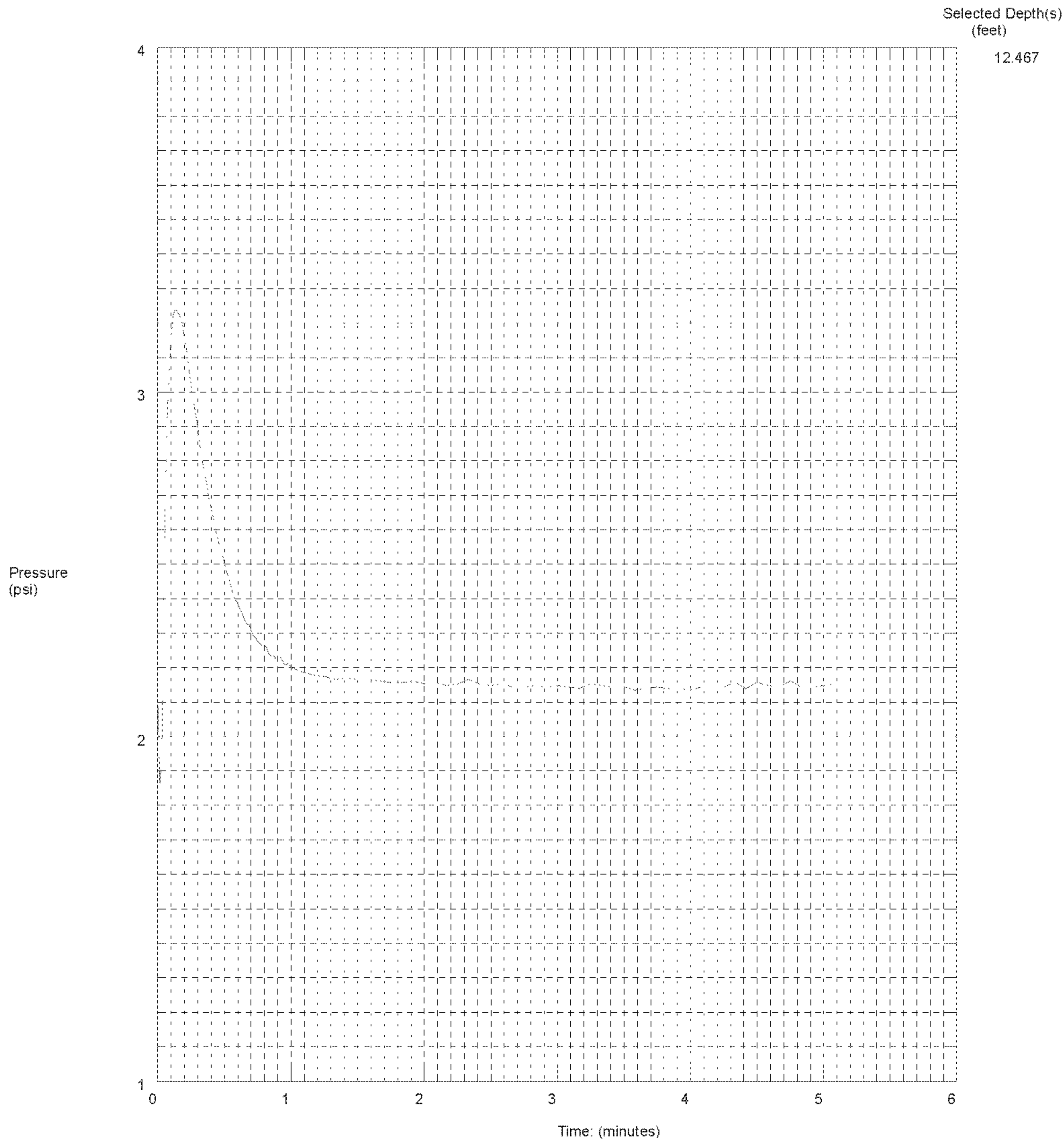
CPT Date/Time: 2/28/2013 9:06:33 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-89
Cone Used: DSG1156

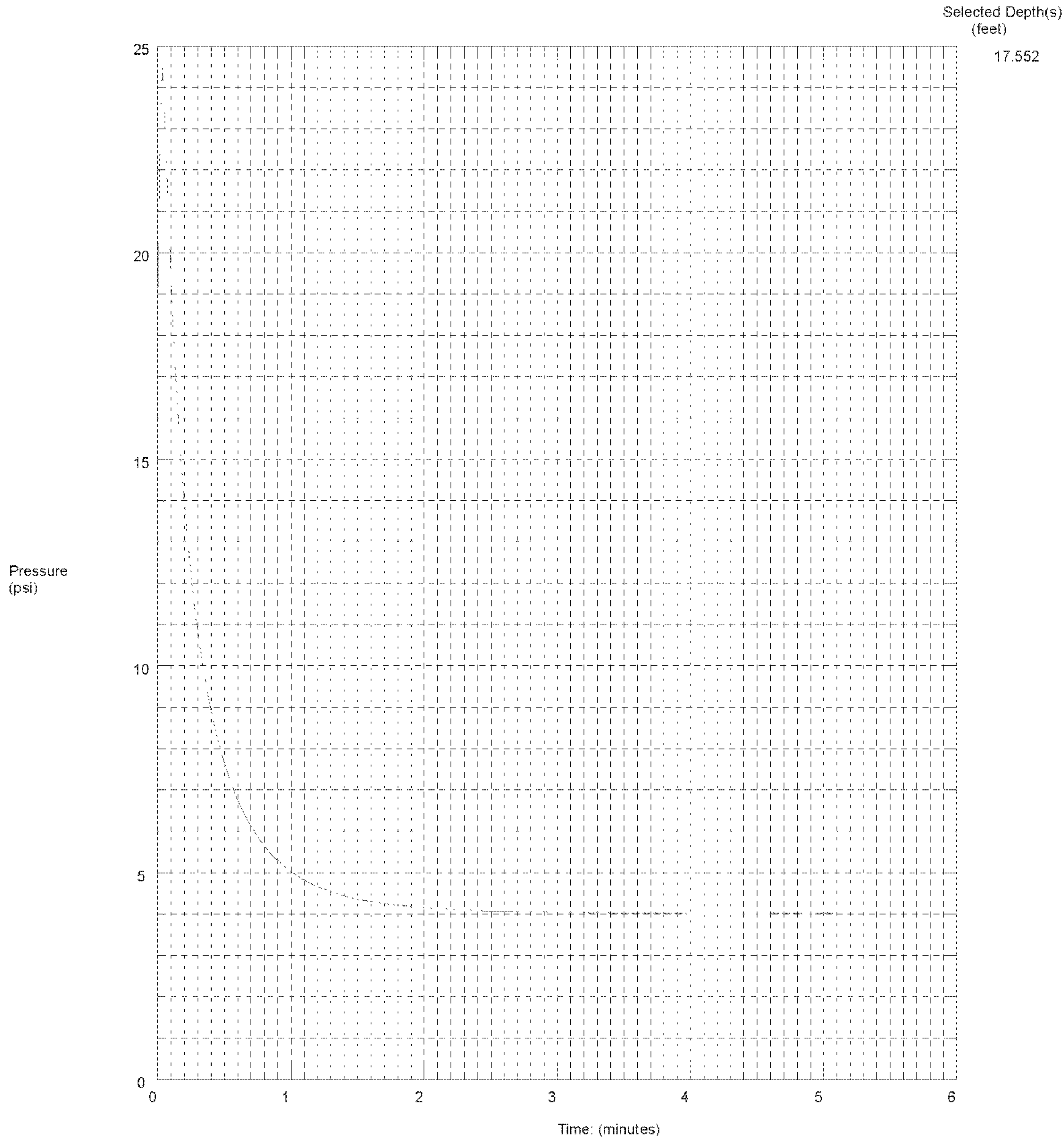
CPT Date/Time: 2/28/2013 8:22:17 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Geosyntec Consultants

Operator Cory Robison
Sounding: CPTO-89
Cone Used: DSG1156

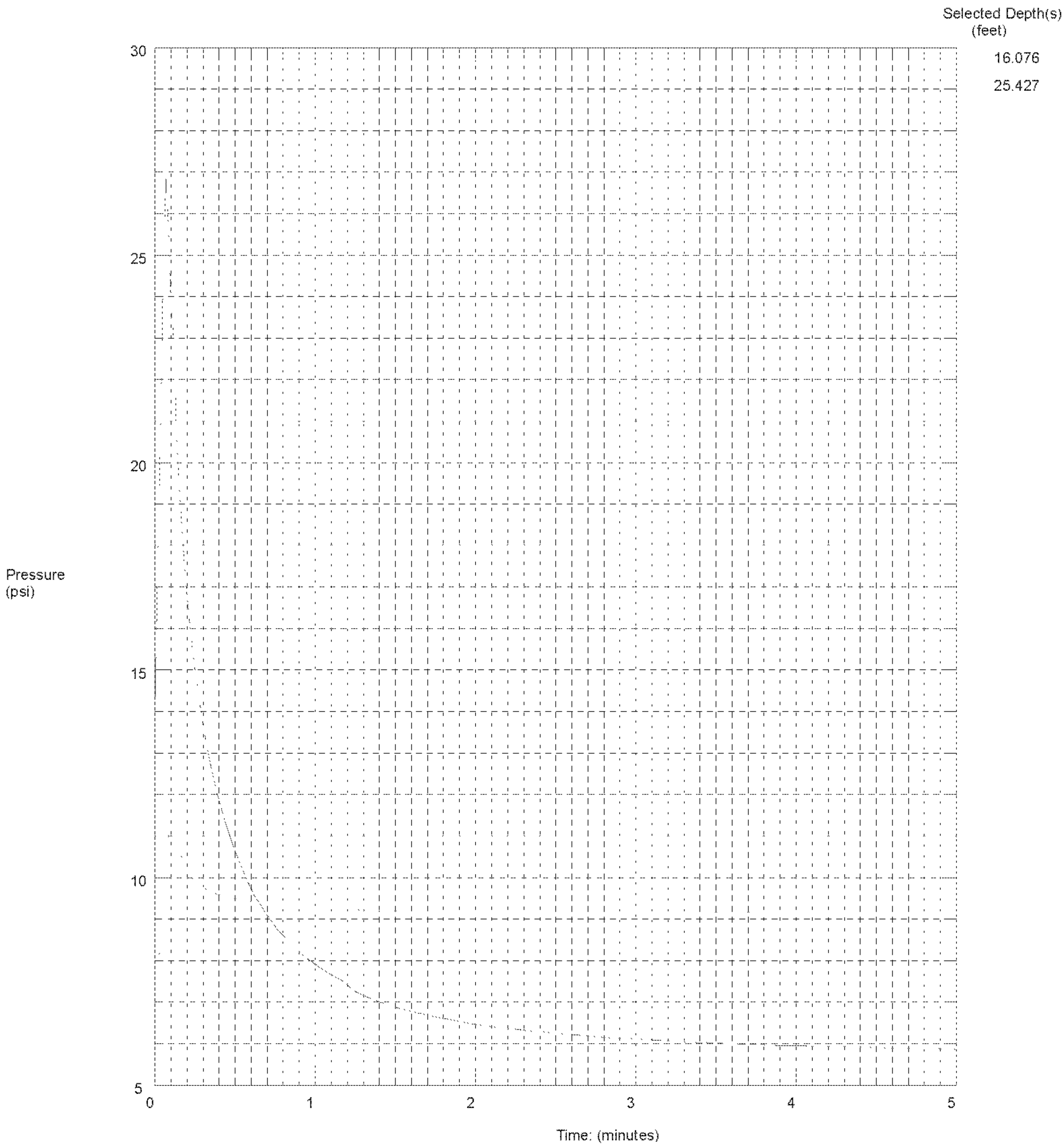
CPT Date/Time: 2/28/2013 8:22:17 AM
Location: Winyah Generating Station
Job Number: GSC5242/01BT



Mid-Atlantic Drilling Inc.

Operator Cory Robison
Sounding: CPT-121
Cone Used: DDG1195

CPT Date/Time: 9/27/2013 11:57:43 AM
Location: Georgetown S.C.
Job Number: GSC-5242



Maximum Pressure = 26.874 psi
Hydrostatic Pressure = 11.035 psi

..
Geosyntec
..

ATTACHMENT 4

Laboratory Testing Results

ATTACHMENT 4-A

Geosyntec Laboratory Testing Results (Provided by Excel Geotechnical Testing and Terracon)

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.: 585

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		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
[redacted]												
GSB-02 (5')	13C036	32.9	19.1	45.0	35.9	7.9	28.0	42	23	19		
GSB-02 (10')	13C037	32.0			18.9							
GSB-02 (15')	13C038	33.0			22.3							
GSB-02 (20')	13C039											
GSB-02 (25')	13C040	34.2			22.8							
GSB-02 (30')	13C041											
GSB-03 (5')	13C042											
GSB-03 (10')	13C043	21.6	8.2	41.3	50.5	12.5	38.0	29	18	11		
GSB-03 (15')	13C044											
GSB-03 (20')	13C045	24.3						50	23	27		
GSB-03 (25')	13C046											
GSB-03 (30')	13C047	58.0	0.2	41.0	58.8							
GSB-03 (35')	13C048	35.4			28.0							
GSB-03 (40')	13C049	33.9	6.8	66.3	26.9							
GSB-03 (44.5')	13C050											
GSB-04 (5')	13C051											
GSB-04 (10')	13C052	19.9	0.0	81.8	18.2	4.8	13.4	35	19	16		
GSB-04 (15')	13C053											
GSB-04 (20')	13C054	20.1						42	22	20		
GSB-04 (25')	13C055											
GSB-04 (32')	13C056	29.1	0.0	80.1	19.9							
GSB-04 (35')	13C057	47.2			39.0							
GSB-04 (40')	13C058	35.3			19.6							

Notes:

5-21-13 NSK

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.: 585

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		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
GSB-04 (45')	13C059											
GSB-04 (47')	13C060	28.4			79.0							
GSB-04 (50')	13C061											
GSB-04 (55')	13C062	41.9	0.0	17.2	82.8	42.2	40.6	70	46	24		
GSB-05 (5')	13C063											
GSB-05 (10')	13C064											
GSB-05 (15')	13C065	20.0	0.0	63.9	36.1			33	19	14		
GSB-05 (20')	13C066											
GSB-05 (25')	13C067											
GSB-05 (30')	13C068	28.2			9.5							
GSB-05 (35')	13C069	22.1			8.8							
GSB-05 (40')	13C070	24.5	1.6	76.5	21.9							
GSB-05 (45')	13C071											
GSB-06 (5')	13C072											
GSB-06 (10')	13C073	27.1	1.2	77.1	21.7							
GSB-06 (15')	13C074	23.2			7.9							
GSB-06 (20')	13C075	30.8			16.3							
GSB-06 (25')	13C076											
GSB-06 (27.5')	13C077											Note 1
GSB-06 (35)	13C078											Note 1
GSB-06 (40')	13C079											Note 1
GSB-07 (5')	13C080											
GSB-07 (10')	13C081											
GSB-07 (15')	13C082	19.1	0.0	62.5	37.5			34	17	17		
GSB-07 (20')	13C083											
GSB-07 (25')	13C084	25.8						37	19	18		
GSB-07 (30')	13C085	23.4	0.0	87.5	12.5							
GSB-07 (35')	13C086	28.2	0.7	56.4	42.9	13.8	29.1					
GSB-07 (40')	13C087	34.7										
GSB-07 (45')	13C088	16.6			17.5							
GSB-07 (50')	13C089											
GSB-08 (5')	13C090											
GSB-08 (10')	13C091	23.2	0.0	54.3	45.7			45	16	29		

Note:
1- No Sample

5-21-13
NJR

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.: 585

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		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
GSB-08 (15')	13C092	23.0			39.6							
GSB-08 (20')	13C093											
GSB-08 (25')	13C094	17.4	0.0	61.7	38.3	8.2	30.1					
GSB-08 (30')	13C095											
GSB-08 (35')	13C096	23.9			12.8							
GSB-08 (40')	13C097	32.0			17.4							
GSB-08 (45')	13C098	23.0			15.1							
GSB-08 (50')	13C099											
GSB-09 (4')	13C100											
GSB-09 (10')	13C101	21.9	6.2	80.5	13.3							
GSB-09 (15')	13C102	29.8			20.8							
GSB-09 (19' 6")	13C103											
GSB-10 (5')	13C104											
GSB-10 (10')	13C105											
GSB-10 (15')	13C106	23.4	0.0	49.7	50.3			47	20	27		
GSB-10 (20')	13C107	21.7			41.4							
GSB-10 (25')	13C108											
GSB-10 (32')	13C109	16.2			26.5							
GSB-10 (35')	13C110	24.4			16.3							
GSB-10 (40')	13C111											
GSB-10 (45')	13C112	20.4			16.0							
GSB-11 (5')	13C113											
GSB-11 (10')	13C114	34.3	1.4	50.0	48.6	8.8	39.8	47	17	30		
GSB-11 (15')	13C115	30.4						44	18	20		
GSB-11 (22')	13C116											
GSB-11 (25')	13C117	32.0	0.0	58.2	41.8			44	15	29		
GSB-11 (30')	13C118											
GSB-11 (35')	13C119	83.5	2.0	57.4	40.6	19.6	21.0	114	88	26		
GSB-11 (40')	13C120	168.5			49.5							
GSB-11 (45')	13C121	140.6			42.6							
GSB-12 (5')	13C122											
GSB-12 (10')	13C123	22.9			9.6							
GSB-12 (15')	13C124	31.8	0.3	84.0	15.7							

Notes:

5-21-13
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.: 585

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		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
GS-B-12 (20')	13C125											
GSB-12 (23')	13C126	11.5			22.7							
GSB-12 (25')	13C127											

[redacted]

Notes:

5-21-13
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.: 585

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		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
[redacted]												
GSB-3 (11.5-13.5')	13C161	19.8	0.0	59.8	40.2	12.4	27.8	36	18	18		Note 2&3
GSB-4 (30-32')	13C162											
GSB-7 (6.5-8.5')	13C163	21.5	0.0	66.9	33.1	8.5	24.6	33	21	12		Note 2&3
GSB-8 (36.5-38.5')	13C164	30.6	0.5	67.6	31.9	8.2	23.7	30	19	11		Note 2&3
GSB-8 (14-16')	13C165											
GSB-9	13C166											
GSB-10 (30-32')	13C167											
GSB-11 (20-22')	13C168											
GSB-12 (8-10')	13C169											
[redacted]												
WGS-TP-1	13C175	63.2	0.0	3.8	96.2	90.4	5.8	NP	NP	NP		
WGS-TP-2	13C176	58.5	0.0	26.5	73.5	68.2	5.3	NP	NP	NP		
WGS-TP-3	13C177											

Notes:
2 - See Triaxial Result
3 - Specific Gravity (see grain size distribution sheets)

5-21-13
MSR

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

Slurry Pond 3&4

Project No.: 618

Sample Information		Test Information										
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	Remarks
			Gravel Content	Sand Content	Fines Content	Silt Content	Clay Content	LL	PL	PI		
(-)	(-)	(%)	(%)	(%)	(%)	(%)	(%)	(-)	(-)	(-)	(-)	
SPT-106A, S-01 (8.0-10.0')	13K141		0	0.7	99.3	92.9	6.4					
SPT-106A, S-03 (16.0-18.0')	13K143		0	0.2	99.8			NP	NP	NP	ML	
SPT-106B, S-01 (6.0-8.0')	13K145		0	5.9	94.1							
SPT-106B, S-03 (14.0-16.0')	13K147		0	0.7	99.3	74.3	25.0					

[redacted]

Notes:

2-20-14
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

Slurry Pond 3&4

Project No.: 618

Sample Information		Test Information									Remarks
Site ID	Lab No.	Carbonate Content ASTM D 4373	pH ASTM D 4972		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-106A, S-01 (8.0-10.0')	13K141				116.7	93.1	25.3				
					115.6	84.0	37.6				
SPT-106B, S-01 (6.0-8.0')	13K145				105.2	71.0	48.2				
SPT-106B, S-03 (14.0-16.0')	13K147				98.6	63.4	55.5				
					90.0	49.3	82.5				
					94.1	55.7	68.9				
[redacted]											

Notes:

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
West 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-106, SS-01 (5.0-6.5')	13J137											
SPT-106, SS-02 (10.0-11.5')	13J138											
SPT-106, SS-03 (15.0-16.5')	13J139											
SPT-140, SS-04 (20.0-21.5')	13J140											
SPT-106, SS-05 (25.0-26.5')	13J141	31.4	0.0	22.3	77.7	11.3	66.4	63	20	43	CH	
SPT-106, ST-06 (30.0-31.5')	13J142											
SPT-106, SS-07 (35.0-36.5')	13J143											
SPT-106, SS-08 (40.0-41.5')	13J144	26.2			6.9							
SPT-106, SS-09 (45.0-46.5')	13J145											
SPT-106, SS-10 (50.0-51.5')	13J146											
SPT-106, SS-11 (55.0-56.5')	13J147											
SPT-106, ST-01 (60.0-62.0')	13J353		0	12.8	87.2	36.7	50.5	75	34	41	CH	
SPT-106, SS-12 (65.0-66.5')	13J148											
SPT-106, SS-13 (70.0-71.5')	13J149											
SPT-106, SS-14 (75.0-76.5')	13J150	42.2	8.8	17.2	74.0	32.7	41.3	69	31	38	CH	
SPT-106, SS-15 (80.0-81.5')	13J151											
SPT-106, SS-16 (85.0-86.5')	13J152											
SPT-106, SS-17 (90.0-91.5')	13J153											
SPT-106, SS-18 (95.0-96.5')	13J154	45.3	0.6	25.7	73.7	35.6	38.1	61	30	31	CH	
SPT-106, SS-19 (101.0-101.5')	13J155											

[redacted]

Notes:

1-16-14
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

West 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-106, ST-01 (60.0-62.0')	13J353				106.6	75.9	40.5				
[redacted]											

Notes:
1 - Method B was used which may not be very accurate..

2-9-14
NSR

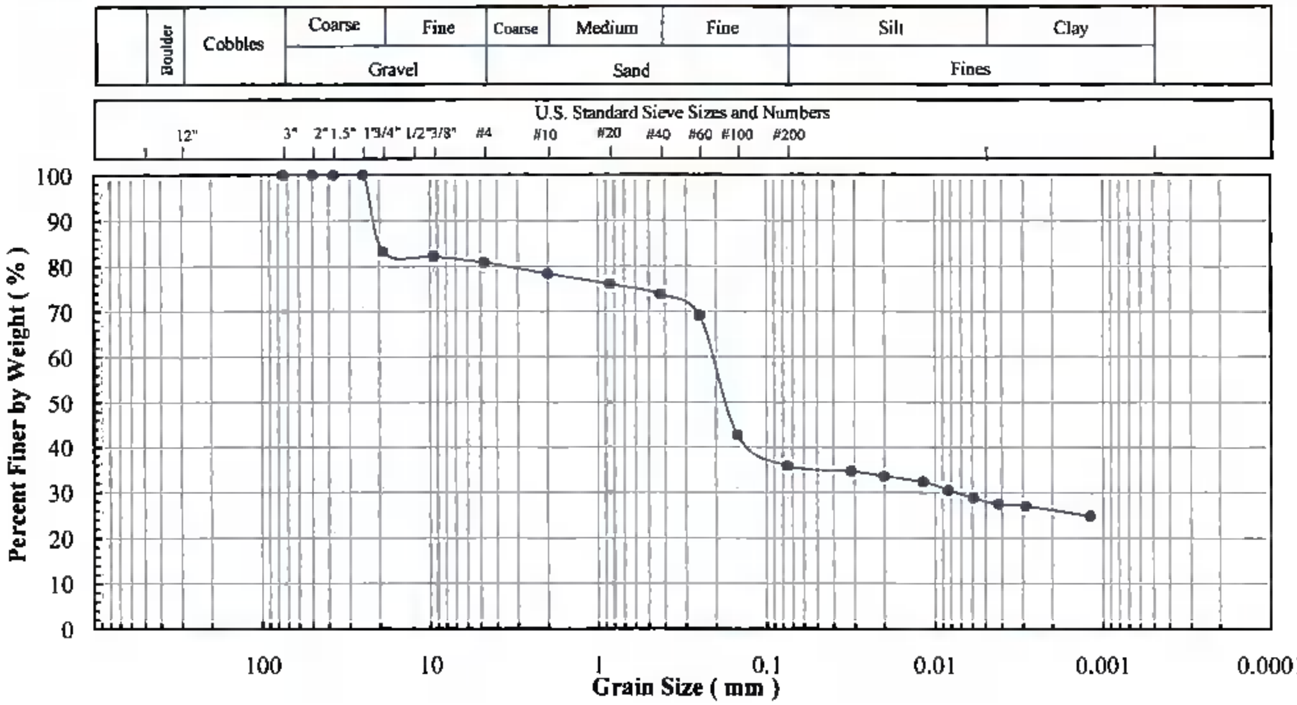
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-02 (5')
Lab Sample No:	13C036

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



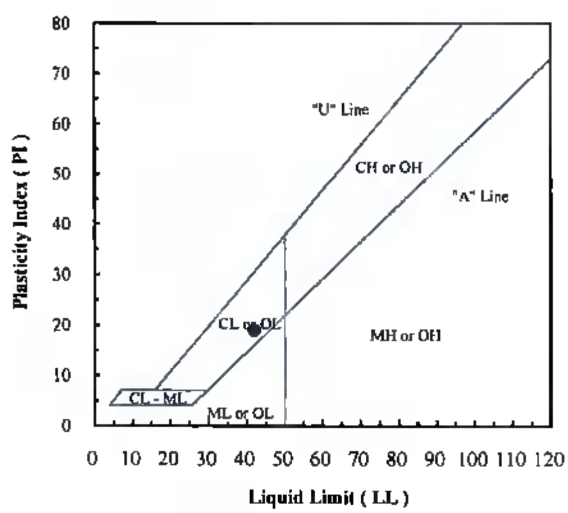
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	83.3
3/8"	9.5	82.2
#4	4.75	80.9
#10	2.00	78.5
#20	0.850	76.1
#40	0.425	73.8
#60	0.250	69.1
#100	0.150	42.7
#200	0.075	35.9

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0316	34.6
0.0117	32.2
0.0059	28.7
0.0029	27.0
0.0012	24.8

Gravel (%):	19.1
Sand (%):	45.0
Fines (%):	35.9
Silt (%):	7.9
Clay (%):	28.0

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
GSB-02 (5')	13C036	32.9	35.9	42	23	19	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

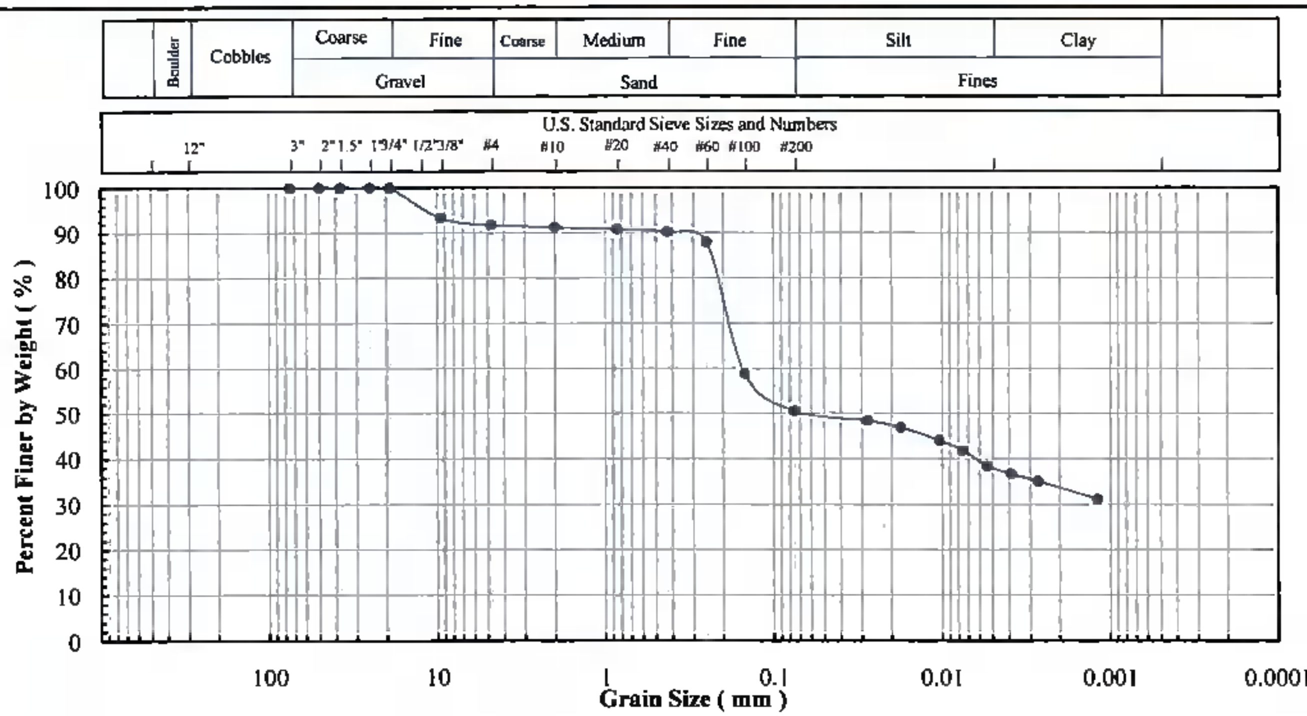
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-03 (10')
Lab Sample No:	13C043

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



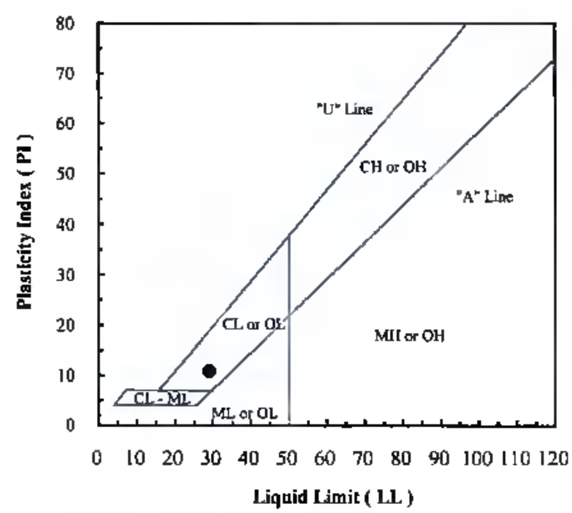
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	93.4
#4	4.75	91.8
#10	2.00	91.3
#20	0.850	90.8
#40	0.425	90.2
#60	0.250	87.9
#100	0.150	58.9
#200	0.075	50.5

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0277	48.3
0.0104	44.0
0.0054	38.4
0.0027	35.0
0.0012	31.2

Gravel (%):	8.2
Sand (%):	41.3
Fines (%):	50.5
Silt (%):	12.5
Clay (%):	38.0

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
GSB-03 (10')	13C043	21.6	50.5	29	18	11	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

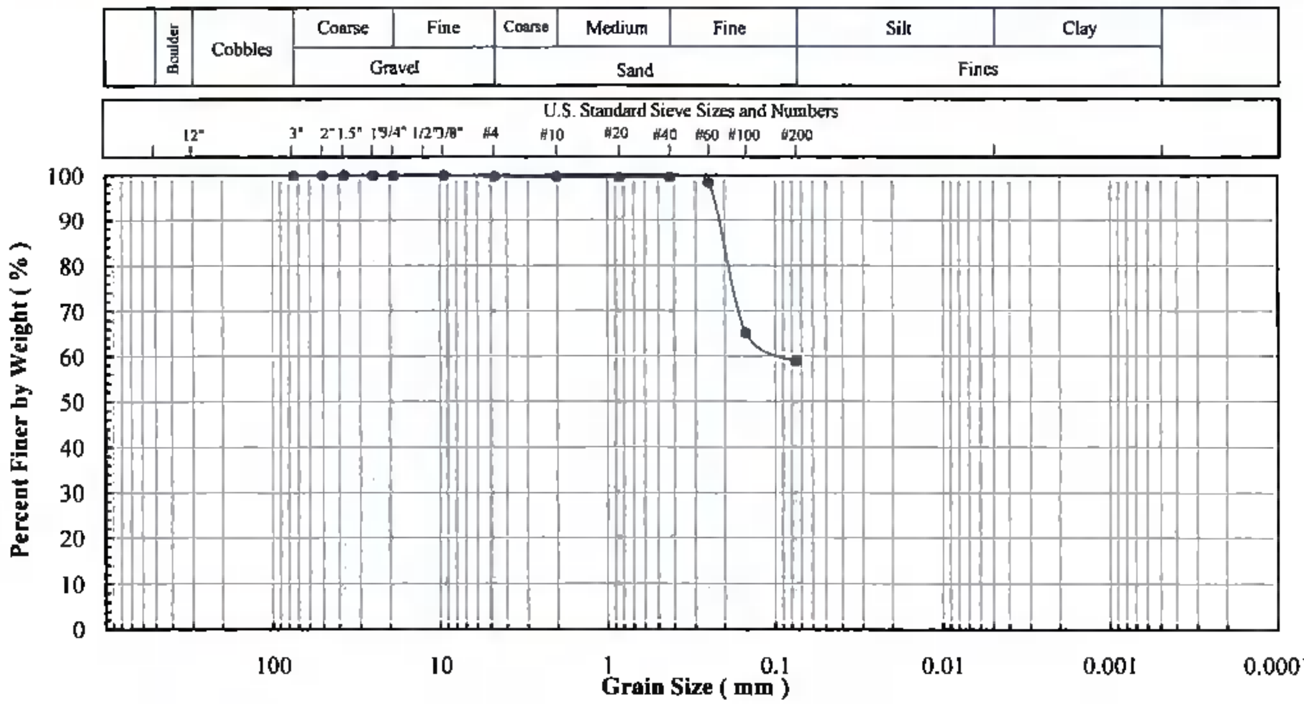
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-B03 (30')
Lab Sample No:	13C047

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



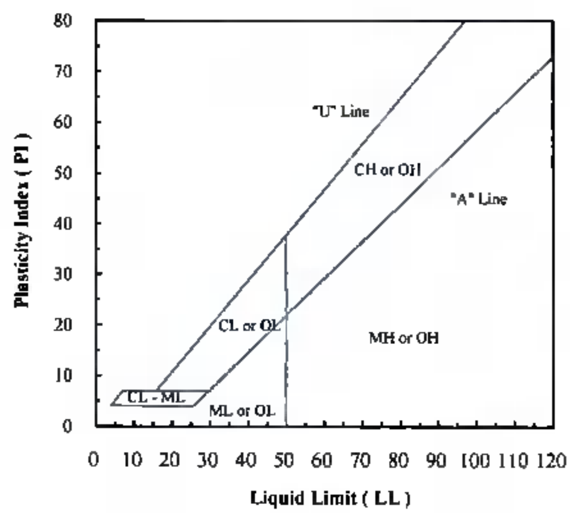
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.8
#10	2.00	99.7
#20	0.850	99.6
#40	0.425	99.6
#60	0.250	98.4
#100	0.150	65.2
#200	0.075	58.8

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	0.2
Sand (%):	41.0
Fines (%):	58.8
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
GSB-B03 (30')	13C047	58.0	58.8				

Note(s):

4-25-13
NSR

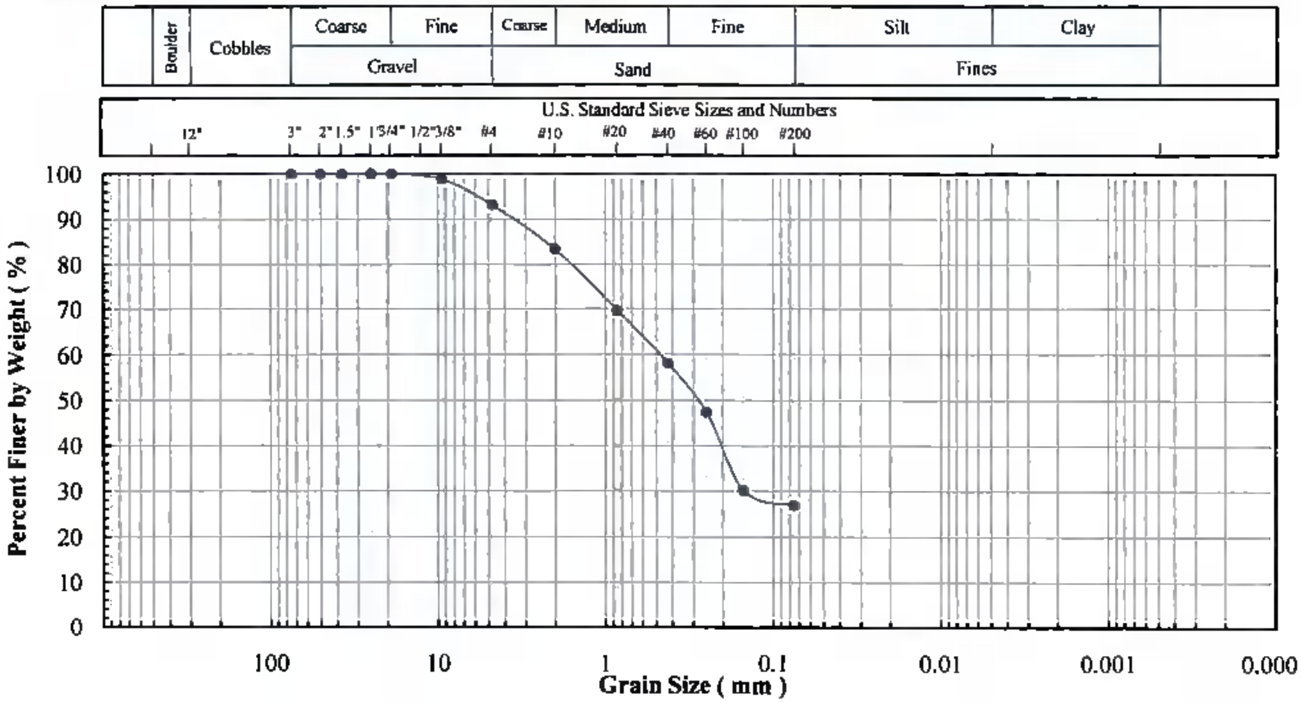
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-03 (40')
Lab Sample No:	13C049

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



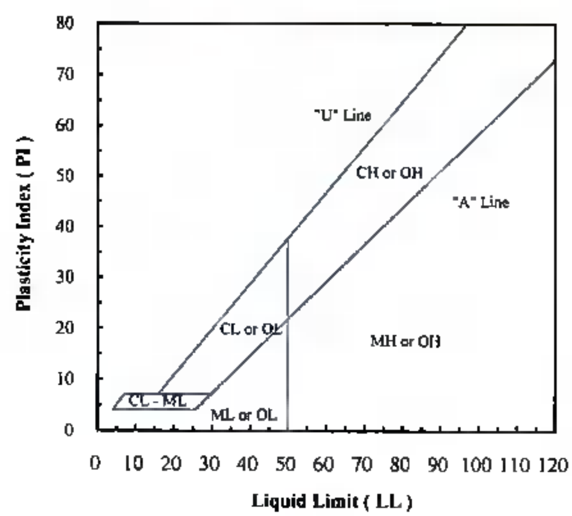
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	98.9
#4	4.75	93.2
#10	2.00	83.4
#20	0.850	69.7
#40	0.425	58.1
#60	0.250	47.4
#100	0.150	30.1
#200	0.075	26.9

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	6.8
Sand (%):	66.3
Fines (%):	26.9
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
GSB-03 (40')	13C049	33.9	26.9				

Note(s):

4-25-13 NSR

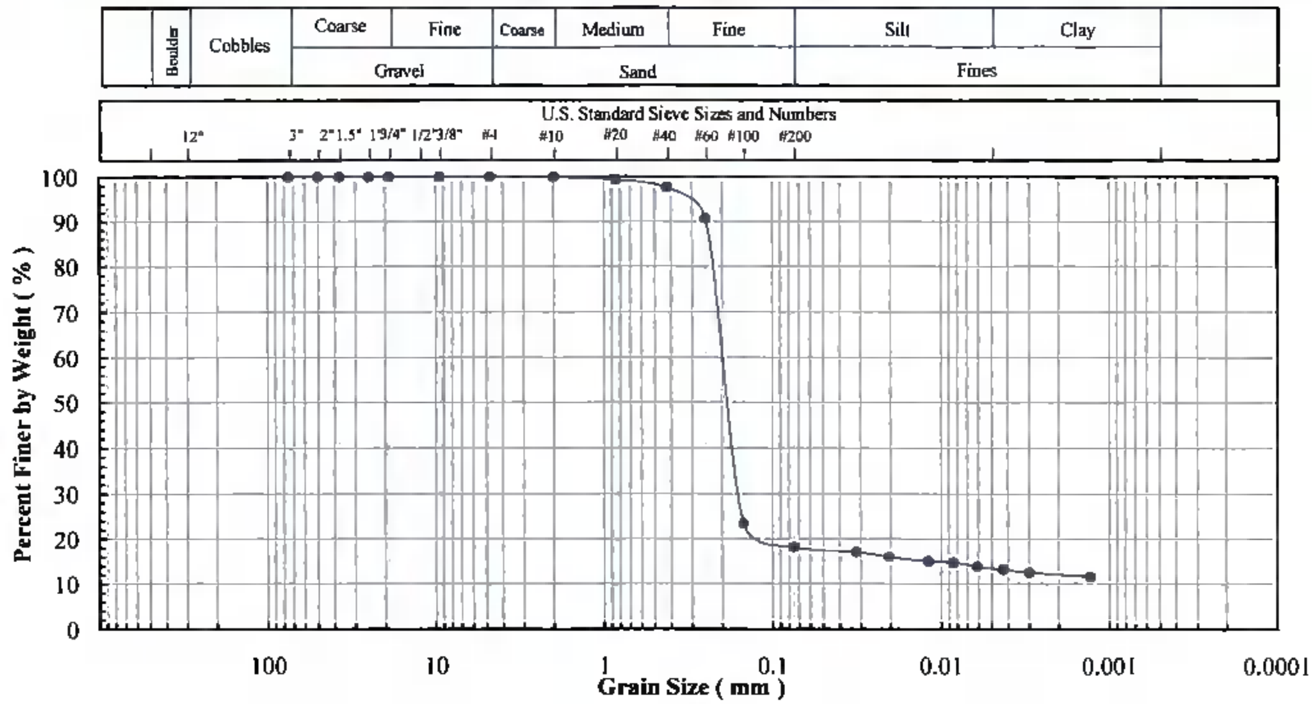
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-04 (10')
Lab Sample No:	13C052

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



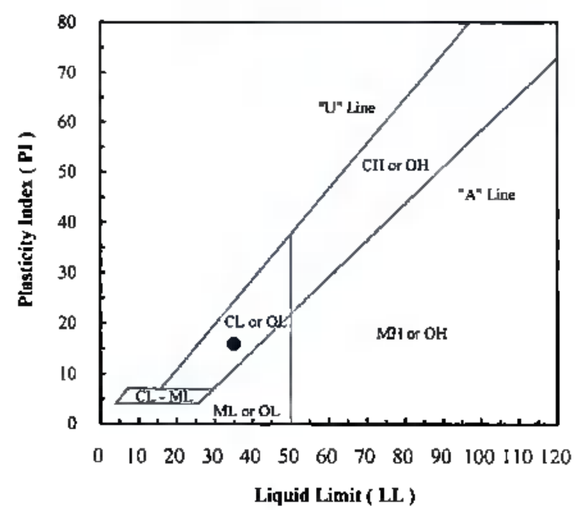
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.4
#40	0.425	97.6
#60	0.250	90.7
#100	0.150	23.5
#200	0.075	18.2

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0322	17.1
0.0119	15.0
0.0061	13.8
0.0030	12.5
0.0013	11.6

Gravel (%):	
Sand (%):	81.8
Fines (%):	18.2
Silt (%):	4.8
Clay (%):	13.4

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
GSB-04 (10')	13C052	19.9	18.2	35	19	16	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

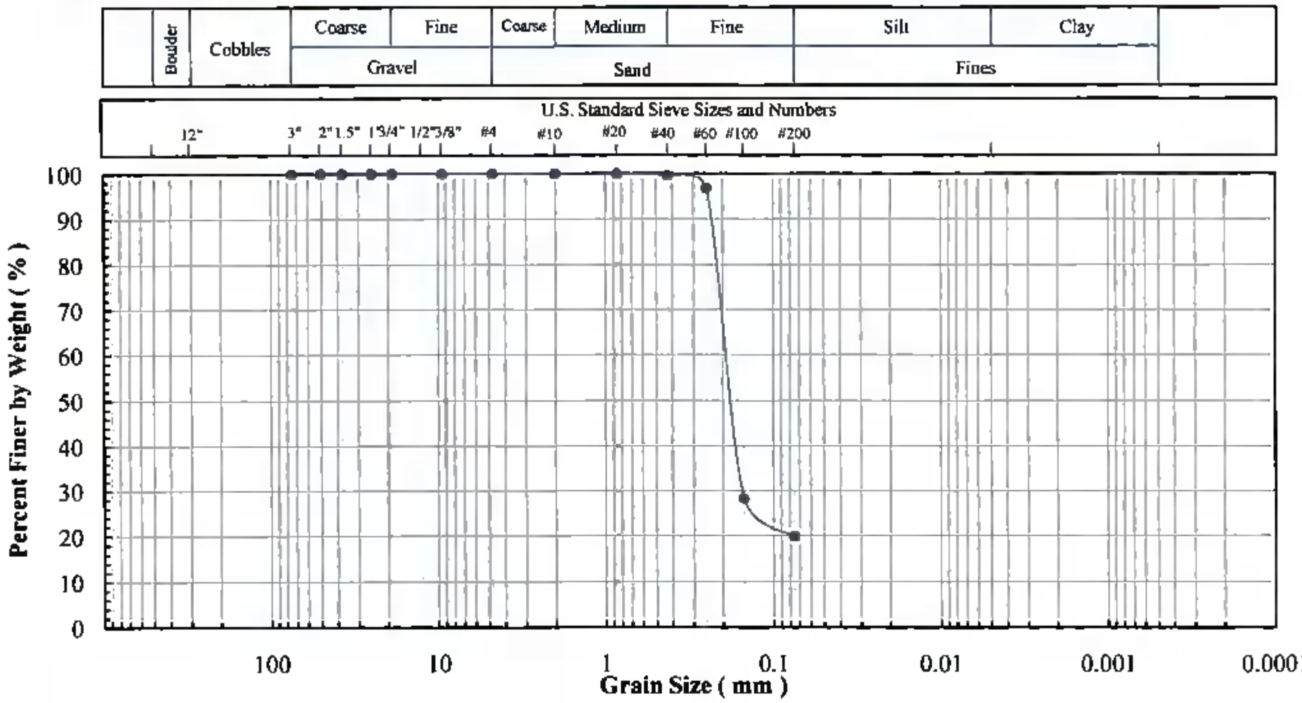
4-25-13
NSR

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:** 585 **Client Sample ID:** GSB-04 (32') **Lab Sample No:** 13C056

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



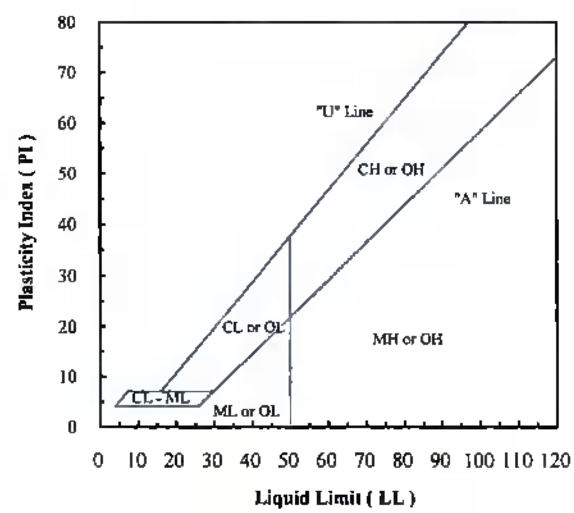
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.6
#60	0.250	96.8
#100	0.150	28.3
#200	0.075	19.9

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	80.1
Fines (%):	19.9
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
GSB-04 (32')	13C056	29.1	19.9				

Note(s):

4-25-13
NSR

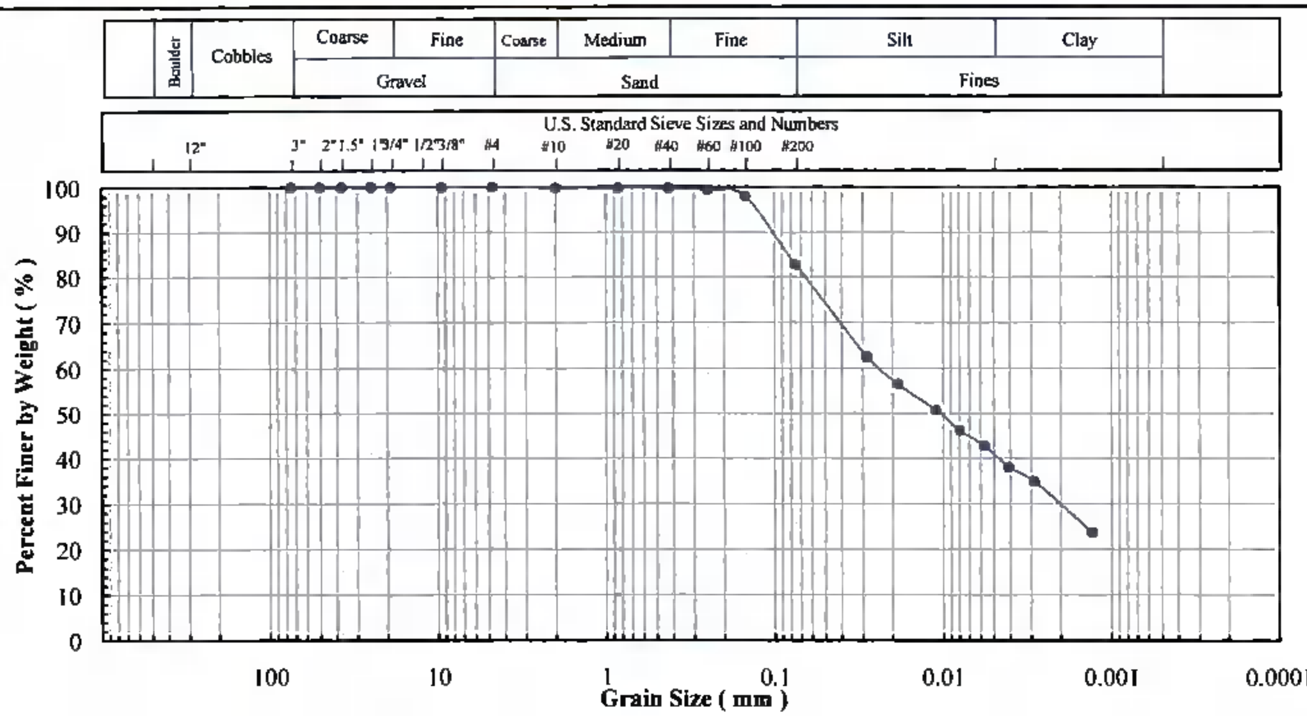
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-04 (55')
Lab Sample No:	13C062

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



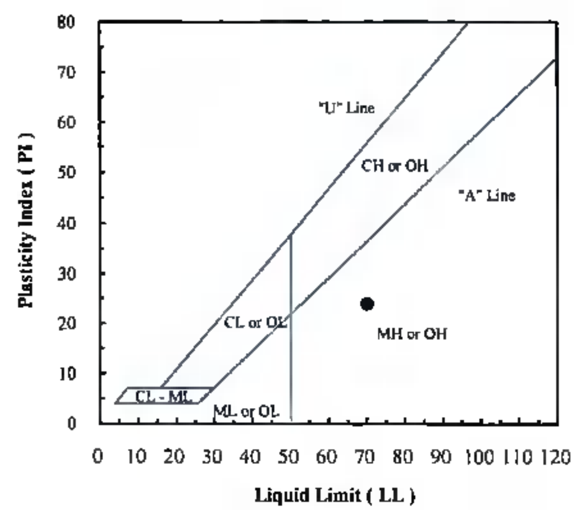
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.8
#40	0.425	99.7
#60	0.250	99.3
#100	0.150	98.0
#200	0.075	82.8

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0284	62.6
0.0110	50.7
0.0057	42.7
0.0029	34.8
0.0013	23.6

Gravel (%):	
Sand (%):	17.2
Fines (%):	82.8
Silt (%):	42.2
Clay (%):	40.6

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
GSB-04 (55')	13C062	41.9	82.8	70	46	24	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

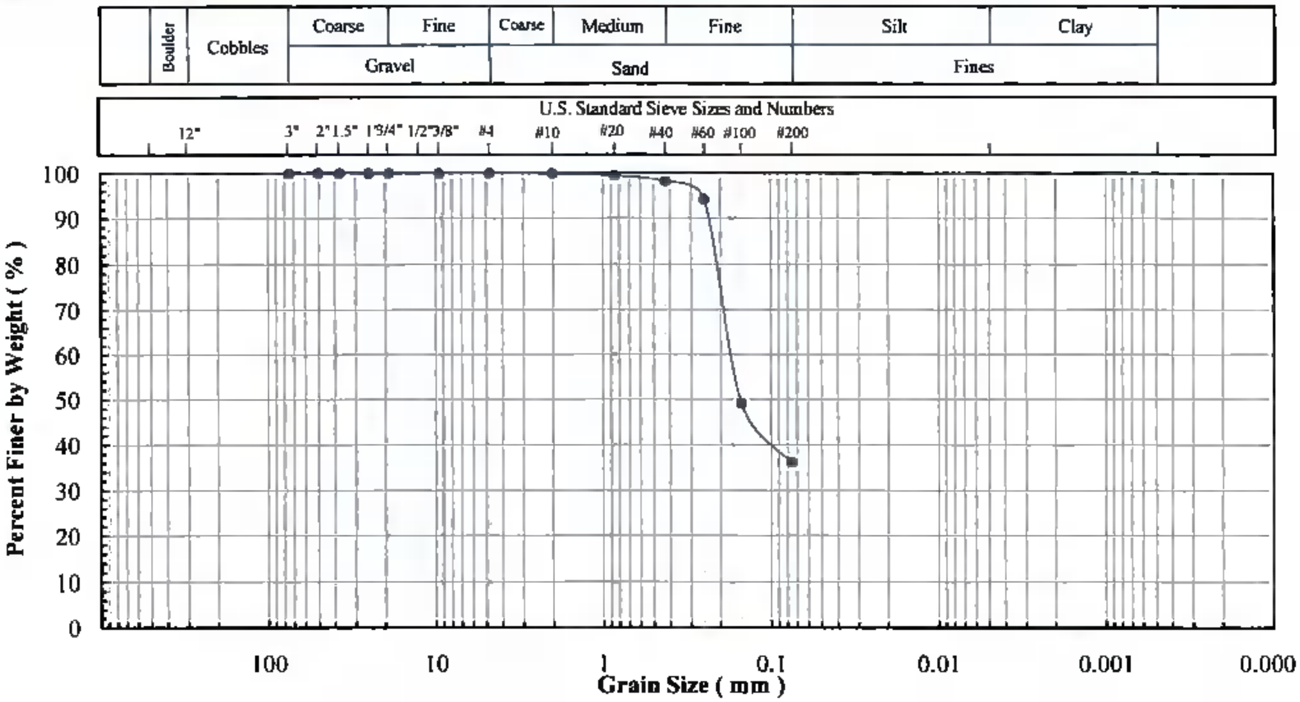
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-05 (15')
Lab Sample No:	13C065

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



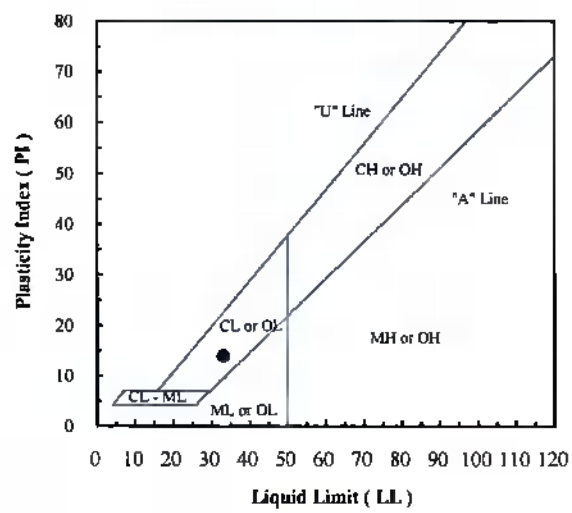
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.4
#40	0.425	98.2
#60	0.250	94.2
#100	0.150	49.1
#200	0.075	36.1

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	63.9
Fines (%):	36.1
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
GSB-05 (15')	13C065	20.0	36.1	33	19	14	

Note(s):

4-25-13
NSR

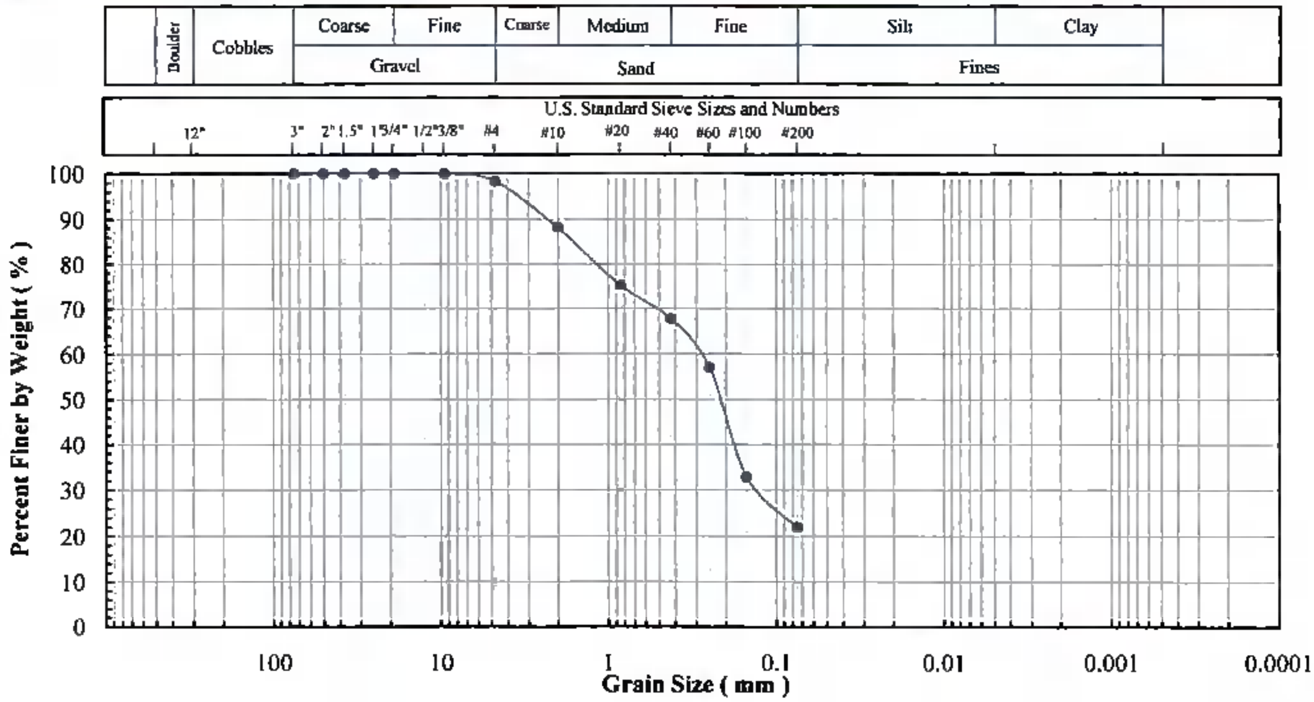
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-05 (40')
Lab Sample No:	13C070

ASTM C 136, D 422, D 854, D 1140, D2216, D 2487, D4319

SOIL INDEX PROPERTIES

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



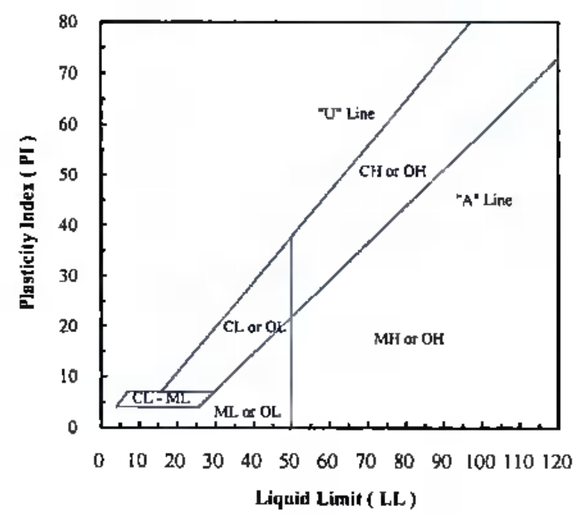
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	98.4
#10	2.00	88.2
#20	0.850	75.3
#40	0.425	67.8
#60	0.250	57.0
#100	0.150	32.8
#200	0.075	21.9

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	1.6
Sand (%):	76.5
Fines (%):	21.9
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
GSB-05 (40')	13C070	24.5	21.9				

Note(s):

4-25-13
NSR

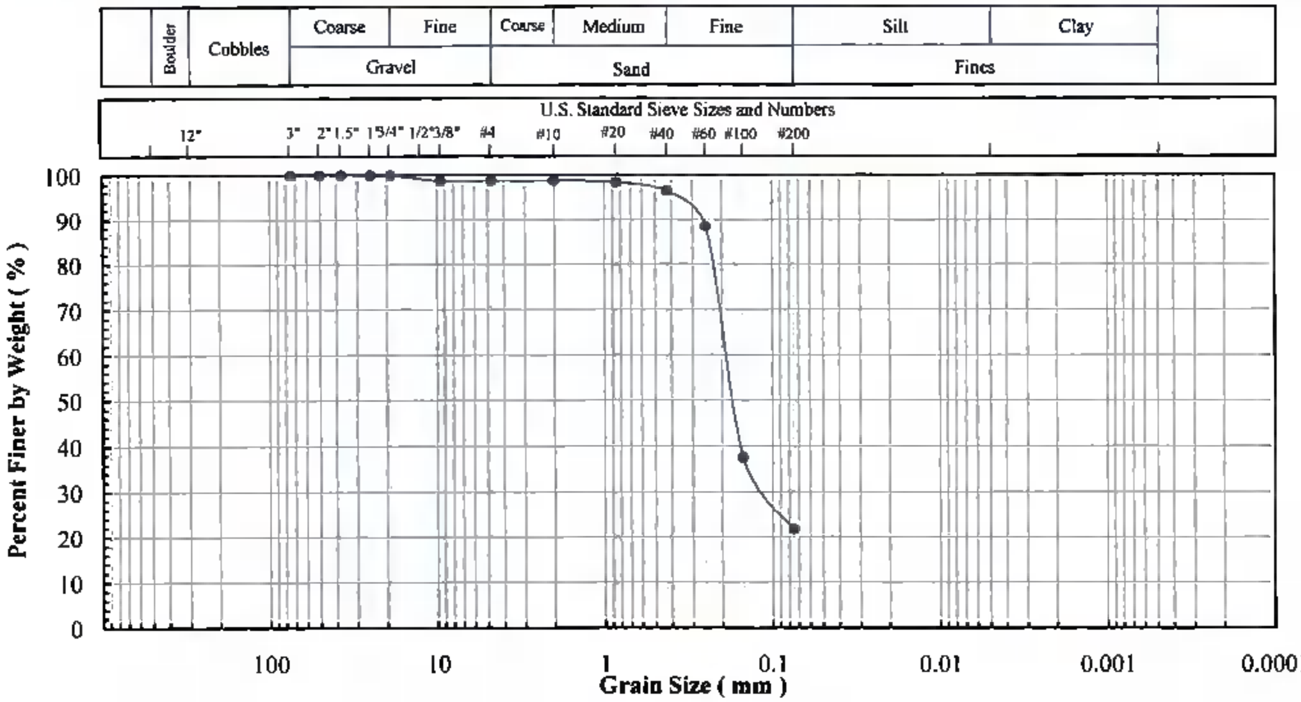
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-06 (10')
Lab Sample No:	13C073

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



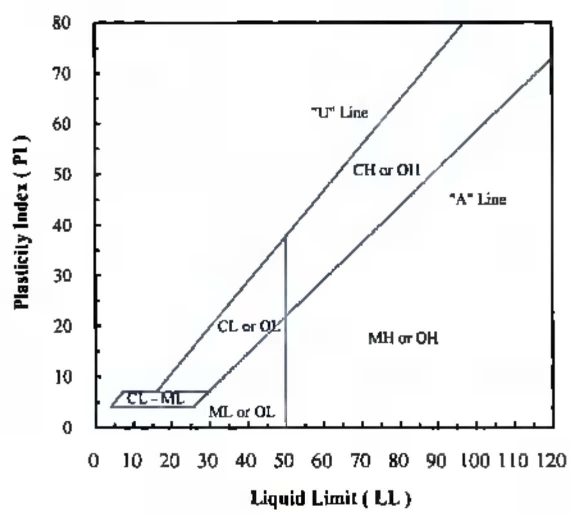
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	98.8
#4	4.75	98.8
#10	2.00	98.8
#20	0.850	98.4
#40	0.425	96.4
#60	0.250	88.6
#100	0.150	37.6
#200	0.075	21.7

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	1.2
Sand (%):	77.1
Fines (%):	21.7
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
GSB-06 (10')	13C073	27.1	21.7				

Note(s):

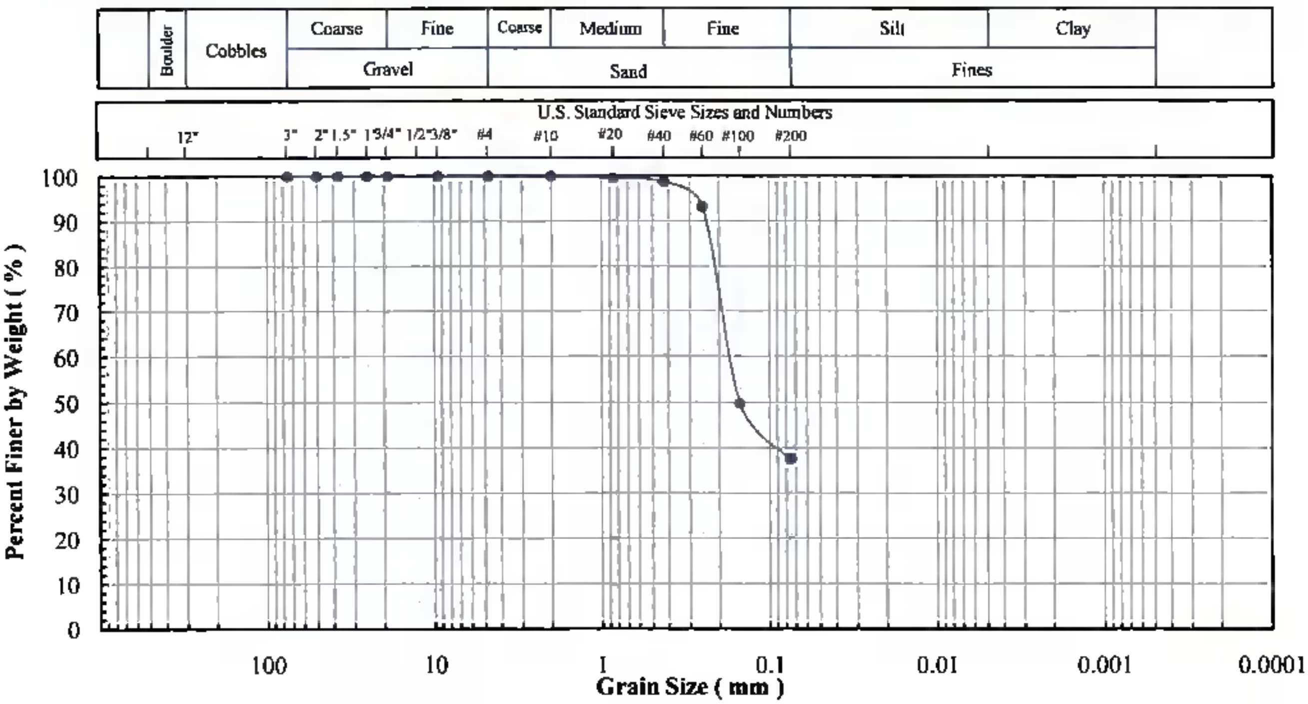
4-25-13
NSR

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:** 585 **Client Sample ID:** GSB-07 (15') **Lab Sample No:** 13C082

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



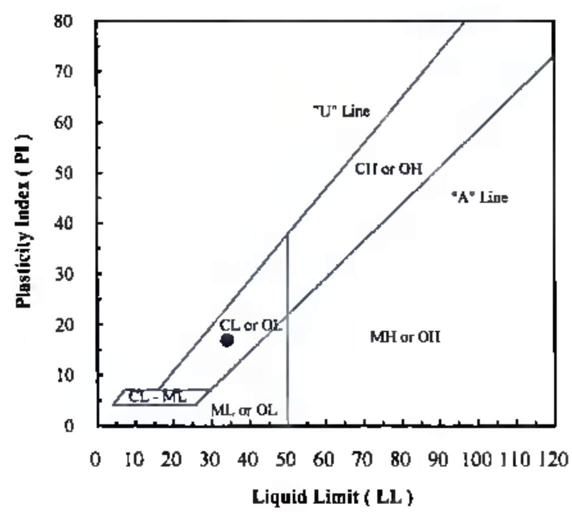
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.6
#40	0.425	98.7
#60	0.250	93.1
#100	0.150	49.8
#200	0.075	37.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	62.5
Fines (%):	37.5
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
GSB-07 (15')	13C082	19.1	37.5	34	17	17	

Note(s):

4-25-13 NSR

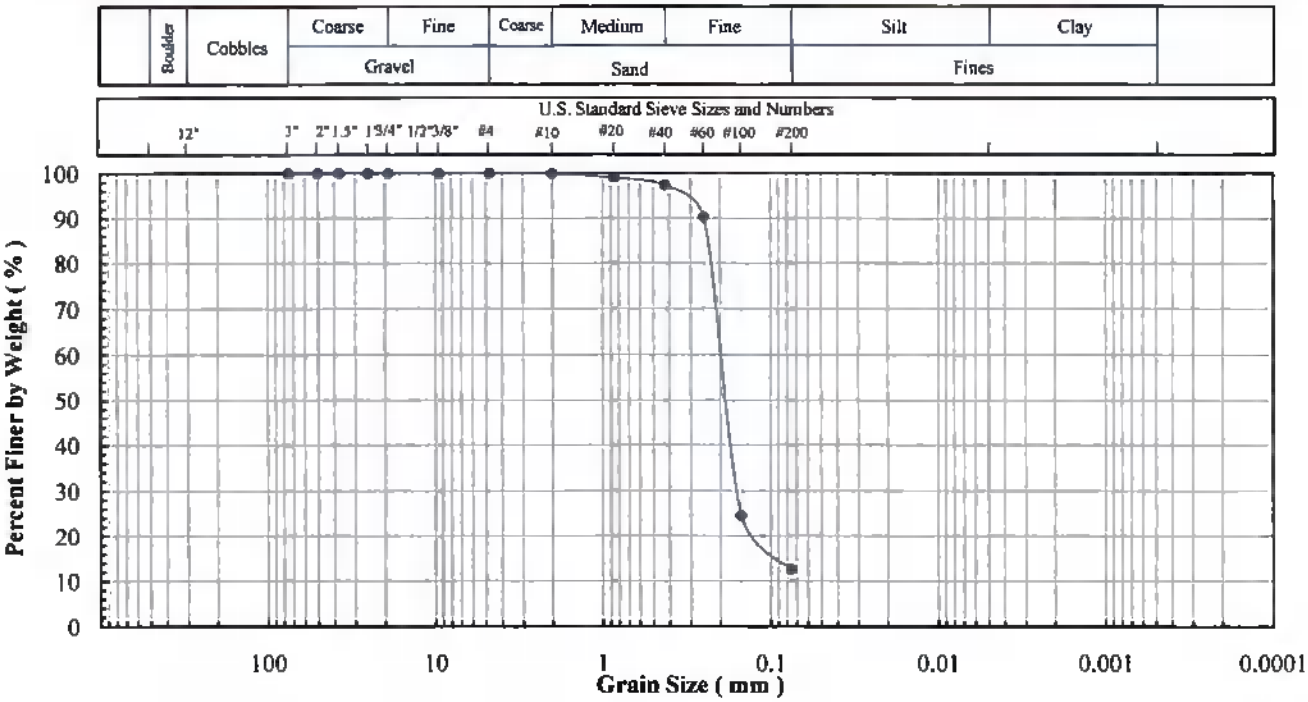
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-07 (30')
Lab Sample No:	13C085

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



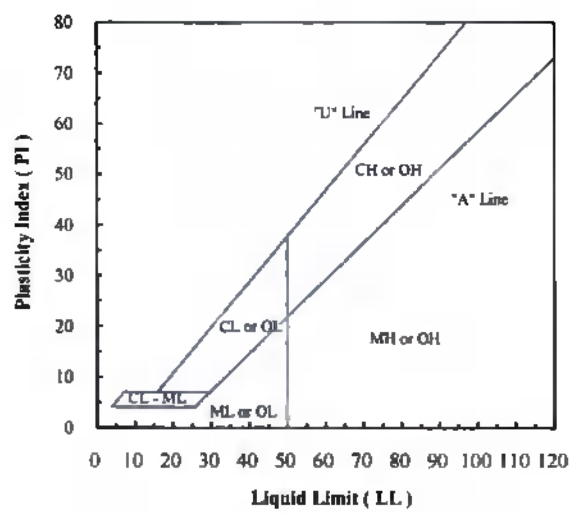
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.1
#40	0.425	97.3
#60	0.250	90.3
#100	0.150	24.6
#200	0.075	12.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	87.5
Fines (%):	12.5
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
GSB-07 (30')	13C085	23.4	12.5				

Note(s):

4-25-13 NSR

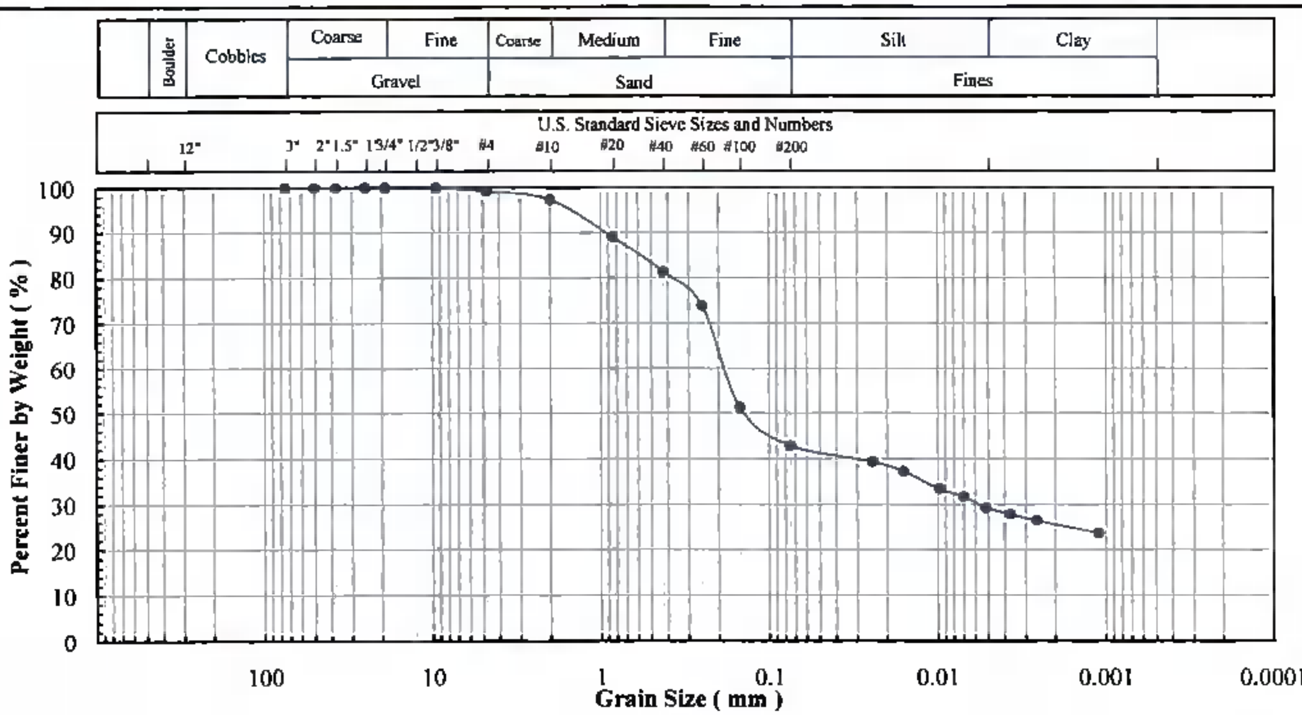
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-07 (35')
Lab Sample No:	13C086

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



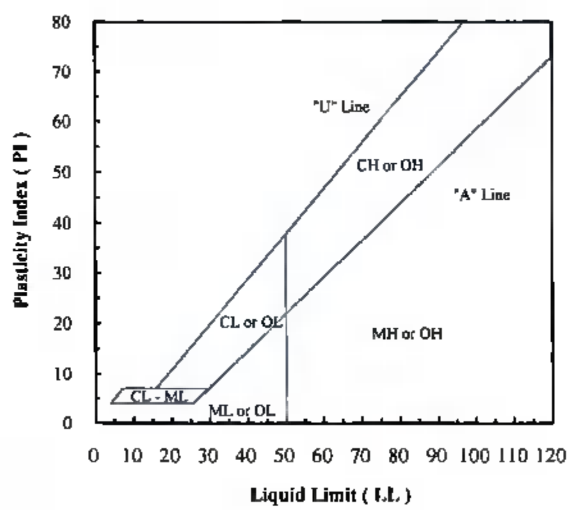
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.3
#10	2.00	97.4
#20	0.850	89.1
#40	0.425	81.4
#60	0.250	73.9
#100	0.150	51.2
#200	0.075	42.9

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0244	39.4
0.0098	33.4
0.0052	29.3
0.0026	26.4
0.0011	23.6

Gravel (%):	0.7
Sand (%):	56.4
Fines (%):	42.9
Silt (%):	13.8
Clay (%):	29.1

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
GSB-07 (35')	13C086	28.2	42.9				

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

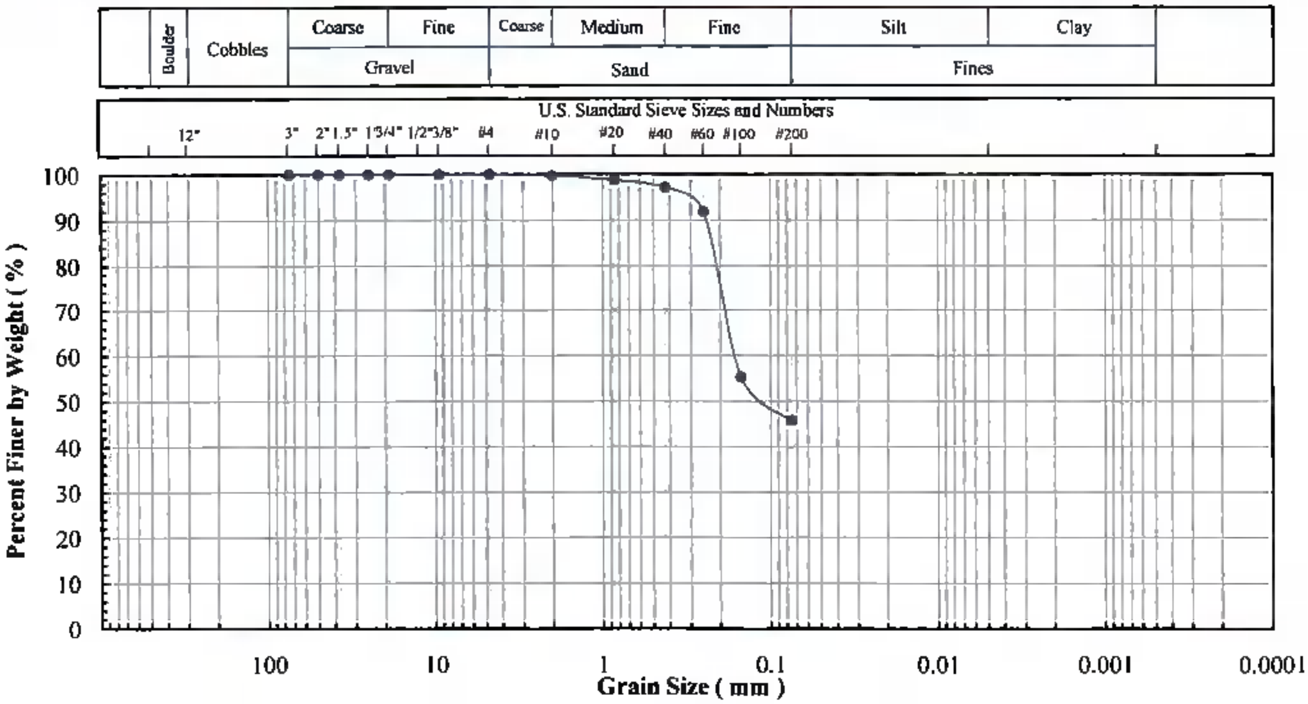
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-08 (10')
Lab Sample No:	13C091

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



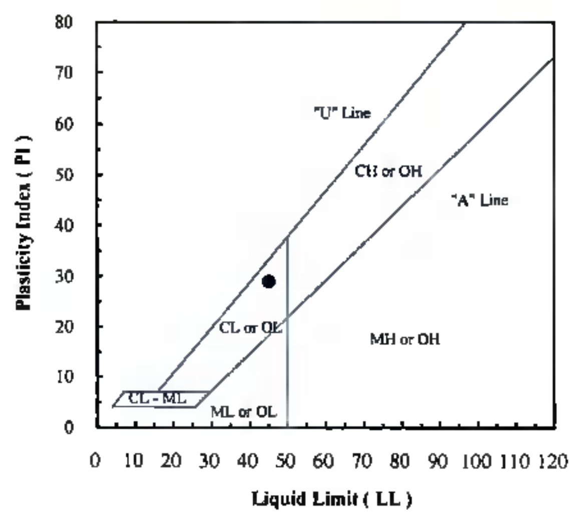
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	98.8
#40	0.425	97.1
#60	0.250	91.9
#100	0.150	55.3
#200	0.075	45.7

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	54.3
Fines (%):	45.7
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
GSB-08 (10')	13C091	23.2	45.7	45	16	29	

Note(s):

11-25-13
NSR

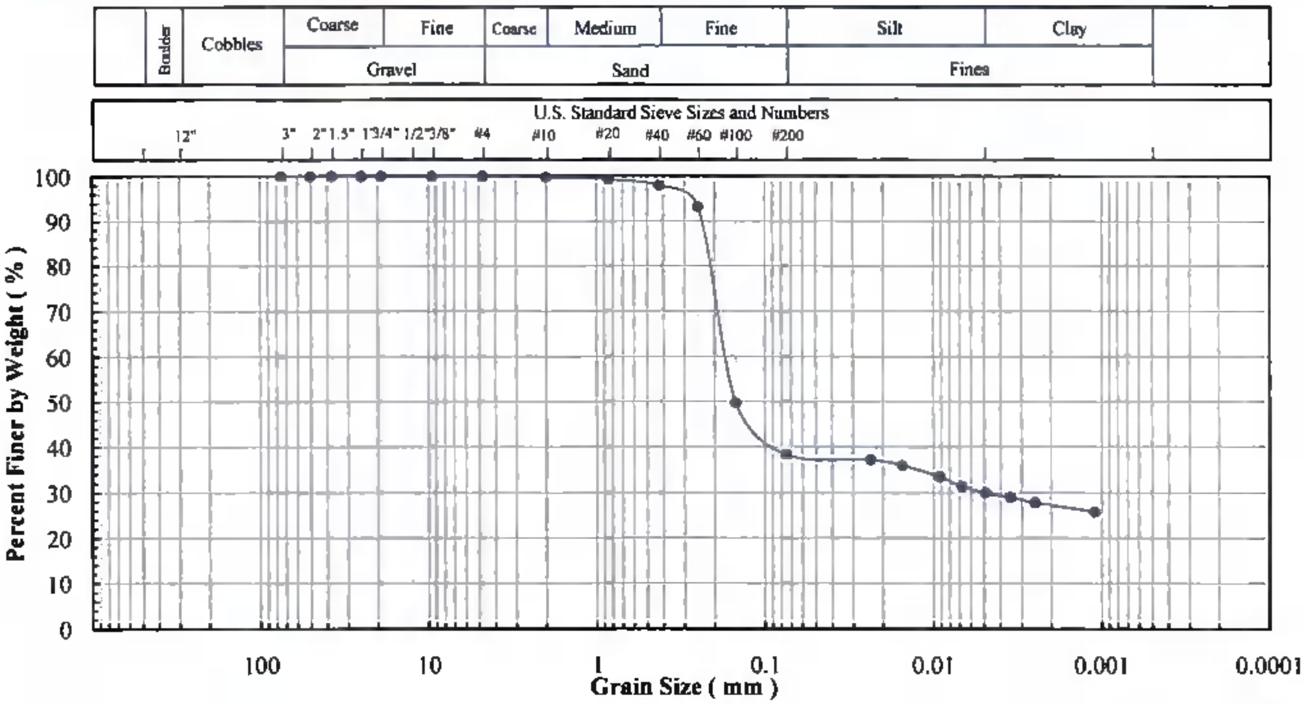
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-08 (25')
Lab Sample No:	13C094

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



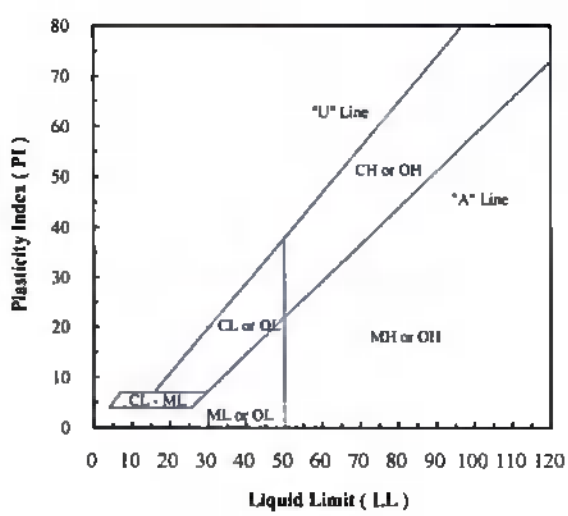
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.4
#40	0.425	98.0
#60	0.250	93.3
#100	0.150	49.7
#200	0.075	38.3

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0238	37.1
0.0092	33.4
0.0049	30.0
0.0025	27.8
0.0011	25.8

Gravel (%):	
Sand (%):	61.7
Fines (%):	38.3
Silt (%):	8.2
Clay (%):	30.1

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
GSB-08 (25')	13C094	17.4	38.3				

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

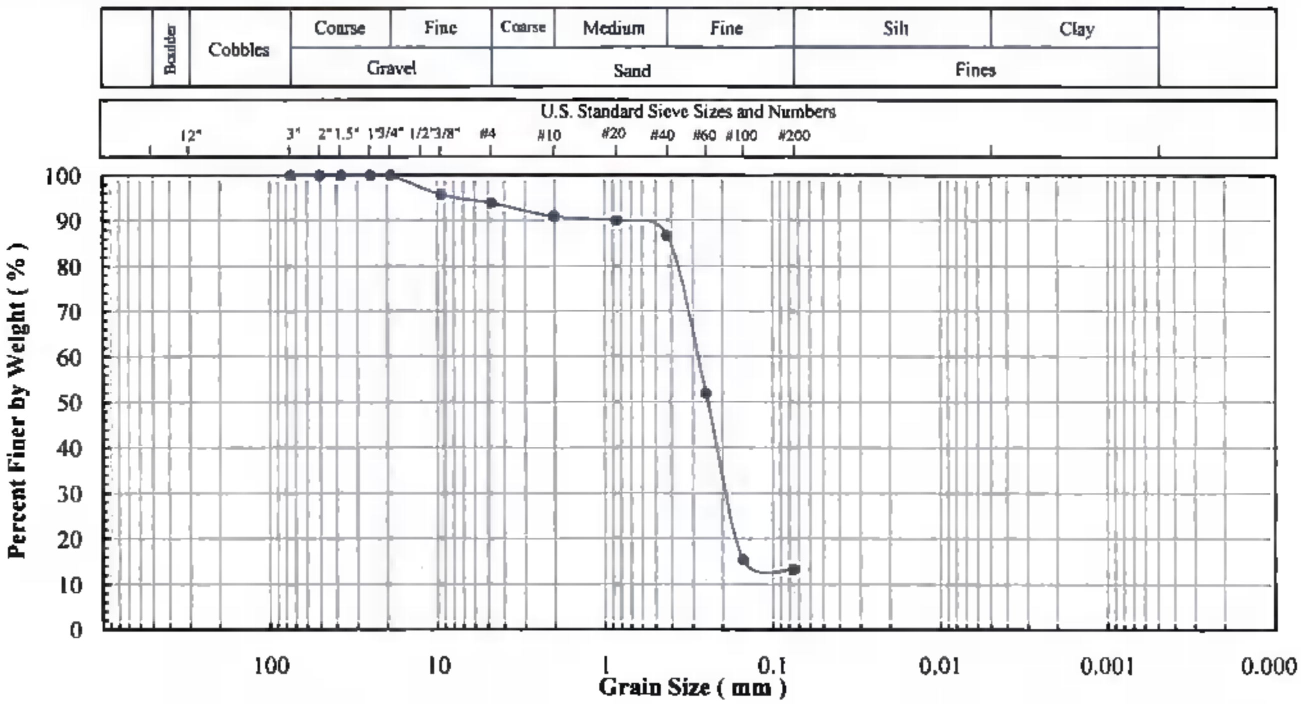
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-09 (10')
Lab Sample No:	13C101

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



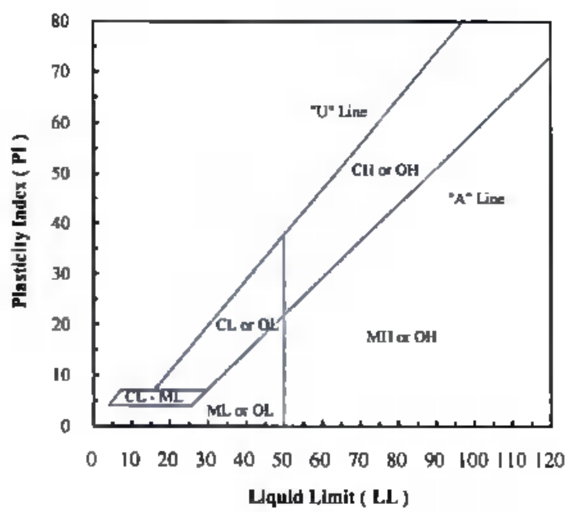
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	95.7
#4	4.75	93.8
#10	2.00	90.9
#20	0.850	89.9
#40	0.425	86.7
#60	0.250	51.8
#100	0.150	15.4
#200	0.075	13.3

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	6.2
Sand (%):	80.5
Fines (%):	13.3
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
GSB-09 (10')	13C101	21.9	13.3				

Note(s):

4-25-13
NSK

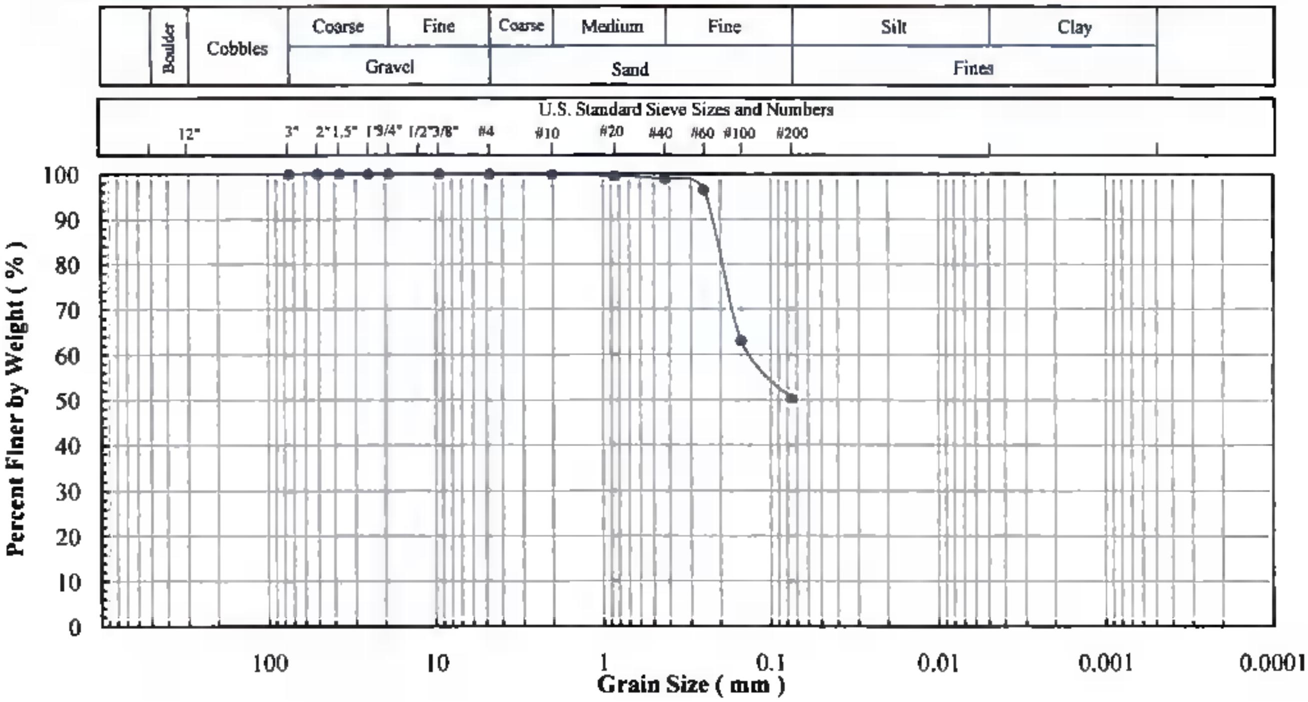
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7637 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-10 (15')
Lab Sample No:	13C106

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



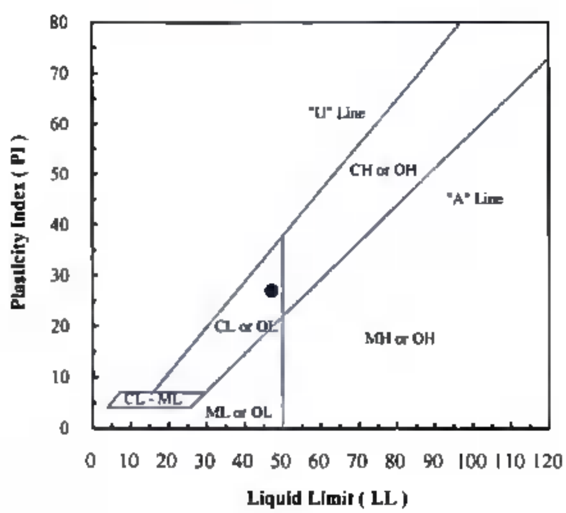
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.6
#40	0.425	98.9
#60	0.250	96.4
#100	0.150	63.1
#200	0.075	50.3

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	49.7
Fines (%):	50.3
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
GSB-10 (15')	13C106	23.4	50.3	47	20	27	

Note(s):

4-25-13
NSR

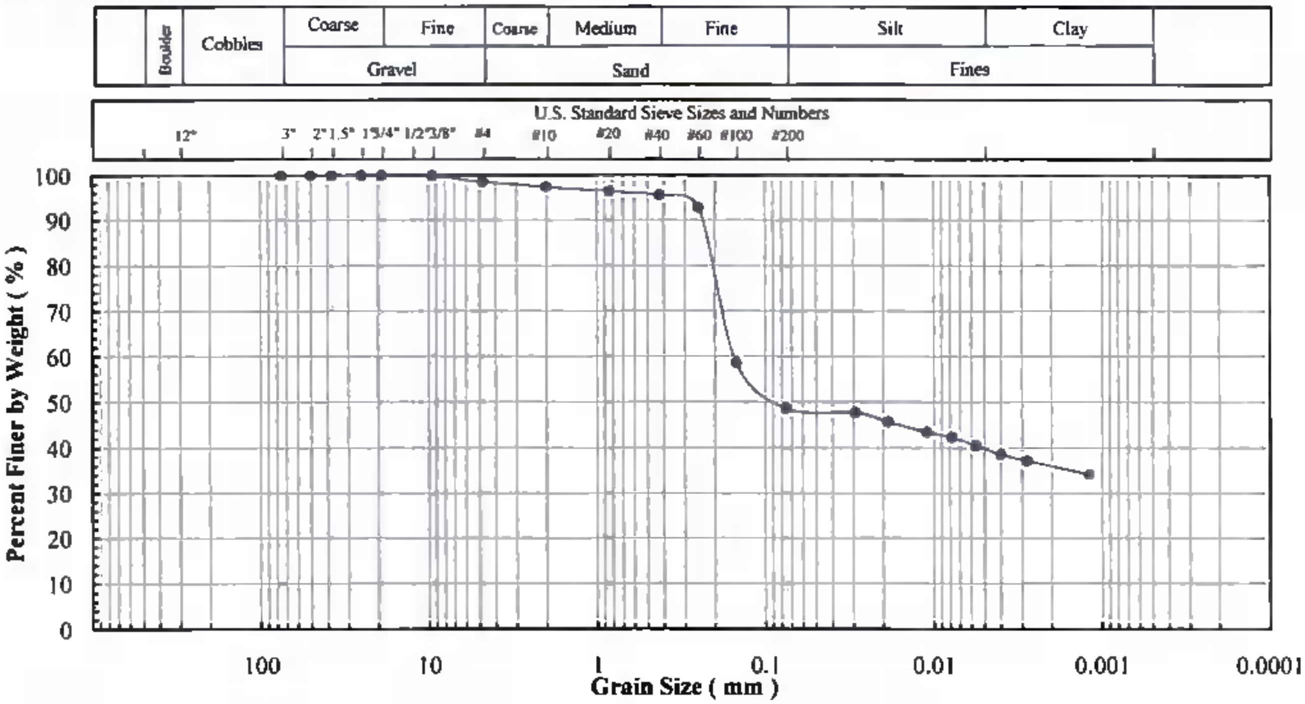
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-11 (10')
Lab Sample No:	13C114

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



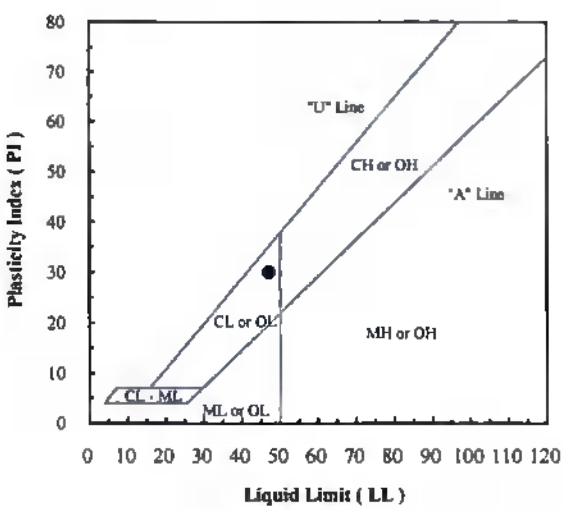
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	98.6
#10	2.00	97.5
#20	0.850	96.5
#40	0.425	95.6
#60	0.250	92.8
#100	0.150	58.6
#200	0.075	48.6

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0293	47.7
0.0110	43.4
0.0056	40.5
0.0028	37.1
0.0012	34.2

Gravel (%):	1.4
Sand (%):	50.0
Fines (%):	48.6
Silt (%):	8.8
Clay (%):	39.8

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
GSB-11 (10')	13C114	34.3	48.6	47	17	30	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
NSR

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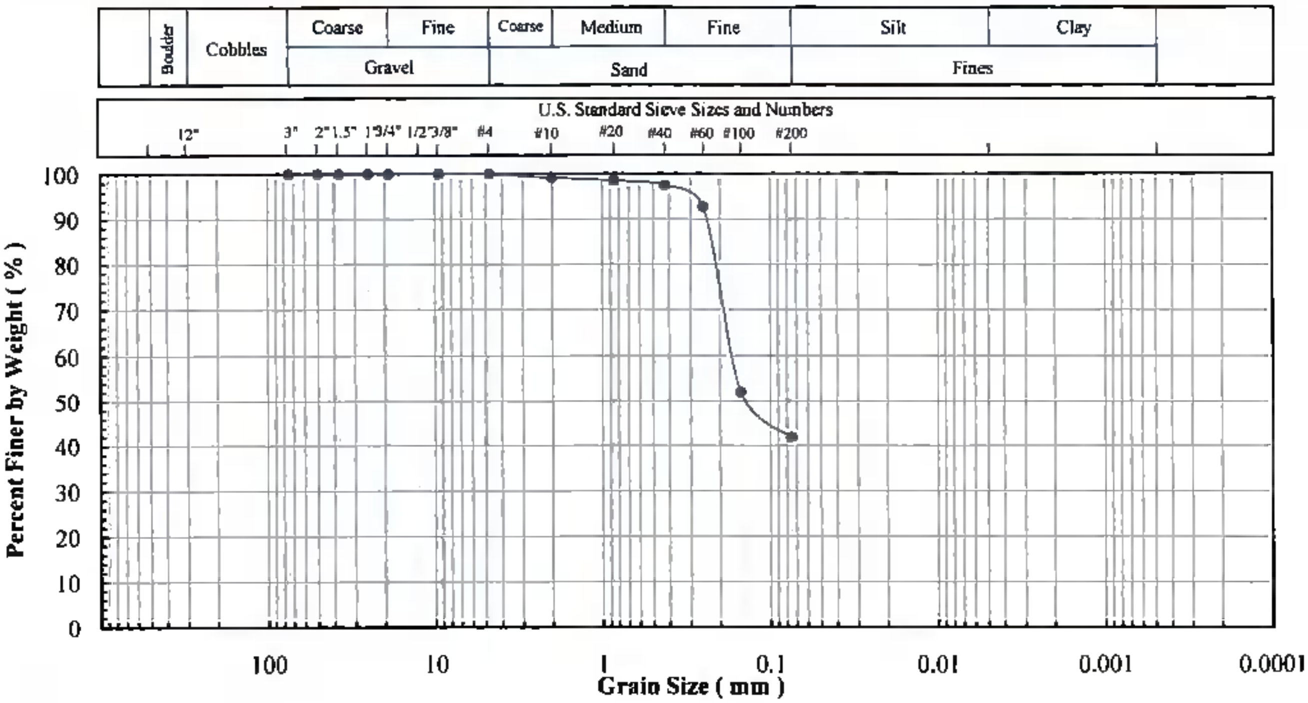
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-11 (25')
Lab Sample No:	13C117

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



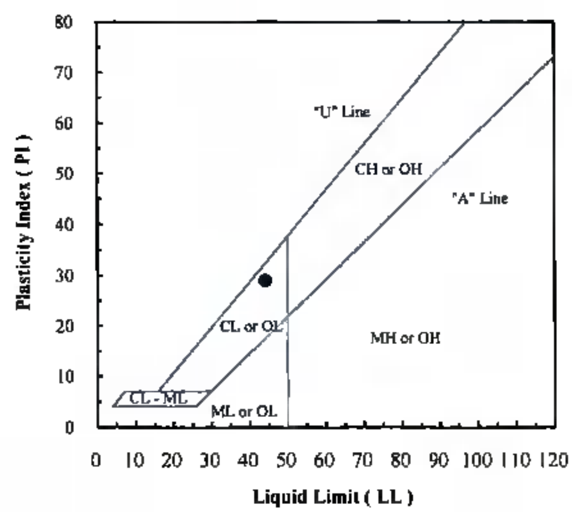
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.2
#20	0.850	98.5
#40	0.425	97.5
#60	0.250	92.7
#100	0.150	52.0
#200	0.075	41.8

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	58.2
Fines (%):	41.8
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
GSB-11 (25')	13C117	32.0	41.8	44	15	29	

Note(s):

4-25-13
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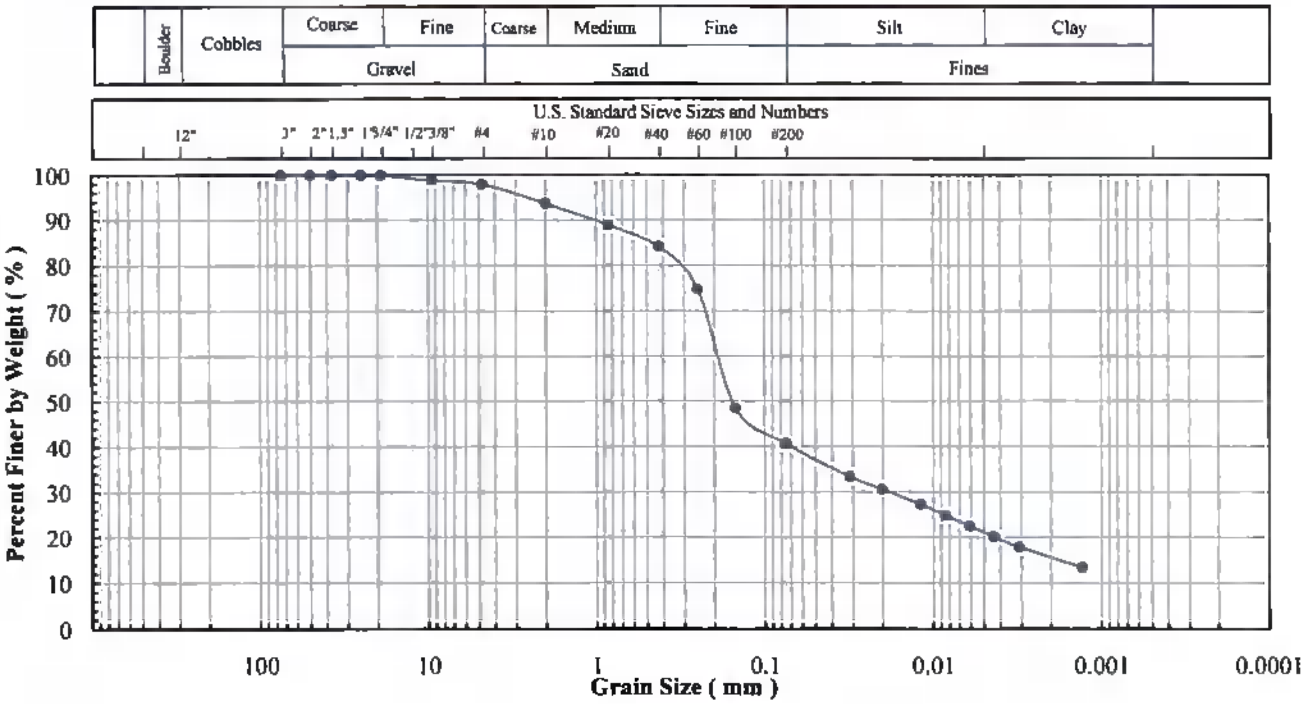
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-11 (35')
Lab Sample No:	13C119

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



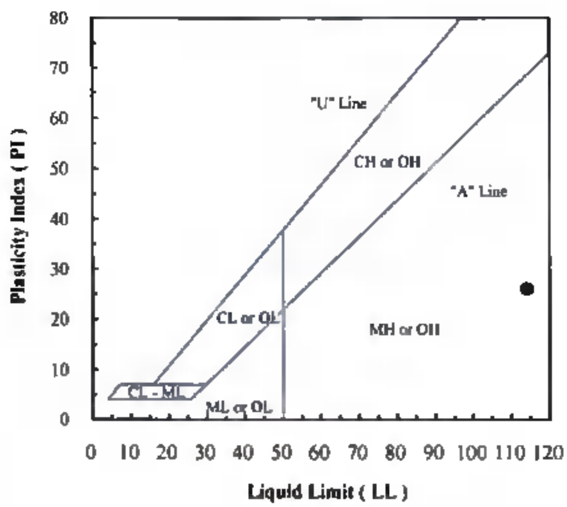
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	98.9
#4	4.75	98.0
#10	2.00	93.7
#20	0.850	88.9
#40	0.425	84.1
#60	0.250	74.9
#100	0.150	48.5
#200	0.075	40.6

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0315	33.4
0.0119	27.3
0.0061	22.5
0.0031	17.9
0.0013	13.3

Gravel (%):	2.0
Sand (%):	57.4
Fines (%):	40.6
Silt (%):	19.6
Clay (%):	21.0

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
GSB-11 (35')	13C119	83.5	40.6	114	88	26	

Note(s): An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.

4-25-13
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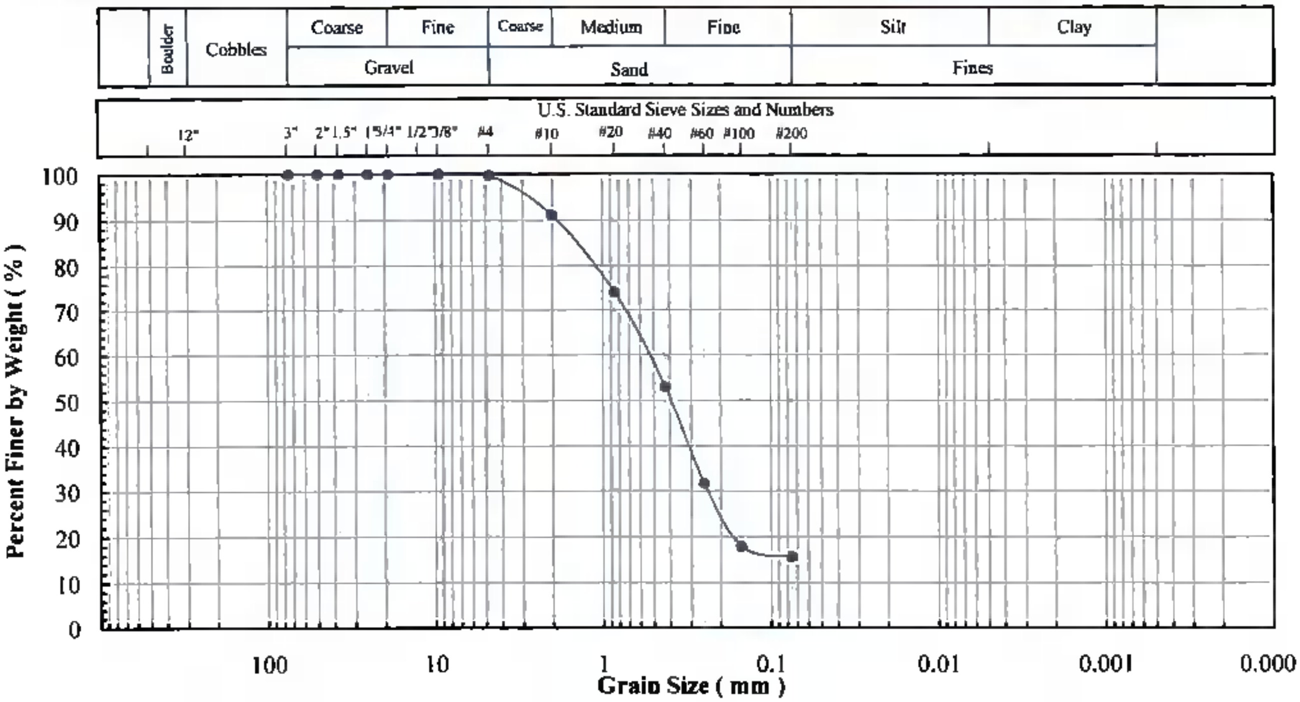
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-12 (15')
Lab Sample No:	13C124

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



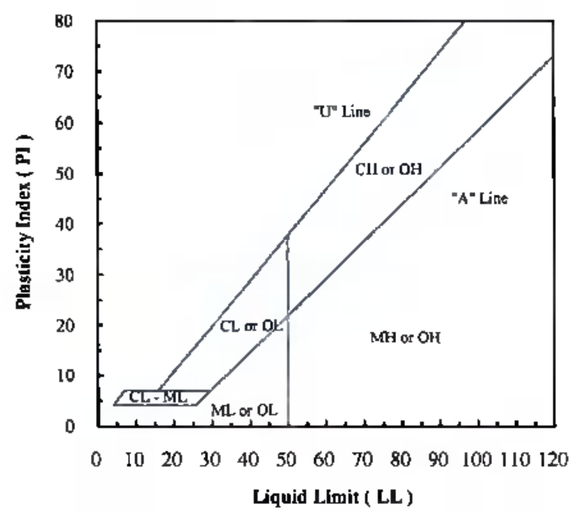
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.7
#10	2.00	91.2
#20	0.850	74.1
#40	0.425	52.9
#60	0.250	31.6
#100	0.150	17.9
#200	0.075	15.7

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	0.3
Sand (%):	84.0
Fines (%):	15.7
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
GSB-12 (15')	13C124	31.8	15.7				

Note(s):

4-25-13
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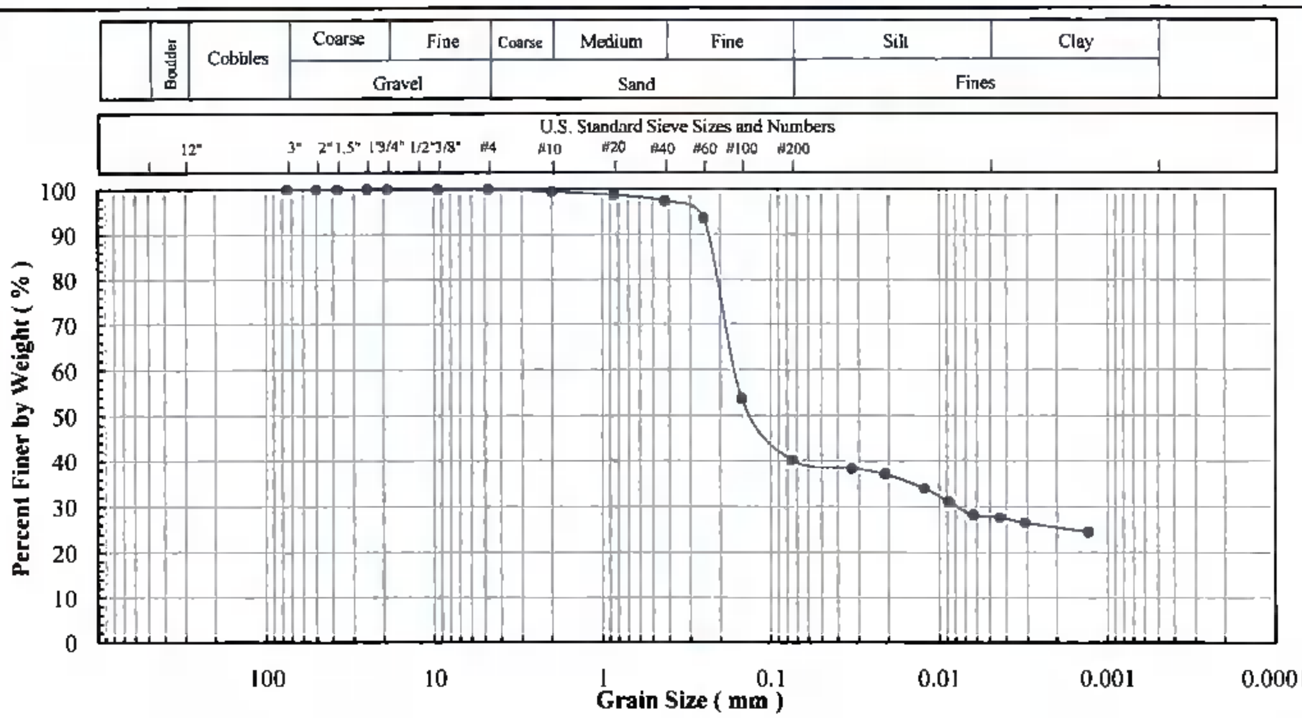
Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-3 (11.5-13.5')
Lab Sample No:	13C161

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



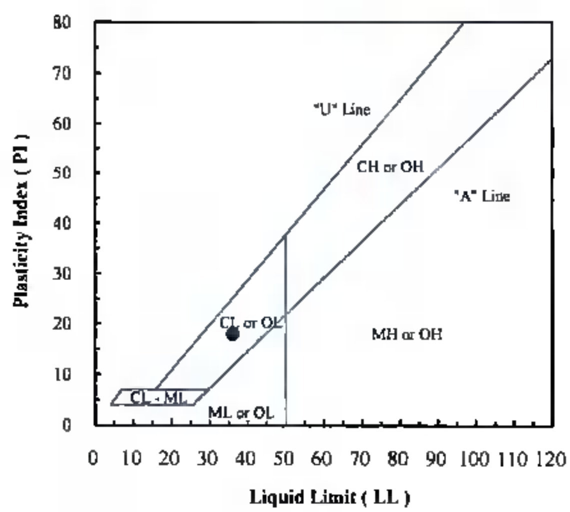
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.6
#20	0.850	98.8
#40	0.425	97.4
#60	0.250	93.6
#100	0.150	53.7
#200	0.075	40.2

Specific Gravity (-):	2.67

Hydrometer Particle Diameter (mm)	% Finer
0.0332	38.4
0.0123	34.0
0.0063	28.2
0.0031	26.5
0.0013	24.5

Gravel (%):	
Sand (%):	59.8
Fines (%):	40.2
Silt (%):	12.4
Clay (%):	27.8

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
GSB-3 (11.5-13.5')	13C161	19.8	40.2	36	18	18	SC - Clayey sand

Note(s): Engineering classification is based on the assumption that the fines are either CL or CH.

4-25-13
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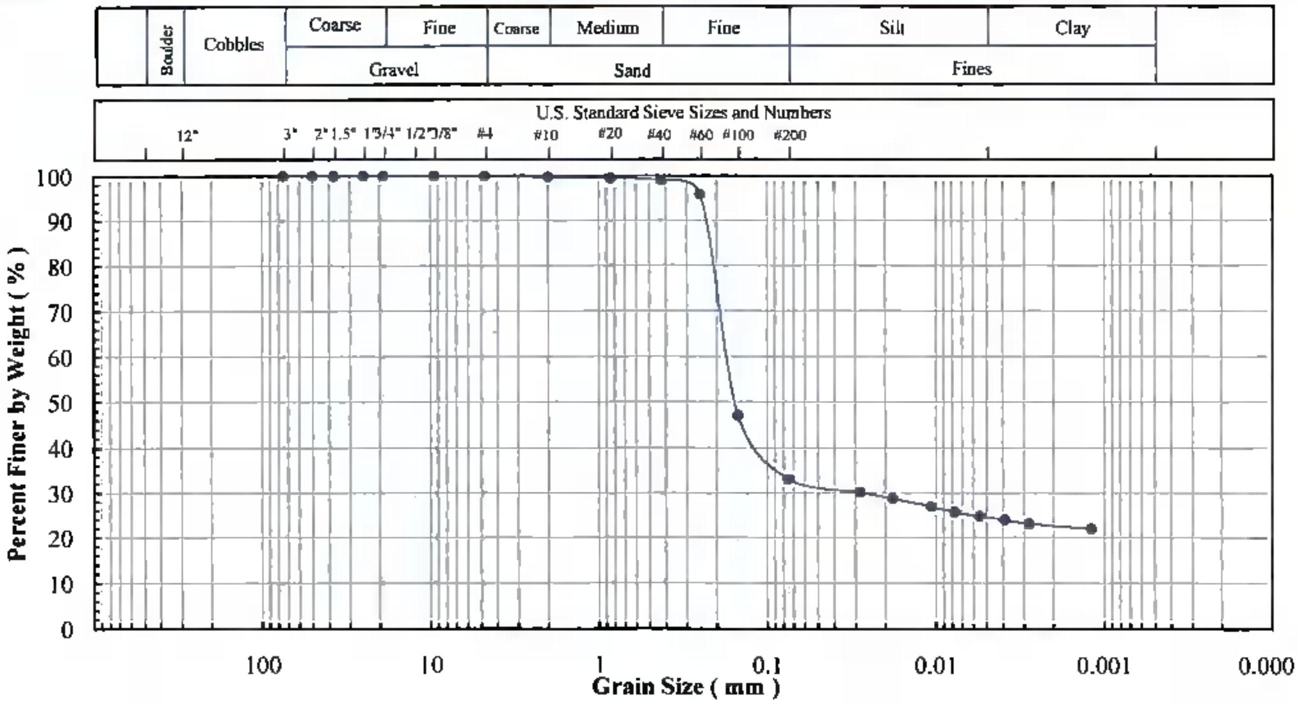
Excel Geotechnical Testing, Inc. "Excellence in Testing" 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	**Project Name:** Winyah Generating Station **Project No:** 585 **Client Sample ID:** GSB-07 (6.5-8.5') **Lab Sample No:** 13C163

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



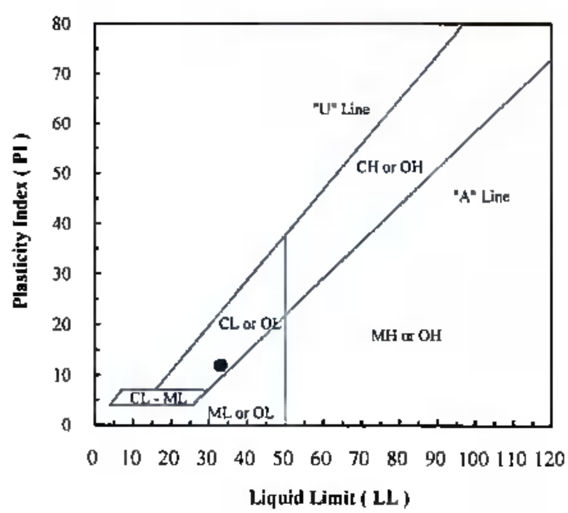
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.5
#40	0.425	99.0
#60	0.250	95.9
#100	0.150	47.0
#200	0.075	33.1

Specific Gravity (-):	2.69

Hydrometer Particle Diameter (mm)	% Finer
0.0282	30.1
0.0107	27.0
0.0055	24.8
0.0028	23.1
0.0012	22.0

Gravel (%):	
Sand (%):	66.9
Fines (%):	33.1
Silt (%):	8.5
Clay (%):	24.6

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
GSB-07 (6.5-8.5')	13C163	21.5	33.1	33	21	12	

Note(s):

4-25-13 NSR

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-08 (36.5-38.5')*
Lab Sample No:	13C164

Excel Geotechnical Testing, Inc.
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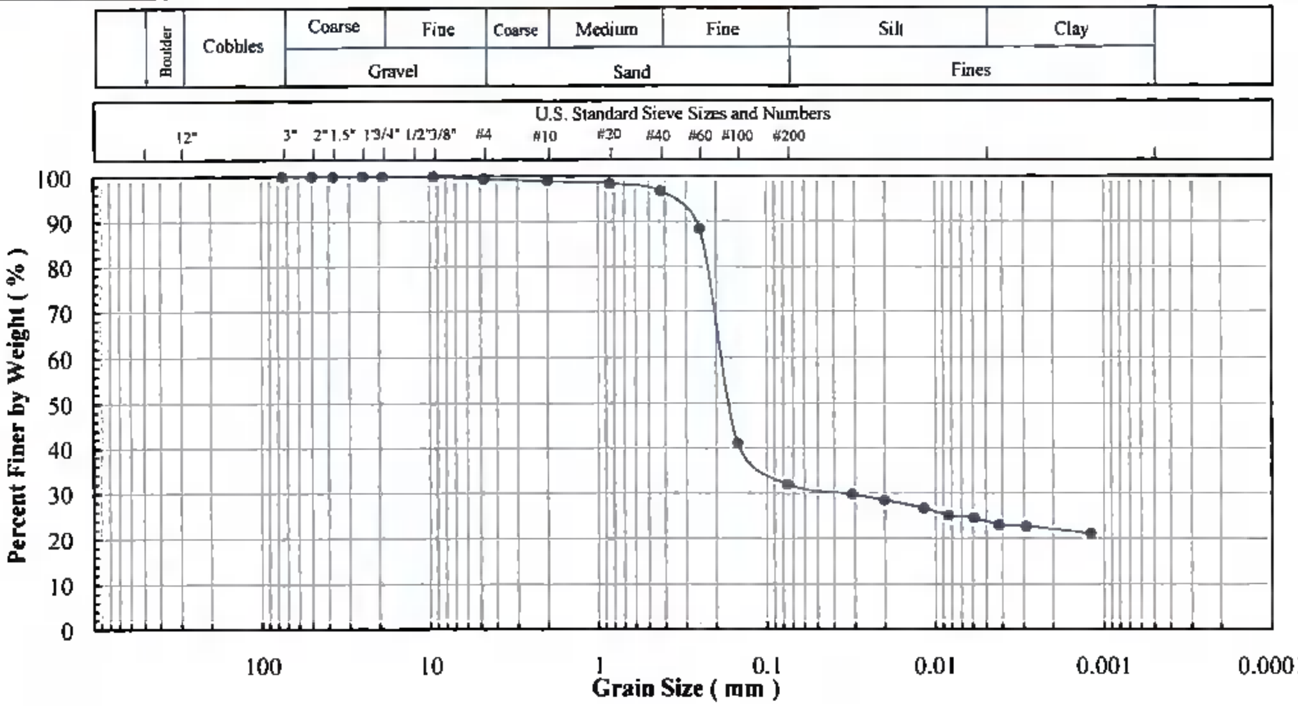
ASTM C 136, D 422, D 854, D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

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



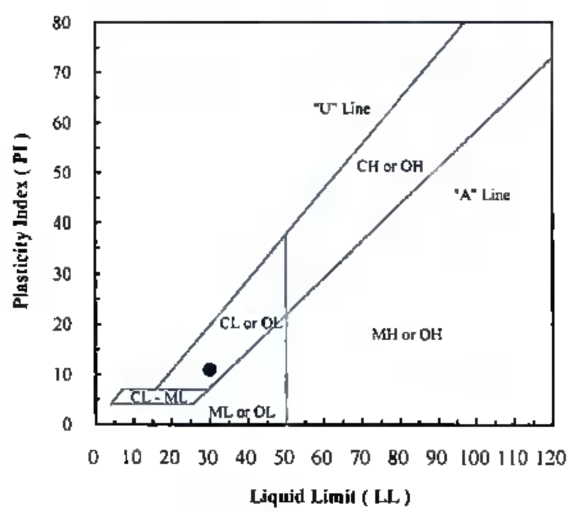
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	99.5
#10	2.00	99.0
#20	0.850	98.5
#40	0.425	96.7
#60	0.250	88.4
#100	0.150	41.1
#200	0.075	31.9

Specific Gravity (-):	2.64

Hydrometer Particle Diameter (mm)	% Finer
0.0315	29.7
0.0117	26.6
0.0059	24.5
0.0029	22.6
0.0012	21.1

Gravel (%):	0.5
Sand (%):	67.6
Fines (%):	31.9
Silt (%):	8.2
Clay (%):	23.7

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
GSB-08 (36.5-38.5')*	13C164	30.6	31.9	30	19	11	

Note(s): * Clayey portion of the triaxial specimens.

4-25-13
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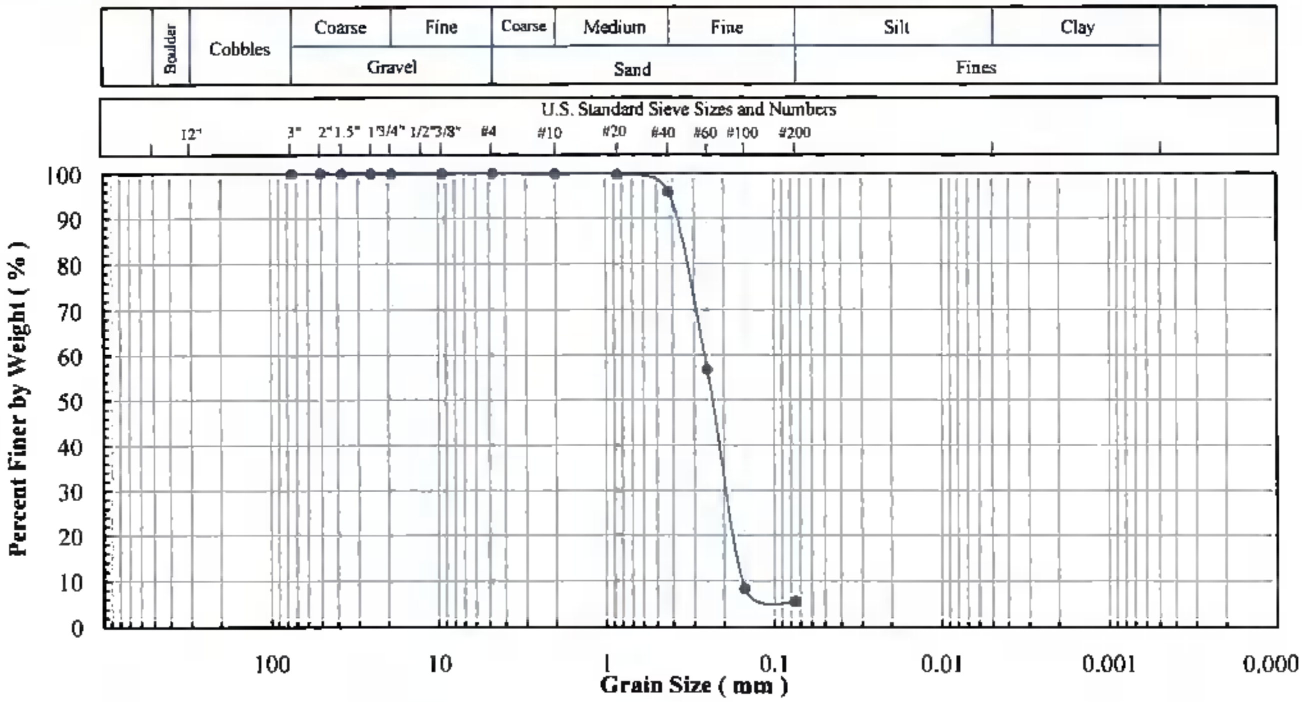
Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	GSB-08 (36.5-38.5')*
Lab Sample No:	13C164

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



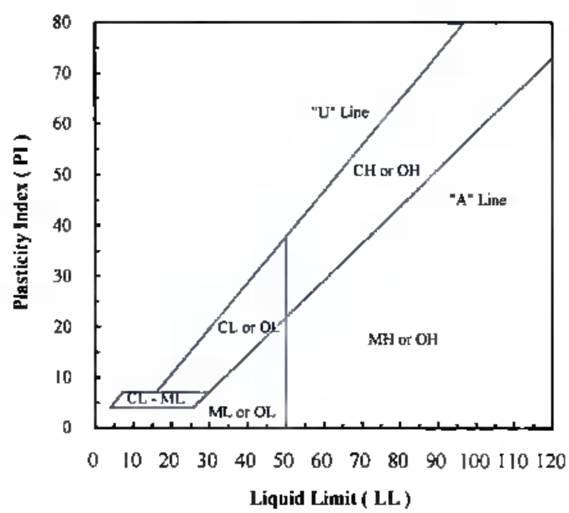
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.7
#40	0.425	95.9
#60	0.250	56.8
#100	0.150	8.3
#200	0.075	5.5

Specific Gravity (-):	

Hydrometer Particle Diameter (mm)	% Finer

Gravel (%):	
Sand (%):	94.5
Fines (%):	5.5
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
GSB-08 (36.5-38.5')*	13C164	13.6	5.5				

Note(s): * Sandy portion of the triaxial specimens

4-25-13
NSR

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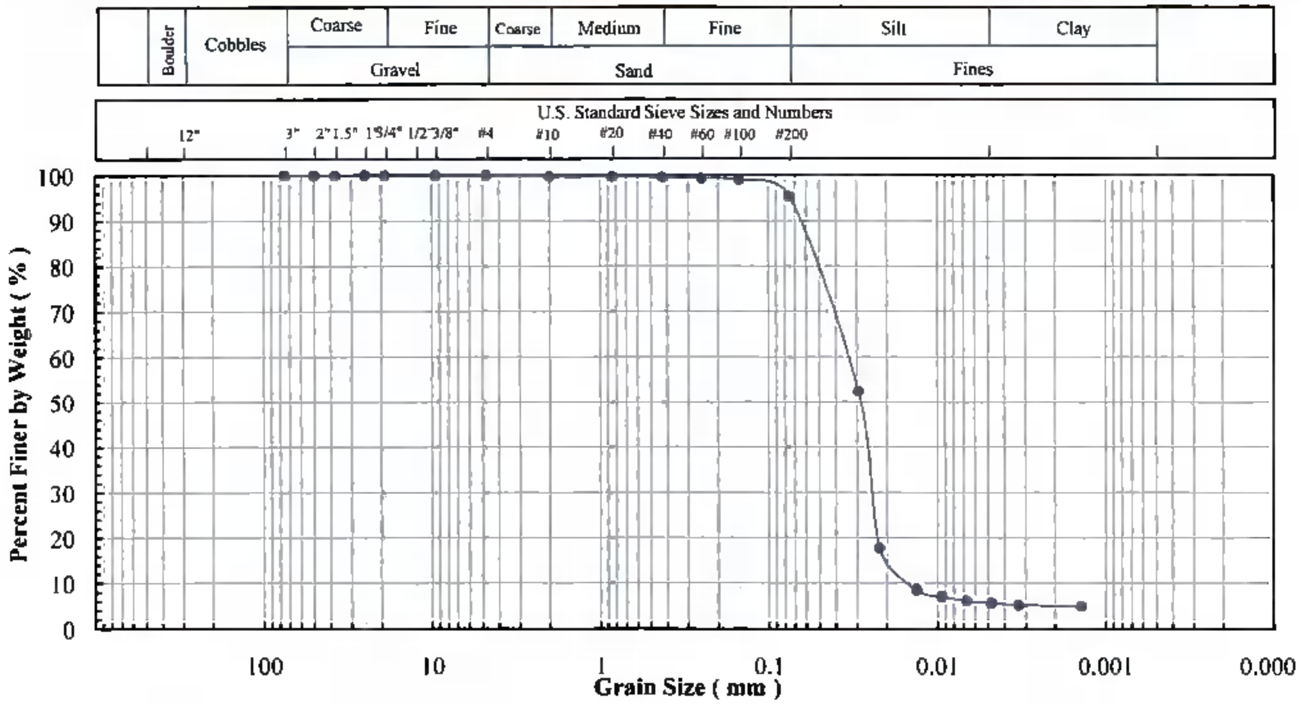
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-01
Lab Sample No:	13C175

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



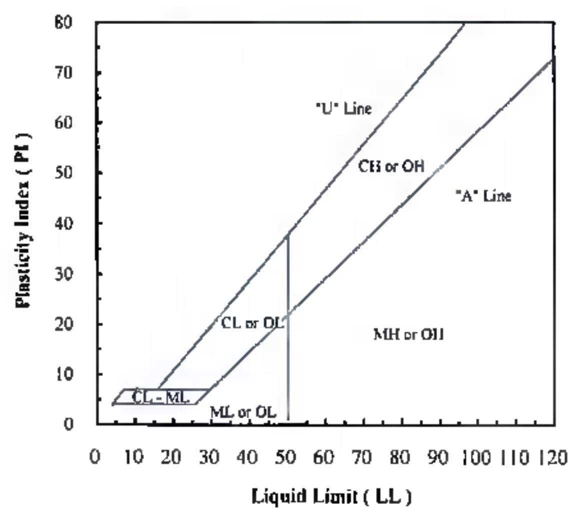
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.7
#40	0.425	99.6
#60	0.250	99.4
#100	0.150	99.0
#200	0.075	95.4

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0295	52.4
0.0133	8.5
0.0067	6.1
0.0033	5.2
0.0014	5.0

Gravel (%):	
Sand (%):	4.6
Fines (%):	95.4
Silt (%):	89.7
Clay (%):	5.7

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
WGS-TP-01	13C175	44.7	95.4	NP	NP	NP	

Note(s): Test material is believed to be gypsum
An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.
Gypsum may dissolve in water which may affect sieve and specially hydrometer results
Moisture content is based on oven drying at 60 degree centigrade

4-25-13
NSA

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-01
Lab Sample No:	13C175

Excel Geotechnical Testing, Inc.
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953 Forrest Street, Roswell, Georgia 30075
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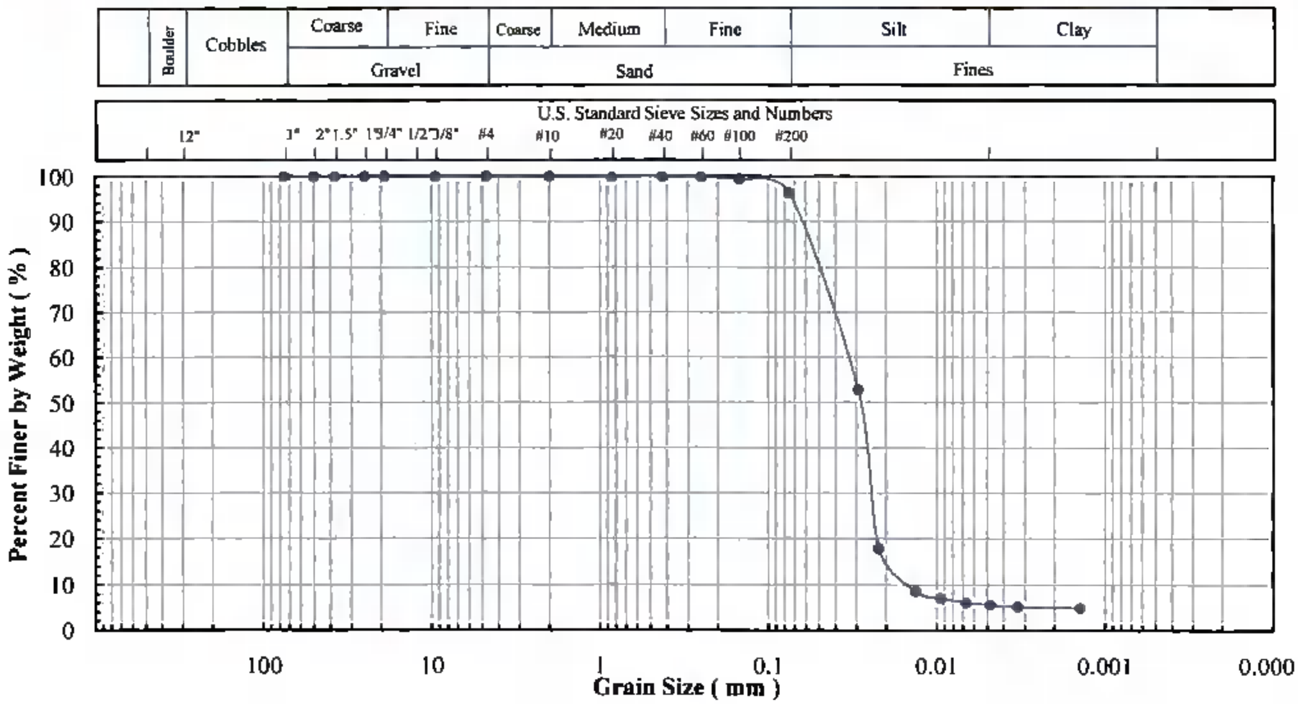
ASTM C 136, D 422, D 854, D 1140, D 2216, D 2487, D 4318

SOIL INDEX PROPERTIES

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



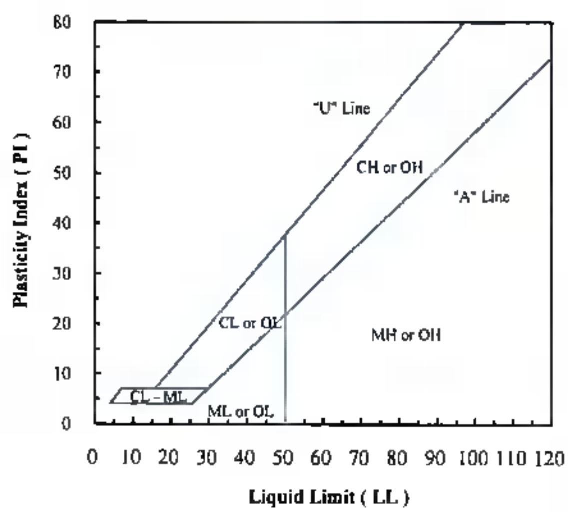
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.9
#60	0.250	99.8
#100	0.150	99.3
#200	0.075	96.2

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0295	52.8
0.0133	8.6
0.0067	6.2
0.0033	5.2
0.0014	5.0

Gravel (%):	
Sand (%):	3.8
Fines (%):	96.2
Silt (%):	90.4
Clay (%):	5.8

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
WGS-TP-01	13C175	63.2	96.2	NP	NP	NP	

Note(s): Test material is believed to be gypsum
An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 100 degree centigrade

4-25-13
NSR

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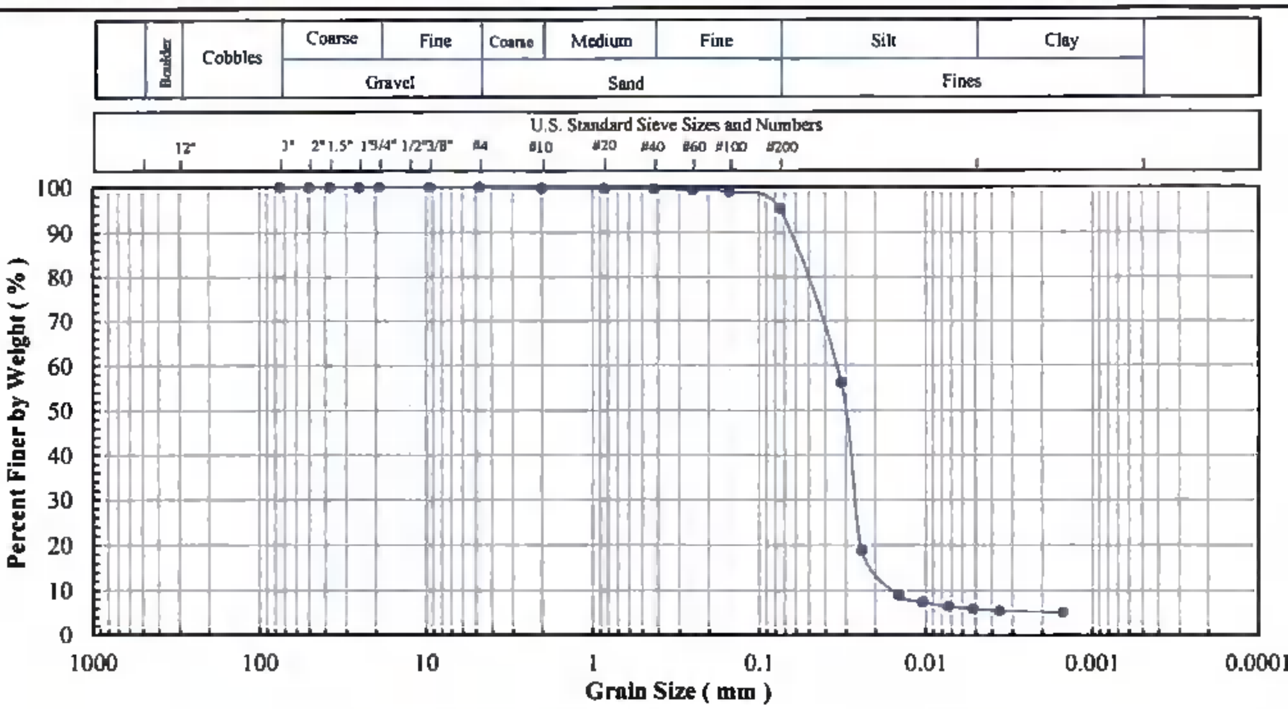
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-01
Lab Sample No:	13C175

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



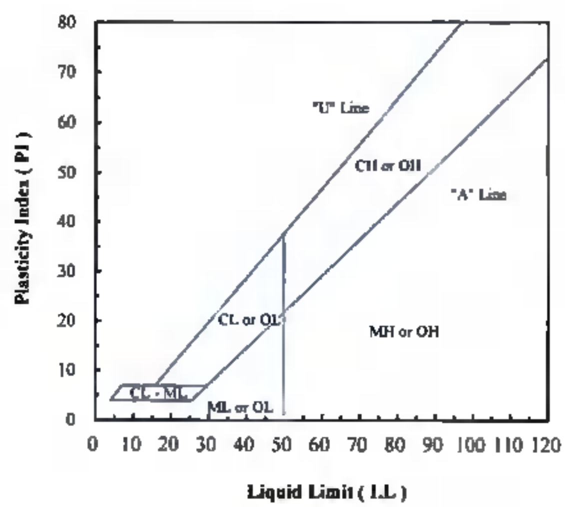
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.8
#20	0.850	99.7
#40	0.425	99.6
#60	0.250	99.4
#100	0.150	99.0
#200	0.075	95.4

Specific Gravity (-):	2.38

Hydrometer Particle Diameter (mm)	% Finer
0.0322	56.3
0.0145	9.1
0.0073	6.6
0.0036	5.6
0.0015	5.3

Gravel (%):	
Sand (%):	4.6
Fines (%):	95.4
Silt (%):	89.3
Clay (%):	6.1

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
WGS-TP-01	13C175	44.7	95.4	NP	NP	NP	

Note(s): Test material is believed to be gypsum
Specific gravity was determined after drying the test specimen in oven at 60 degree centigrade.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 60 degree centigrade

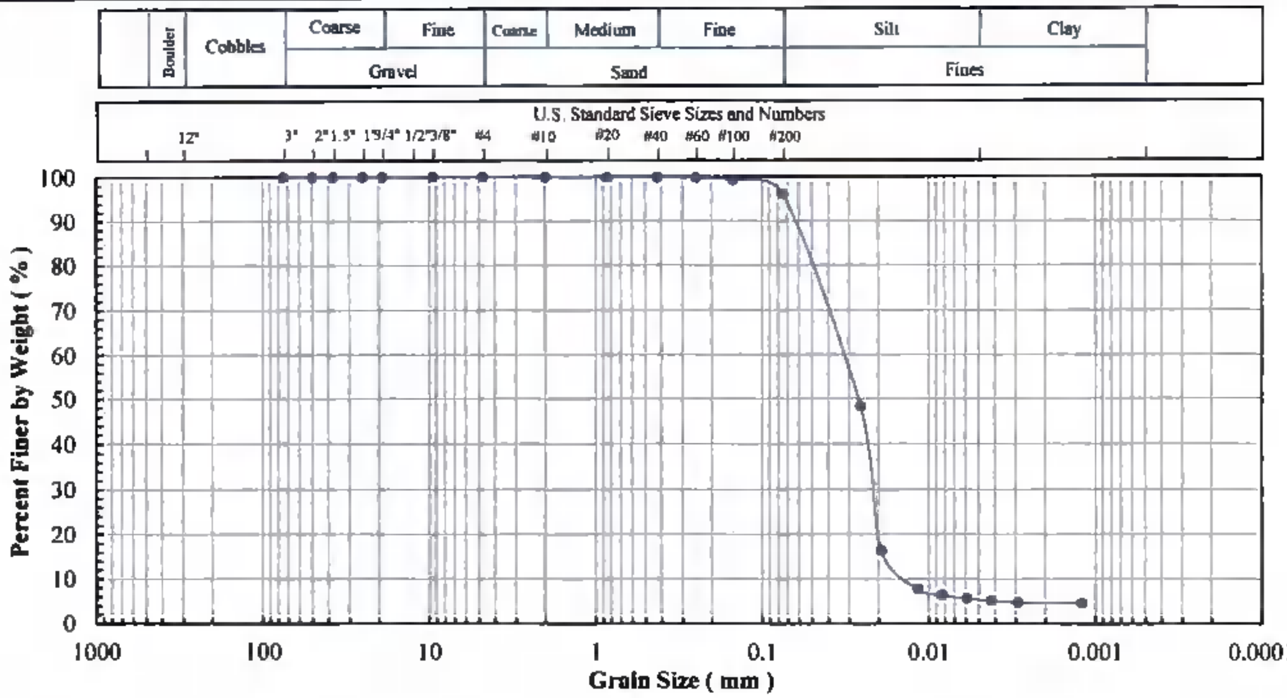
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
963 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-01
Lab Sample No:	13C175

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



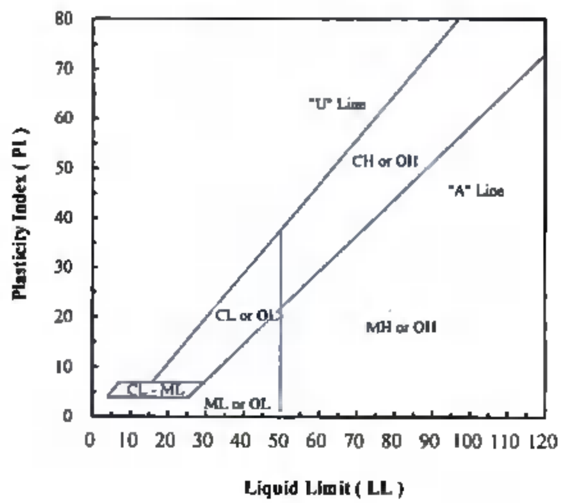
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.9
#60	0.250	99.8
#100	0.150	99.3
#200	0.075	96.2

Specific Gravity (-):	3.12

Hydrometer Particle Diameter (mm)	% Finer
0.0257	48.4
0.0116	7.8
0.0059	5.7
0.0029	4.8
0.0012	4.6

Gravel (%):	
Sand (%):	3.8
Fines (%):	96.2
Silt (%):	90.8
Clay (%):	5.4

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
WGS-TP-01	13C175	63.2	96.2	NP	NP	NP	

Note(s): Test material is believed to be gypsum
Specific gravity was determined after drying the test specimen in oven at 110 degree centigrade.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 110 degree centigrade

5-17-13 NSR

Excel Geotechnical Testing, Inc.

"Excellence in Testing"

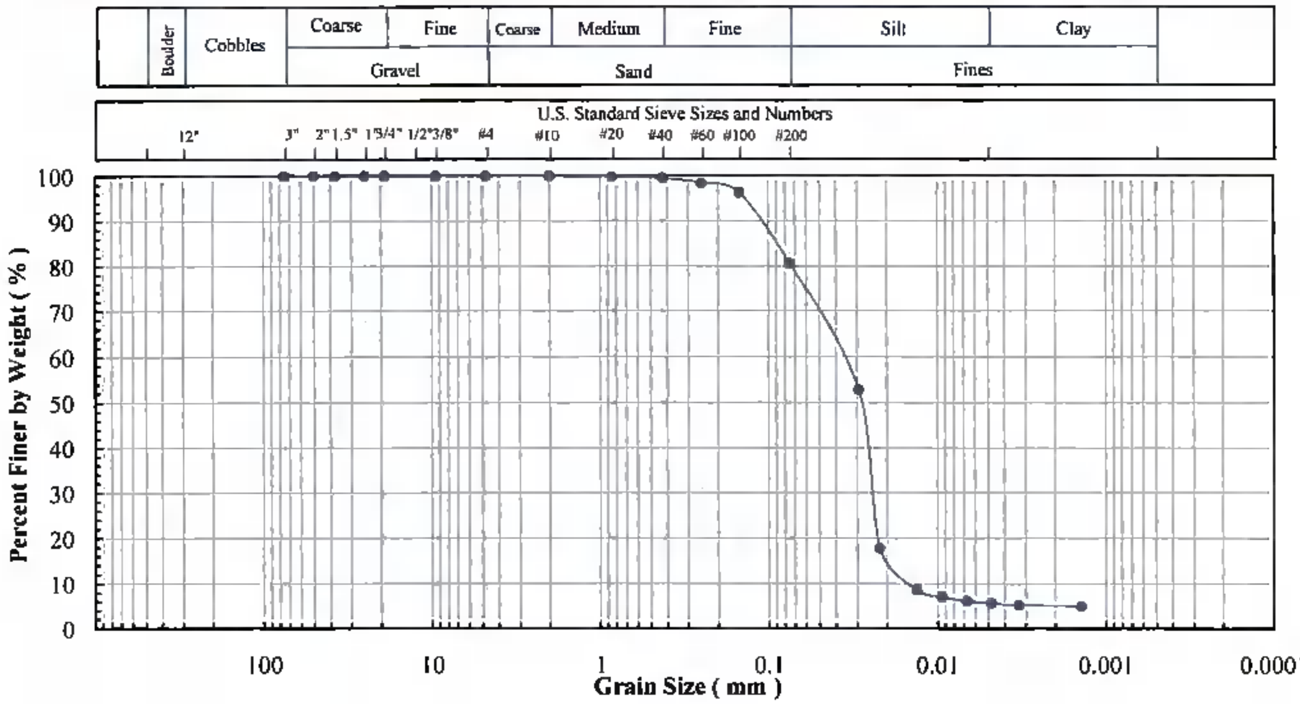
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-02
Lab Sample No:	13C176

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



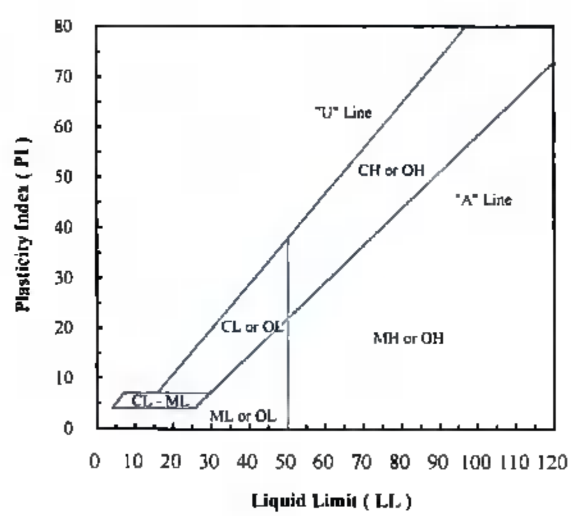
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.5
#60	0.250	98.5
#100	0.150	96.3
#200	0.075	80.6

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0295	52.9
0.0133	8.6
0.0067	6.2
0.0033	5.2
0.0014	5.0

Gravel (%):	
Sand (%):	19.4
Fines (%):	80.6
Silt (%):	74.8
Clay (%):	5.8

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
WGS-TP-02	13C176	46.8	80.6	NP	NP	NP	

Note(s): Test material is believed to be gypsum
An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.
Gypsum may dissolve in water which may affect sieve and specially hydrometer results
Moisture content is based on oven drying at 60 degree centigrade

4-25-13 NSK

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

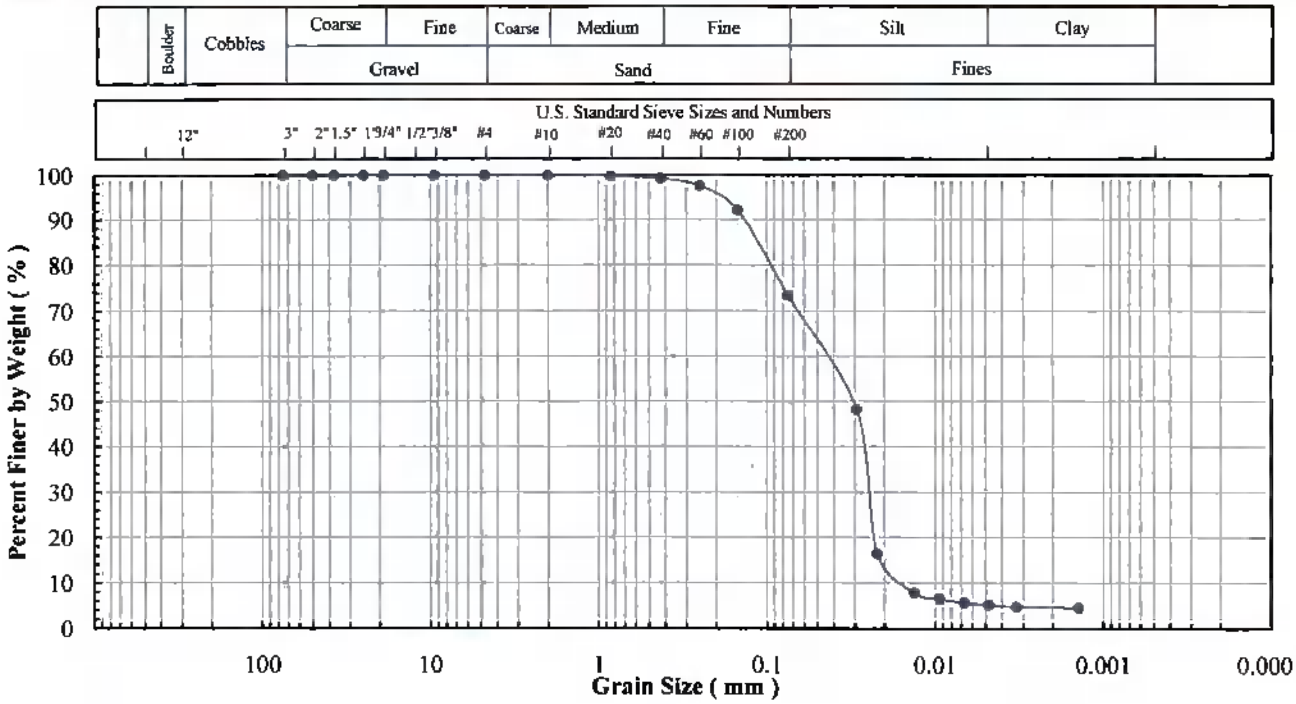
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-02
Lab Sample No:	13C176

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



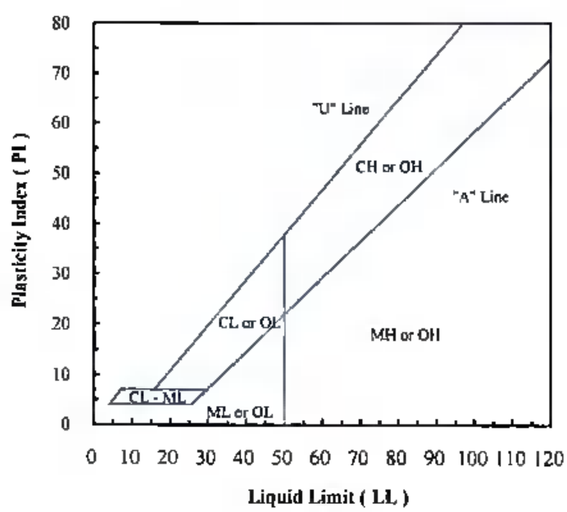
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.3
#60	0.250	97.6
#100	0.150	92.1
#200	0.075	73.5

Specific Gravity (-):	2.65

Hydrometer Particle Diameter (mm)	% Finer
0.0295	48.2
0.0133	7.8
0.0067	5.7
0.0033	4.8
0.0014	4.6

Gravel (%):	
Sand (%):	26.5
Fines (%):	73.5
Silt (%):	68.2
Clay (%):	5.3

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
WGS-TP-02	13C176	58.5	73.5	NP	NP	NP	

Note(s): Test material is believed to be gypsum
An assumed specific gravity of 2.65 was used when analyzing the hydrometer test results.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 100 degree centigrade

4-25-13
NDR

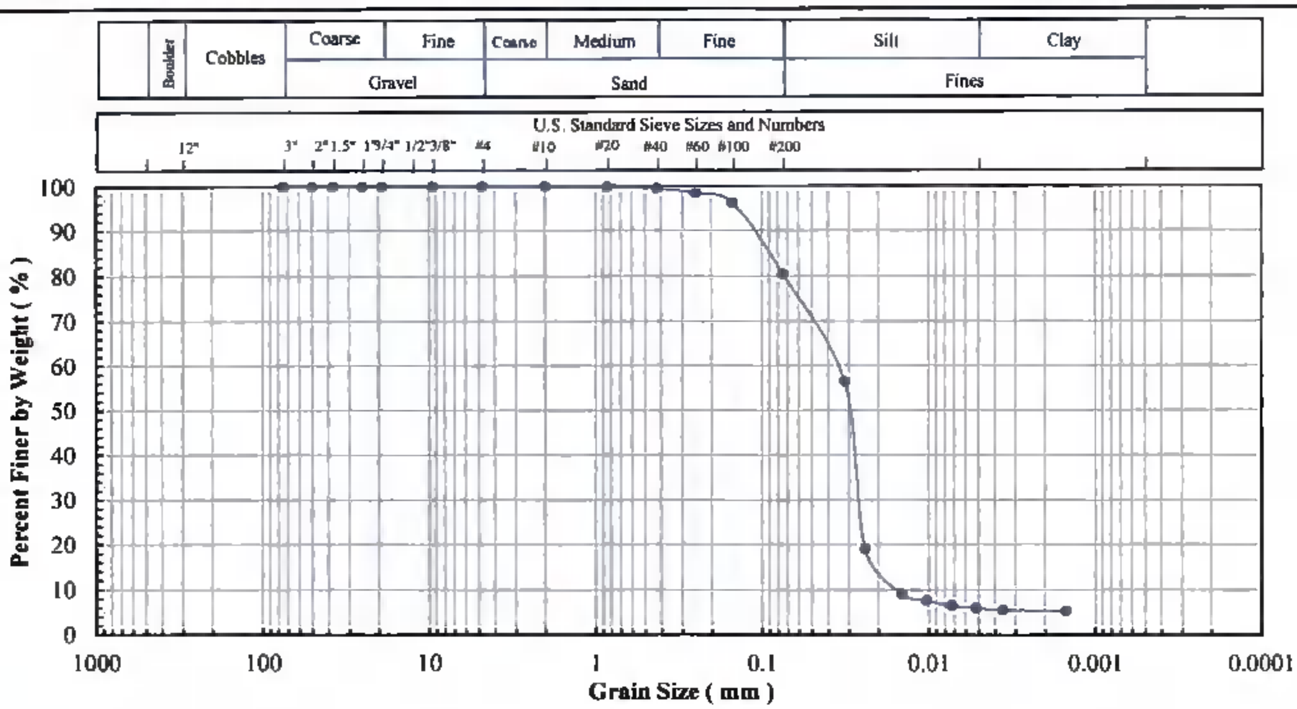
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-02
Lab Sample No:	13C176

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



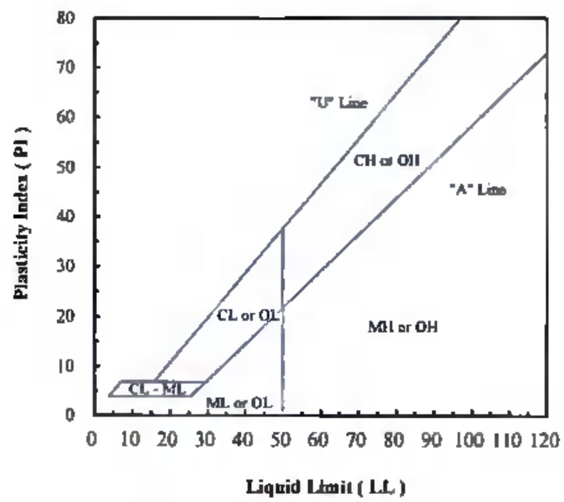
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.5
#60	0.250	98.5
#100	0.150	96.3
#200	0.075	80.6

Specific Gravity (-):	2.39

Hydrometer Particle Diameter (mm)	% Finer
0.0321	56.6
0.0145	9.2
0.0073	6.6
0.0036	5.6
0.0015	5.4

Gravel (%):	
Sand (%):	19.4
Fines (%):	80.6
Silt (%):	74.5
Clay (%):	6.1

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
WGS-TP-02	13C176	46.8	80.6	NP	NP	NP	

Note(s): Test material is believed to be gypsum
Specific gravity was determined after drying the test specimen in oven at 60 degree centigrade.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 60 degree centigrade

5-17-13 NJS

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

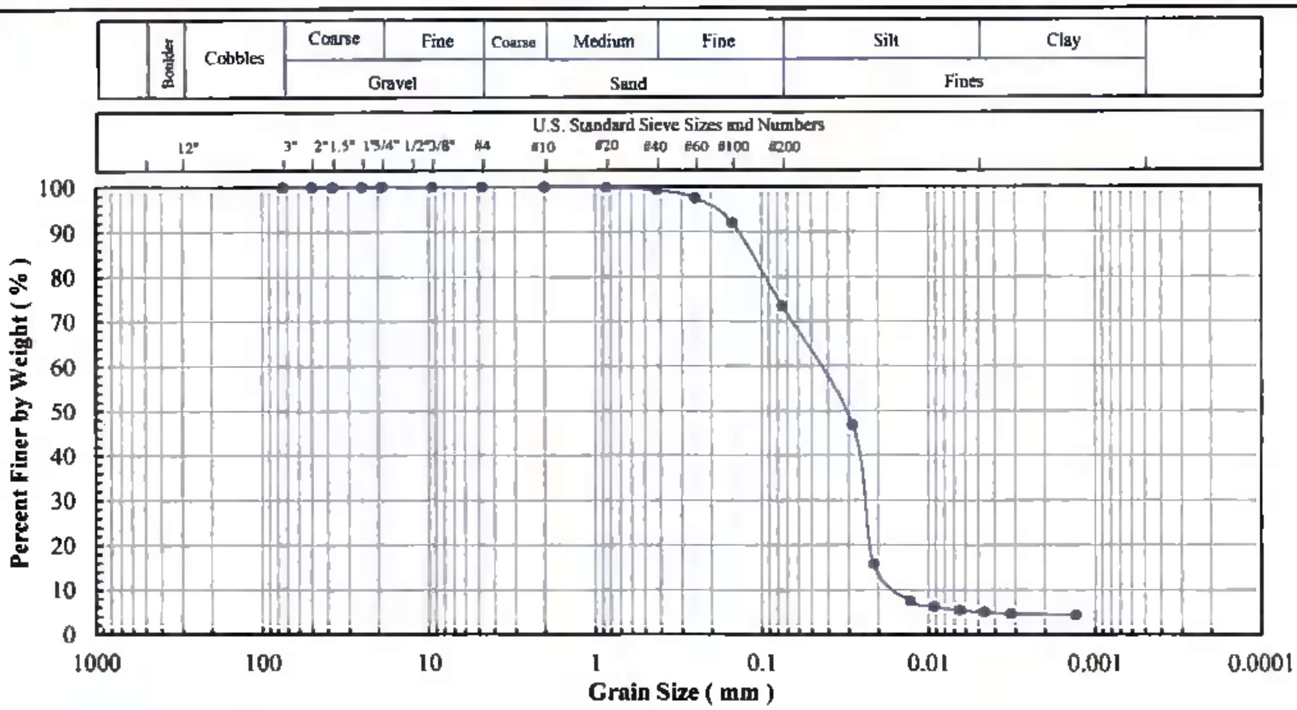
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	585
Client Sample ID:	WGS-TP-02
Lab Sample No:	13C176

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

SOIL INDEX PROPERTIES

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



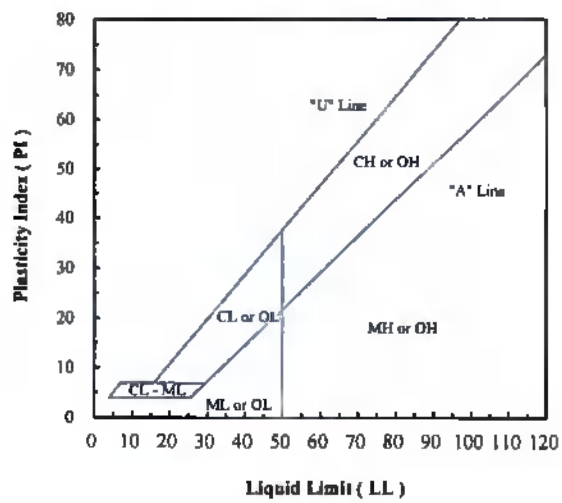
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	100.0
#20	0.850	99.9
#40	0.425	99.3
#60	0.250	97.6
#100	0.150	92.1
#200	0.075	73.5

Specific Gravity (-):	2.77

Hydrometer Particle Diameter (mm)	% Finer
0.0285	47.0
0.0129	7.6
0.0065	5.5
0.0032	4.7
0.0013	4.4

Gravel (%):	
Sand (%):	26.5
Fines (%):	73.5
Silt (%):	68.3
Clay (%):	5.2

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
WGS-TP-02	13C176	58.5	73.5	NP	NP	NP	

Note(s): Test material is believed to be gypsum
Specific gravity was determined after drying the test specimen in oven at 110 degree centigrade.
Gypsum may disolve in water which may effect sieve and specially hydrometer results
Moisture content is based on oven drying at 110 degree centigrade

5-17-13 NSK

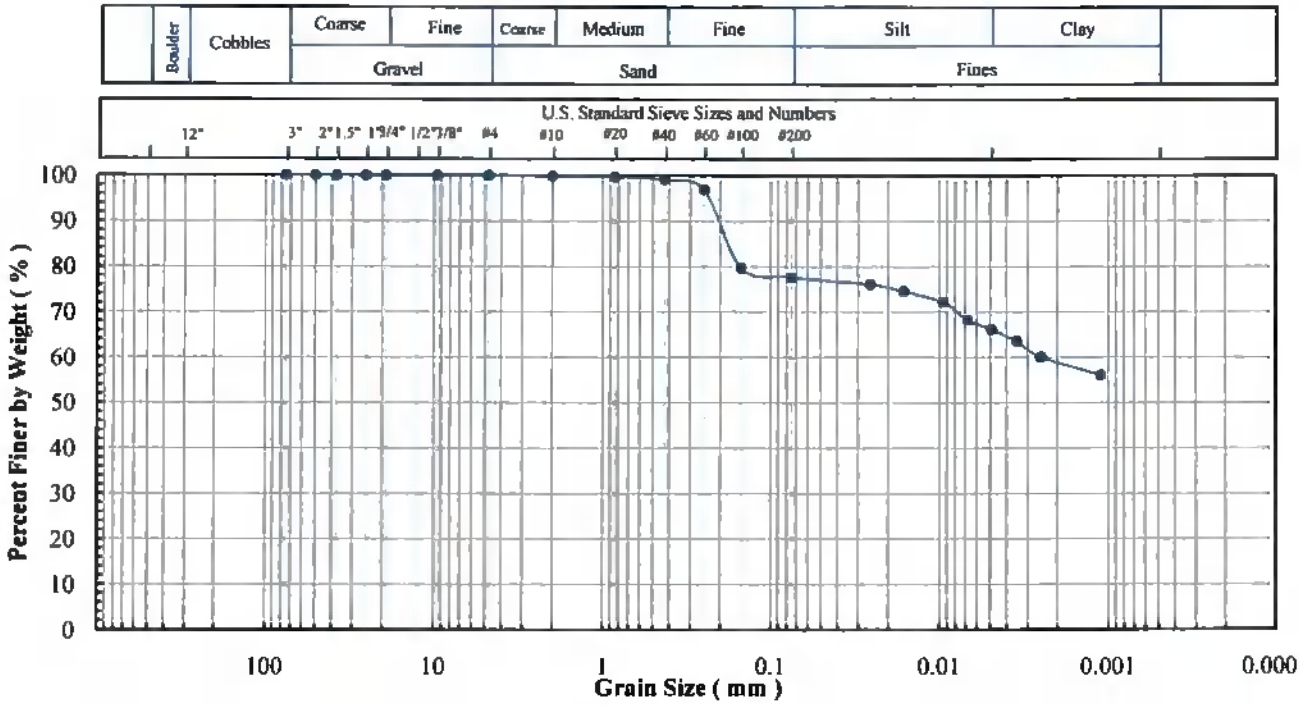
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106, SS-05 (25.0-26.5')
Lab Sample No:	13J141

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



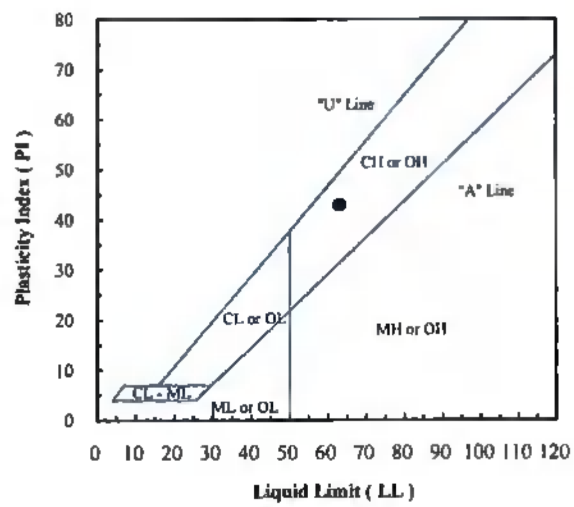
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.9
#20	0.850	99.7
#40	0.425	99.1
#60	0.250	96.9
#100	0.150	79.7
#200	0.075	77.7

Specific Gravity (-):	2.70

Hydrometer Particle Diameter (mm)	% Finer
0.0255	76.1
0.0095	72.2
0.0049	66.2
0.0025	60.1
0.0011	56.2

Gravel (%):	
Sand (%):	22.3
Fines (%):	77.7
Silt (%):	11.3
Clay (%):	66.4

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
SPT-106, SS-05 (25.0-26.5')	13J141	31.4	77.7	63	20	43	CH - Fat clay with sand

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

11-11-13
TK, NSR

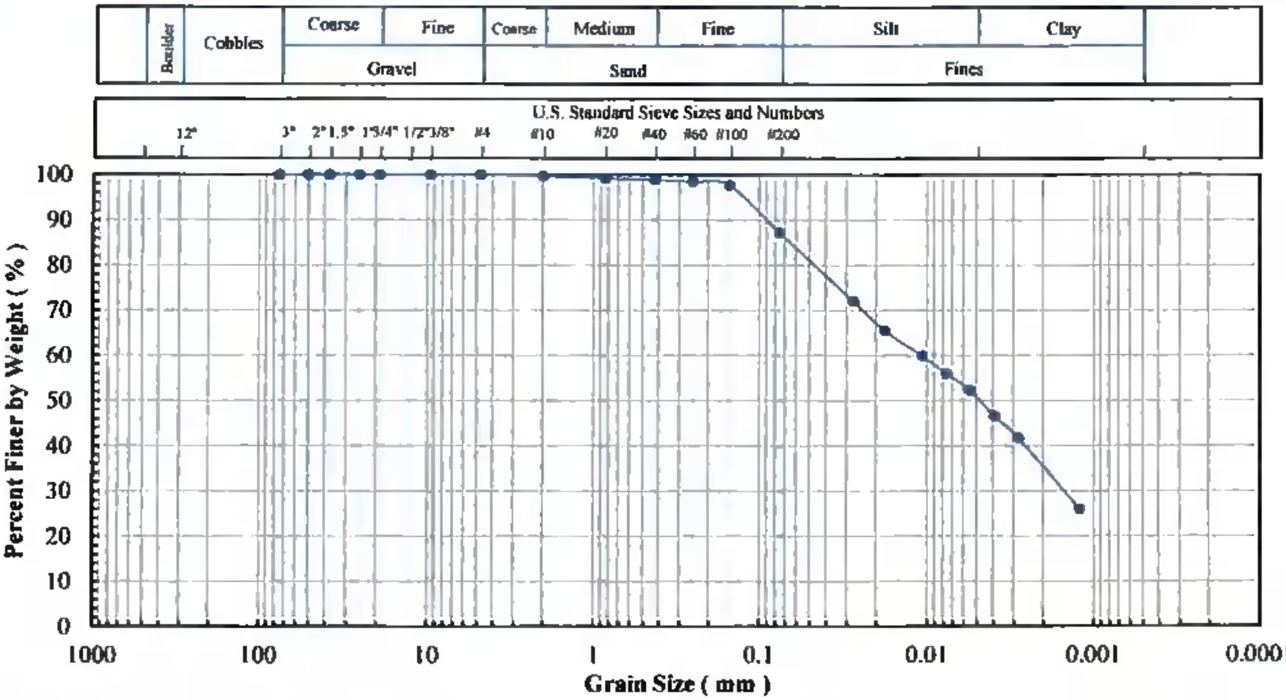
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106, ST-01 (60.0-62.0')
Lab Sample No:	13J353

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



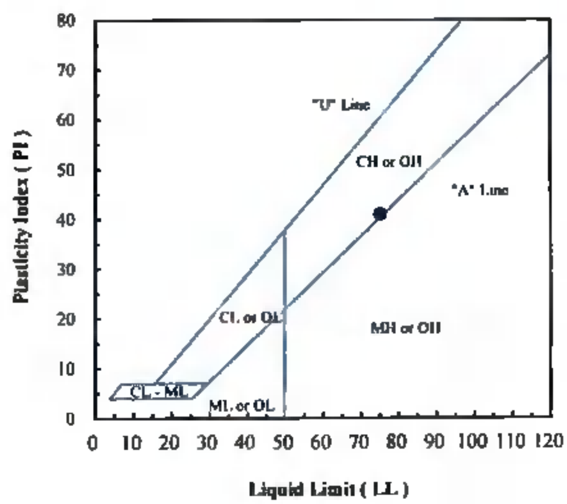
Sieve No.	Size (mm)	% Finer
3"	75	100.0
2"	50	100.0
1.5"	37.5	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.00	99.7
#20	0.850	99.3
#40	0.425	98.9
#60	0.250	98.5
#100	0.150	97.7
#200	0.075	87.2

Specific Gravity (-):	2.7

Hydrometer Particle Diameter (mm)	% Finer
0.0274	72.0
0.0106	59.9
0.0055	52.3
0.0028	41.7
0.0012	26.0

Gravel (%):	
Sand (%):	12.8
Fines (%):	87.2
Silt (%):	36.7
Clay (%):	50.5

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-106, ST-01 (60.0-62.0')	13J353		87.2	75	34	41	CH - Fat clay

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

1-04-14
NJA

BORING LOG NO. GSB-11

PROJECT: Winyah Generation Station

CLIENT: Santee Cooper, Moncks Corner, South Carolina

SITE: Georgetown, South Carolina

LOCATION See Exhibit A-2

Latitude: 33.3351° Longitude: -79.3696°

Depth to base (ft)	Material Description	Sample depth (Ft.)	Water Level Observations	Sample Type	Recovery (In.)	Field Test Results	Test Type	Compressive Strength (tsf)	Strain (%)	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
8.0	**CLAYEY SAND (SC)**, orange to brown, loose to medium dense, dry				10	1-2-3-5 N=5				17		
					13	4-3-5-5 N=8				19		
		5			20	8-6-8-9 N=14				20		
			▽		17	5-6-9-11 N=15				20		
16.0	**SAND (SP)**, yellowish brown to gray, loose, moist				19	8-11-9-8 N=20				27		
		10			12	1-1-2-3 N=3				27		
					17	4-7-5-5 N=12				23		
		15			24	4-5-6-9 N=11						
20.0	**SANDY FAT CLAY (CH)**, mottled orange, soft to stiff, moist				9					21		
					24					18	36-13-23	39
24.0	**SAND (SP)**, yellowish brown to dark brown, medium dense, moist	20			17	2-2-4-5 N=6				25		
					3	1-1-1-2 N=2				39		
38.0	**CLAYEY SAND (SC)**, gray to brown, loose to medium dense, moist	25			15	8-9-9-8 N=18				18		
					24	4-4-4-4 N=8				26		
					24							
		30										
					10	2-4-5 N=9				23		
		35			0							
40.0	**NO RECOVERY**											
43.0	**SAND WITH CLAY (SP-SC)**, coarse grained, gray, loose, moist	40			17	2-1-2 N=3				28		
					18	2-2-3 N=5				34		
47.0	**NO RECOVERY**	45			0							
50.0	**CRUSH ROCK with SAND (SP)**, gray, very dense, moist	50			0.5	50/0"				12		

Boring Terminated at 50 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
Mud Rotary

Abandonment Method:
Borings backfilled with cement-bentonite grout upon completion.

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

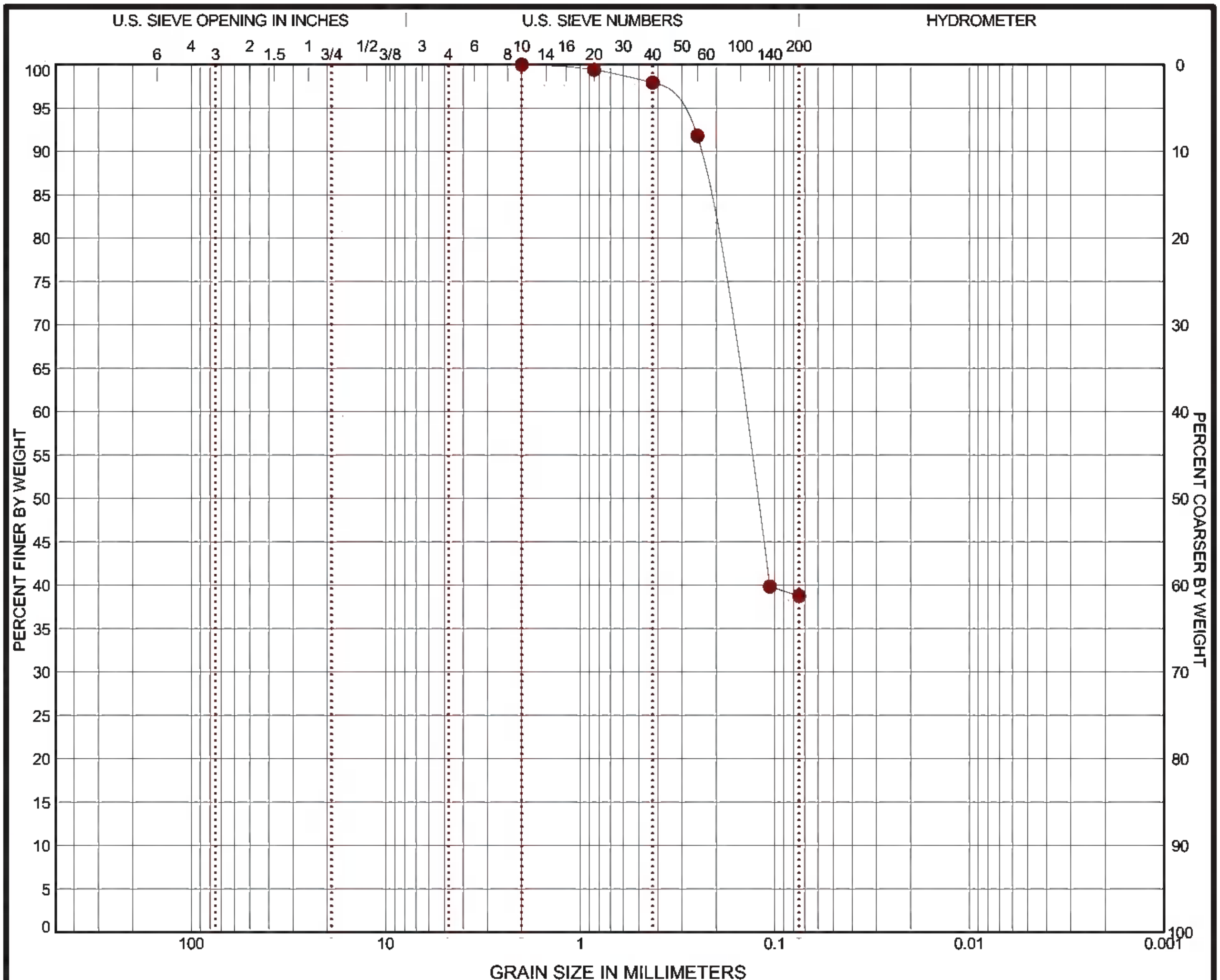
▽ *8.2 ft at time of exploration*

Terracon
1450 Fifth St W
North Charleston, SC

Boring Started: 3/30/2016	Boring Completed: 3/30/2016
Drill Rig: CME-45C	Driller: Carolina Drilling, Inc.
Project No.: EN165065	Exhibit: A-5

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**GSB-11**	**18 - 20**	**0.0**	**0.0**	**61.2**		**38.8**		**SC**

	GRAIN SIZE		
	●		
D_{60}	**0.148**		
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE (size)	PERCENT FINER ●		
1 1/2"			
1"			
3/4"			
1/2"			
3/8"			
#4			
#10	**100.0**		
#20	**99.41**		
#40	**97.93**		
#60	**91.8**		
#100			
#200	**38.77**		

SOIL DESCRIPTION

● CLAYEY SAND (SC)

REMARKS

●

PROJECT: Winyah Generation Station

SITE: Georgetown, South Carolina

Terracon

1450 Fifth St W
North Charleston, SC

PROJECT NUMBER: EN165065

CLIENT: Santee Cooper
Moncks Corner, South Carolina

EXHIBIT: B-1

Triaxial Testing

Excel Geotechnical Testing, Inc.
"Excellence in Testing"

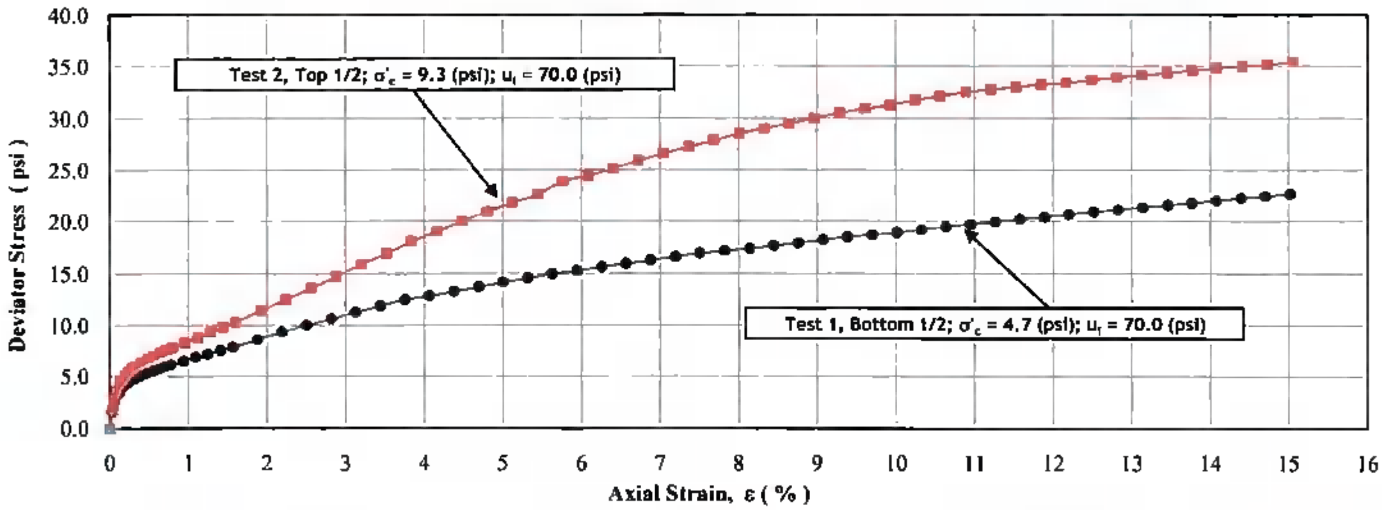
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

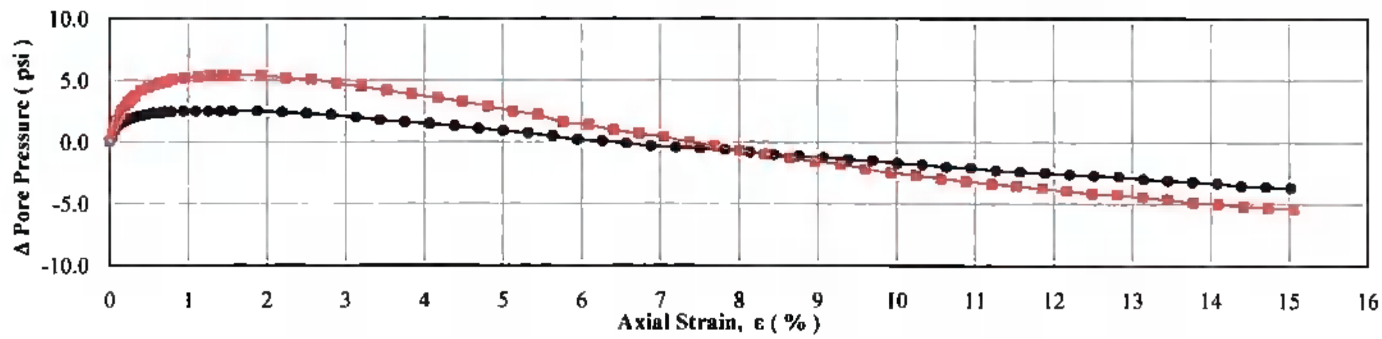
Project Name:	Winyah Generating Station
Project No:	585
Site Sample ID:	GSB-03 (11.5-13.5')
Lab Sample No:	13C161

ASTM D 4767

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

Figure 1





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	22.7	31.1	8.4	66.3	15.0
2	35.4	50.1	14.7	64.6	15.1

Test Specimen No.	Strength at App. 15% Axial Strain				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	22.7	31.1	8.4	66.3	15.0
2	35.4	50.1	14.7	64.6	15.1

Notes:

σ'_c = Consolidation pressure, (psi) u_t = Initial pore pressure,(psi) u = Pore pressure,(psi) ε_a = Axial strain, (%)

σ'_1 = Effective axial stress, (psi) σ'_3 = Effective radial stress (confining pressure), (psi) $\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)

4-25-13
NSR

Excel Geotechnical Testing, Inc.

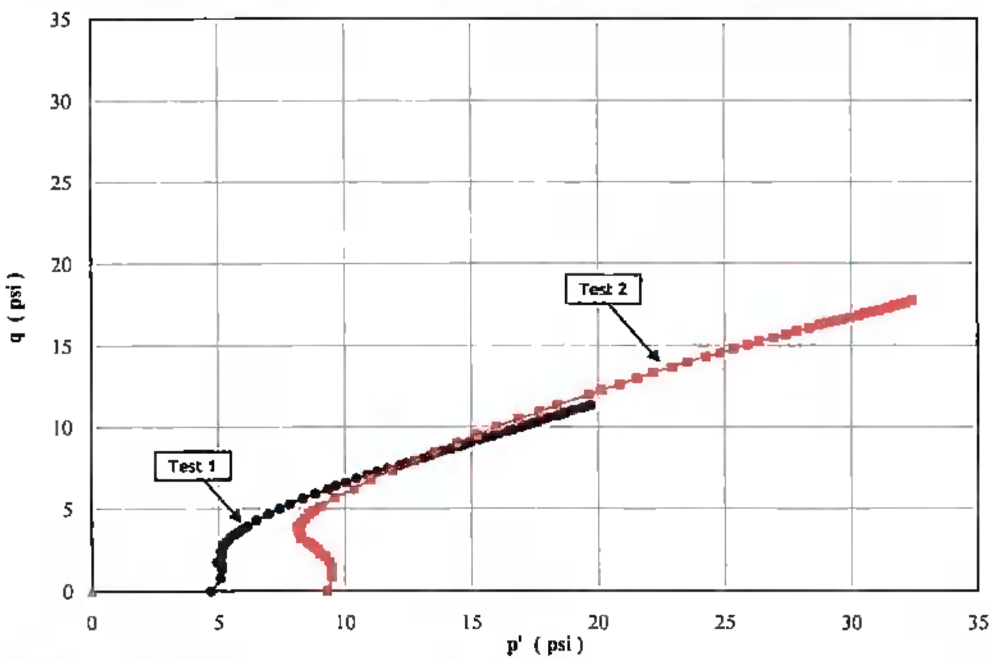
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 585
Site Sample ID: GSB-03 (11.5-13.5')
Lab Sample No: 13C161

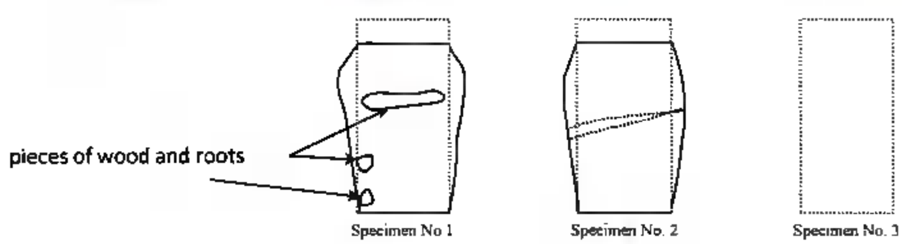
ASTM D 4767

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

Figure 2



Test Specimen No.	Initial Conditions							Strain Rate	Specimen Quality Bad to Good
	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_c (psi)	σ'_c (psi)	(% / min)	(1 to 10)
1	6.43	2.84	20.2	109.4	0.96	70.0	4.7	0.062	7
2	6.33	2.83	22.1	101.8	0.97	70.0	9.3	0.063	5



Notes:

4-25-13
NSA

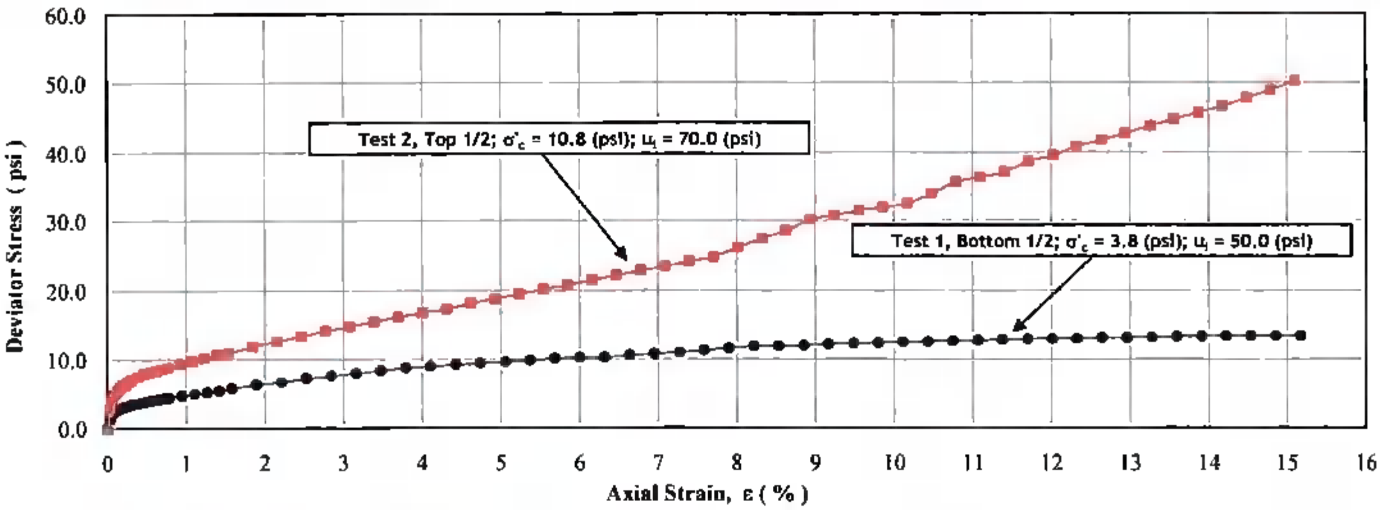
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

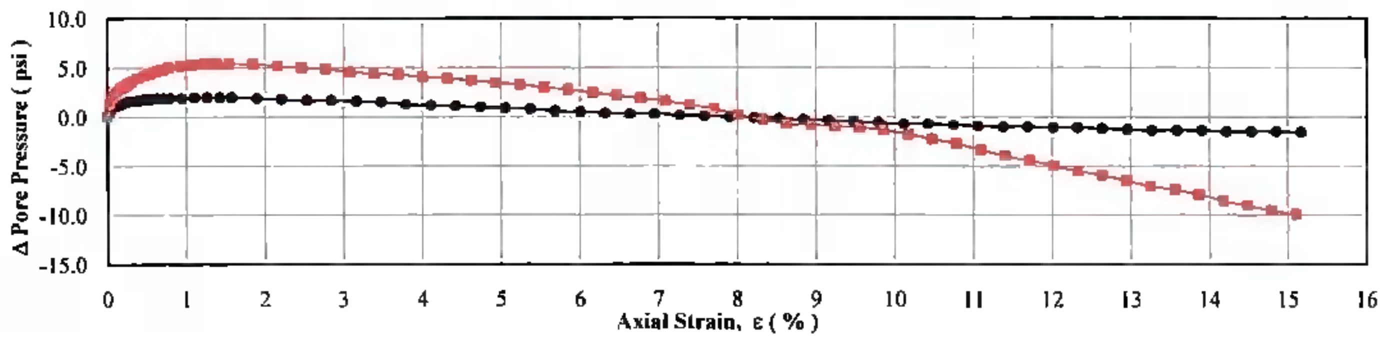
Project Name:	Winyah Generating Station
Project No:	585
Site Sample ID:	GSB-07 (6.5- 8.5')
Lab Sample No:	13C163

ASTM D 4767

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

Figure 1





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	13.5	18.9	5.4	48.4	15.2
2	50.1	70.9	20.8	60.0	15.1

Test Specimen No.	Strength at App. 15% Axial Strain				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	13.5	18.9	5.4	48.4	15.2
2	50.1	70.9	20.8	60.0	15.1

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi) u = Pore pressure,(psi) ε_a = Axial strain, (%)

σ'_1 = Effective axial stress, (psi) σ'_3 = Effective radial stress (confining pressure), (psi) $\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)

4-25-13
NSK

Excel Geotechnical Testing, Inc.

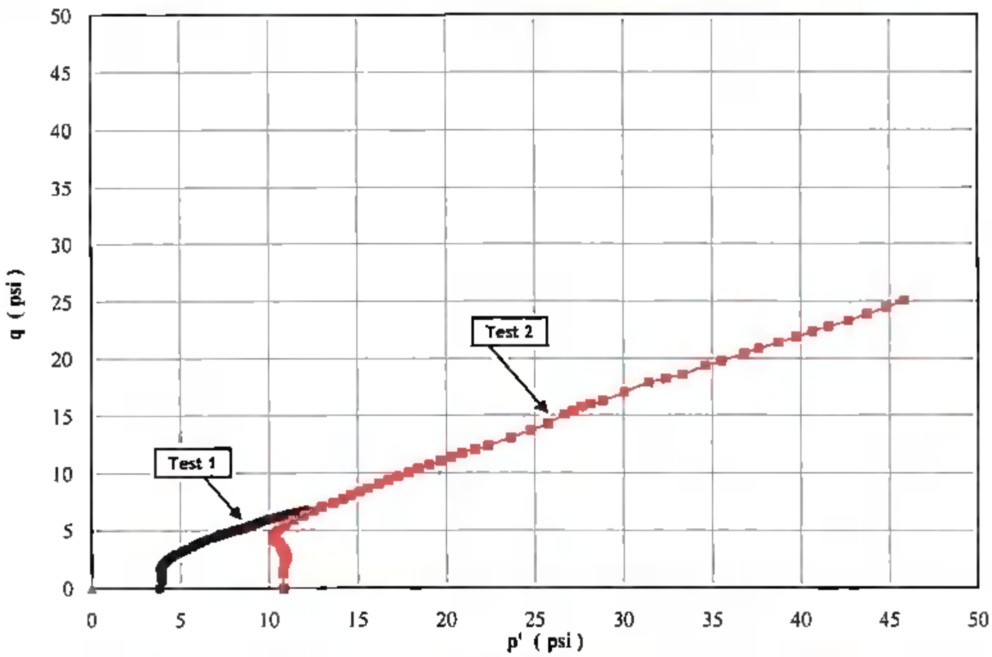
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 585
Site Sample ID: GSB-07 (6.5- 8.5')
Lab Sample No: 13C163

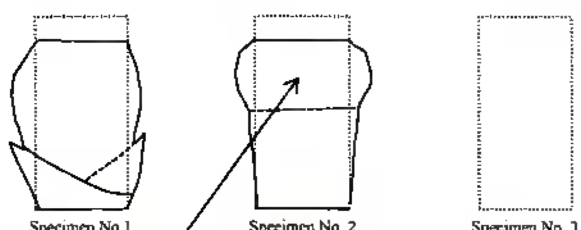
ASTM D 4767

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

Figure 2



Test Specimen No.	Initial Conditions Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_i (psi)	σ'_c (psi)	Strain Rate (% / min)	Specimen Quality Bad to Good (1 to 10)
1	6.36	2.85	15.5	111.0	0.98	50.0	3.8	0.063	8
2	6.55	2.84	17.1	112.8	0.96	70.0	10.8	0.061	6



Notes:

Top 1/3 was soft and contained small pieces of wood that was replaced with similar soil

11-25-13
NSR

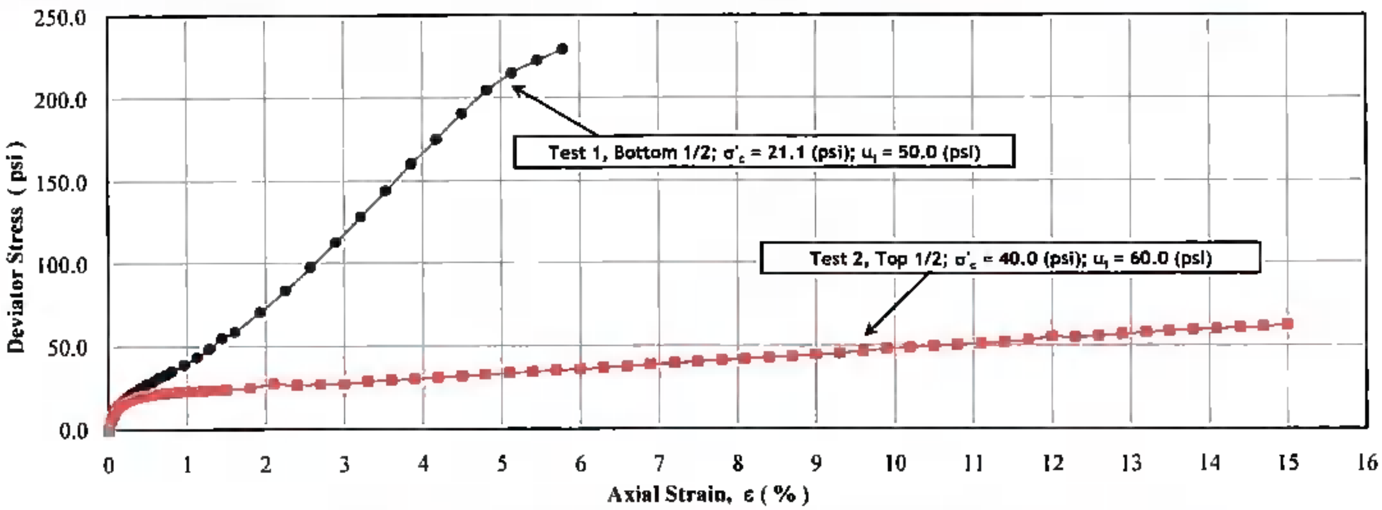
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

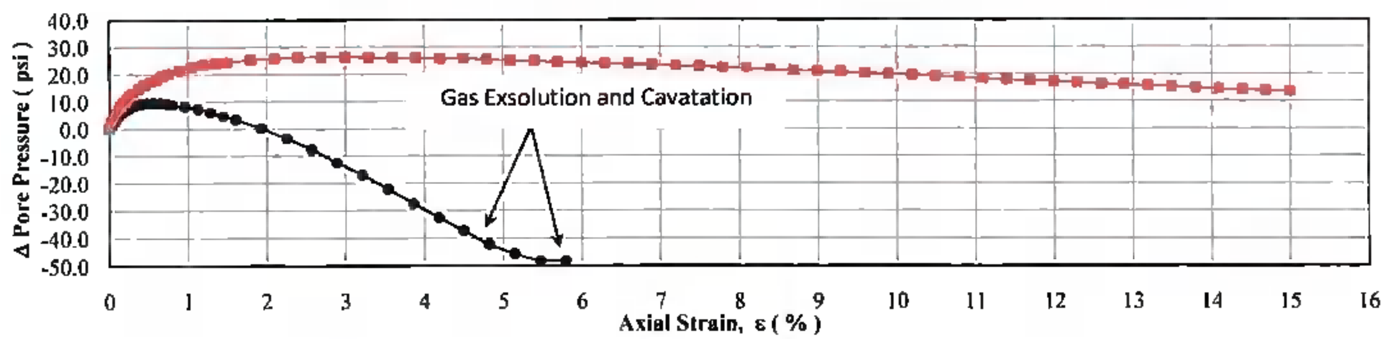
Project Name:	Winyah Generating Station
Project No:	585
Site Sample ID:	GSB-08 (36.5-38.5')
Lab Sample No:	13C164

ASTM D 4767

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

Figure 1





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	229.5	250.0	20.5	50.6	5.8
2	61.9	88.7	26.8	73.2	15.0

Test Specimen No.	Strength at App. 15% Axial Strain				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	229.5	250.0	20.5	50.6	5.8
2	61.9	88.7	26.8	73.2	15.0

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi) u = Pore pressure,(psi) ε_a = Axial strain, (%)

σ'_1 = Effective axial stress, (psi) σ'_3 = Effective radial stress (confining pressure), (psi) $\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)

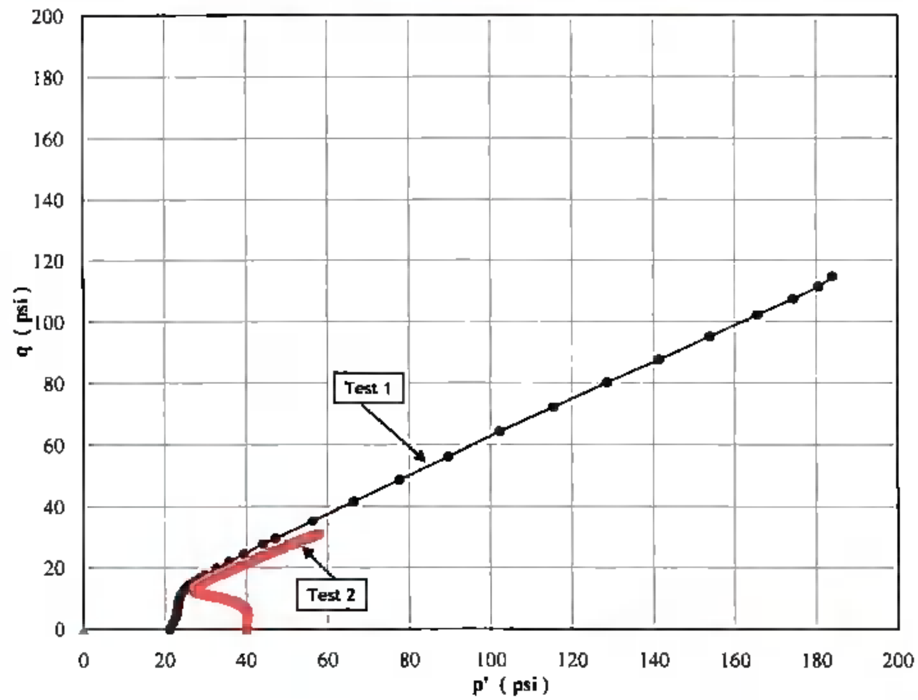
4-25-13
NSR

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:	585
	Site Sample ID:	GSB-08 (36.5-38.5')
	Lab Sample No:	13C164

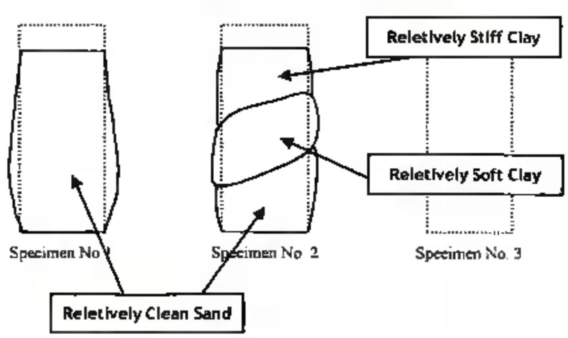
ASTM D 4767

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

Figure 2



Test Specimen No.	Initial Conditions							Strain Rate	Specimen Quality Bad to Good
	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_i (psi)	σ'_c (psi)	(% / min)	(1 to 10)
1	6.25	2.85	25.0	101.2	0.95	50.0	21.1	0.096	5
2	6.82	2.80	22.3	106.2	0.95	60.0	40.0	0.088	5



Notes:

4-25-13
NSR

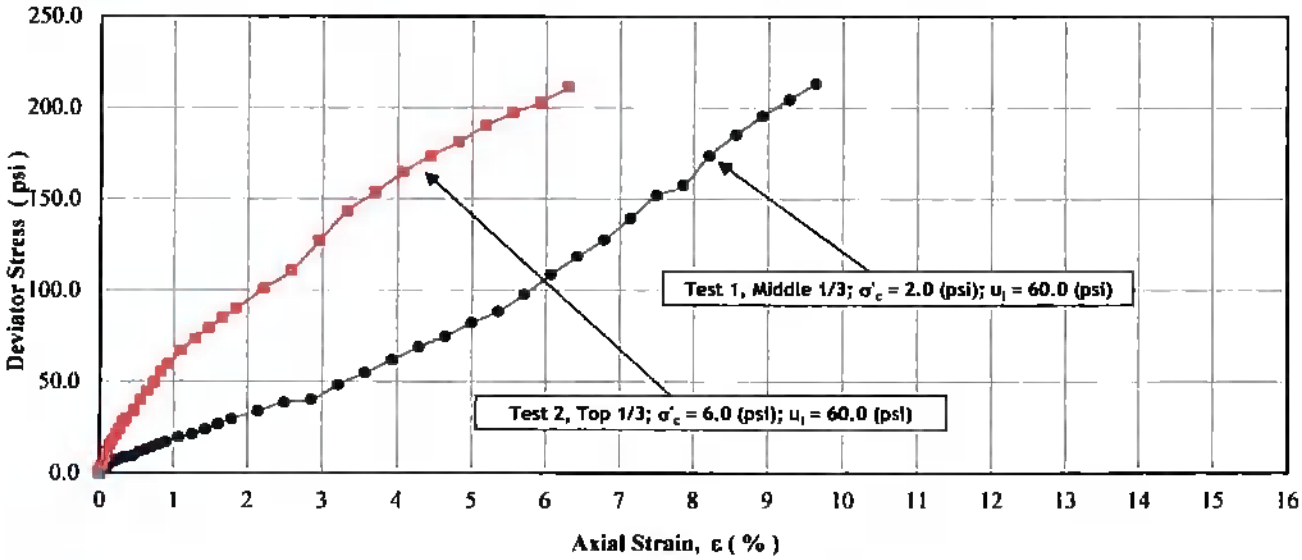
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

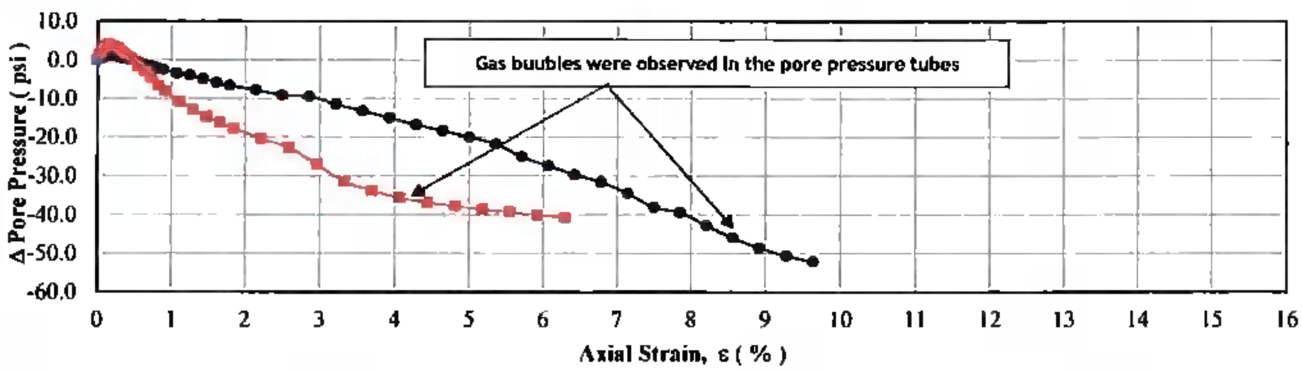
Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-106A, S-01 (8.0-10.0')
Lab Sample No:	13K141

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	213.2	267.3	54.1	7.9	9.6
2	211.2	257.9	46.7	19.3	6.3

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	213.2	267.3	54.1	7.9	9.6
2	211.2	257.9	46.7	19.3	6.3

Notes:

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

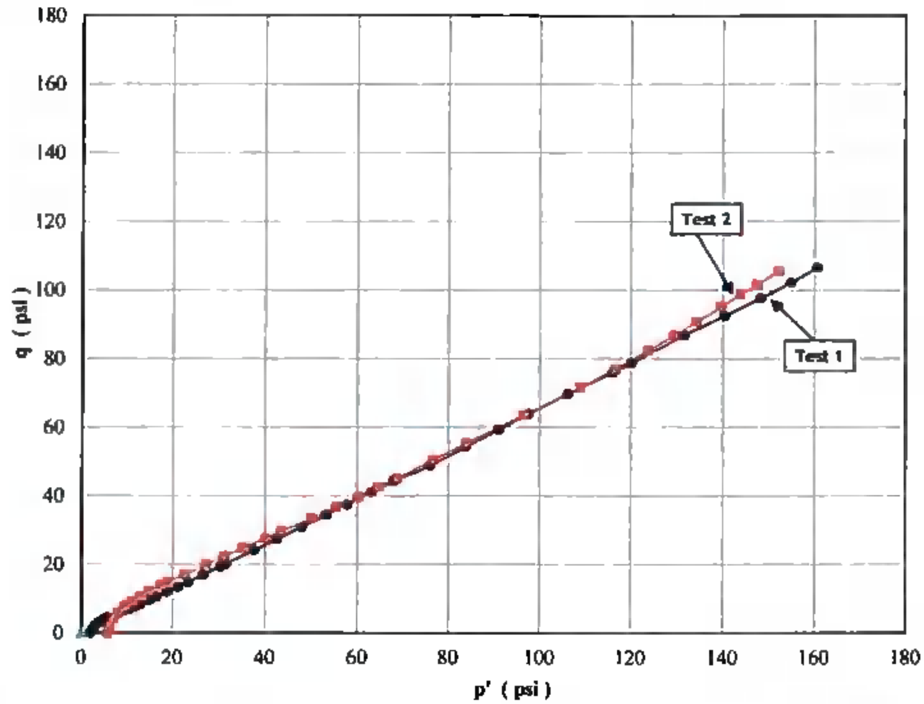
2-2-14
NJR

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 **Site Sample ID:** SPT-106A, S-01 (8.0-10.0') **Lab Sample No:** 13K141

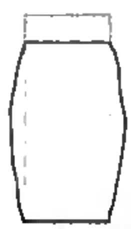
ASTM D 4767

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

Figure 2



Test Specimen No.	Initial Conditions							Axial	Specimen
	Height	Diameter	Moisture Content	Dry Unit Weight	B Parameter	Initial Pore Pressure (u_i)	Consolidation Pressure (σ'_c)	Strain	Quality Bad to Good
	(in.)	(in.)	(%)	(pcf)	(-)	(psi)	(psi)	(% / min)	(1 to 10)
1	5.92	2.81	25.3	94.0	0.99	60.0	2.0	0.051	4
2	5.89	2.79	37.6	83.5	0.96	60.0	6.0	0.051	7



Specimen No.1

Yellow sandy silt (gypsum)

Specimen No. 2

Yellow sandy silt (gypsum)

Specimen No. 3

N/A

Notes:

2-2-14 NSR

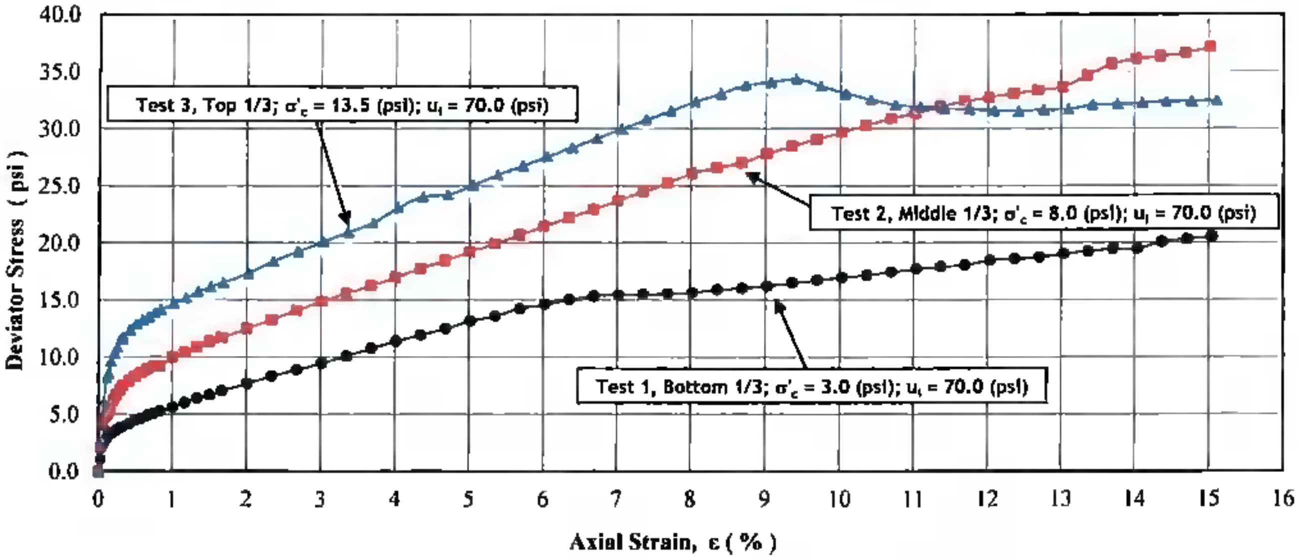
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

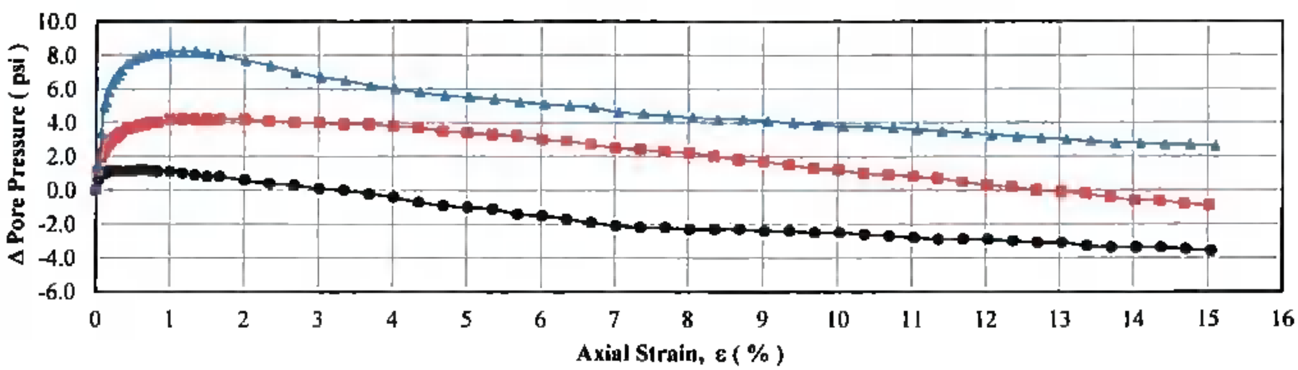
Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-106B, S-03 (14.0-16.0')
Lab Sample No:	13K147

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	20.5	27.1	6.6	66.4	15.0
2	37.1	46.0	8.9	69.1	15.0
3	34.4	43.9	9.5	74.0	9.4

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	20.5	27.1	6.6	66.4	15.0
2	37.1	46.0	8.9	69.1	15.0
3	32.5	43.4	10.9	72.6	15.1

Notes:

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

2-2-14
NSR

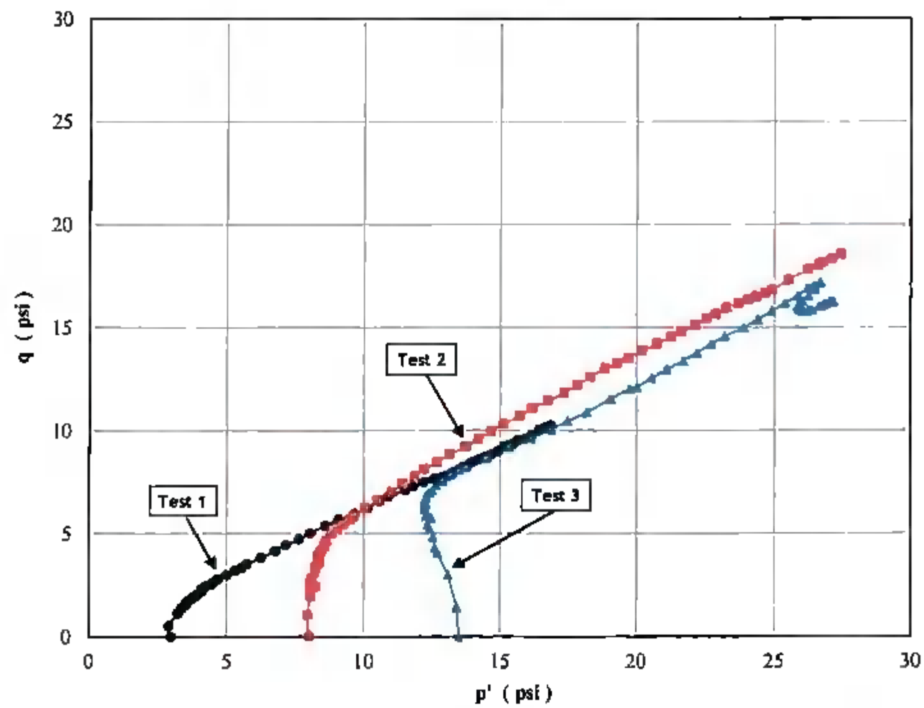
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-106B, S-03 (14.0-16.0')
Lab Sample No:	13K147

ASTM D 4767

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

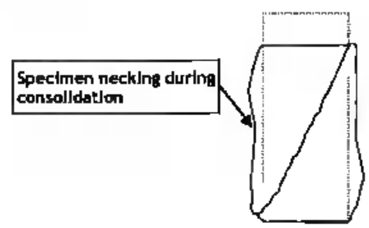
Figure 2



Test Specimen No	Initial Conditions							Axial	Specimen
	Height (in)	Diameter (in)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	Initial Pore Pressure (u_i) (psi)	Consolidation Pressure (σ'_c) (psi)	Strain (% / min)	Quality Bad to Good (1 to 10)
1	6.02	2.78	55.5	66 2	0.99	70.0	3.0	0.050	7
2	6.13	2.77	82 5	51.8	0.98	70 0	8 0	0.049	7
3	6 26	2.74	68 9	56.1	1 00	70 0	13 5	0 048	4

Specimen No. 1
Black silty sand (Gypsum)

Specimen No. 2
Black silty sand (Gypsum)



Specimen No. 3
Gray silty sand (Gypsum)

Notes:

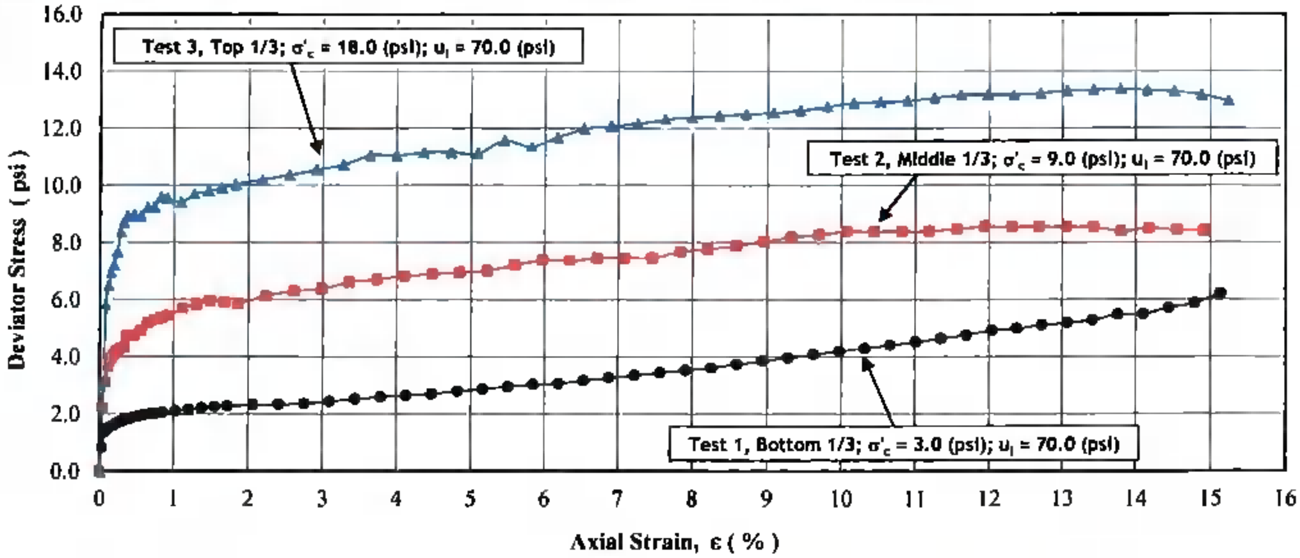
2-2-14
NSK

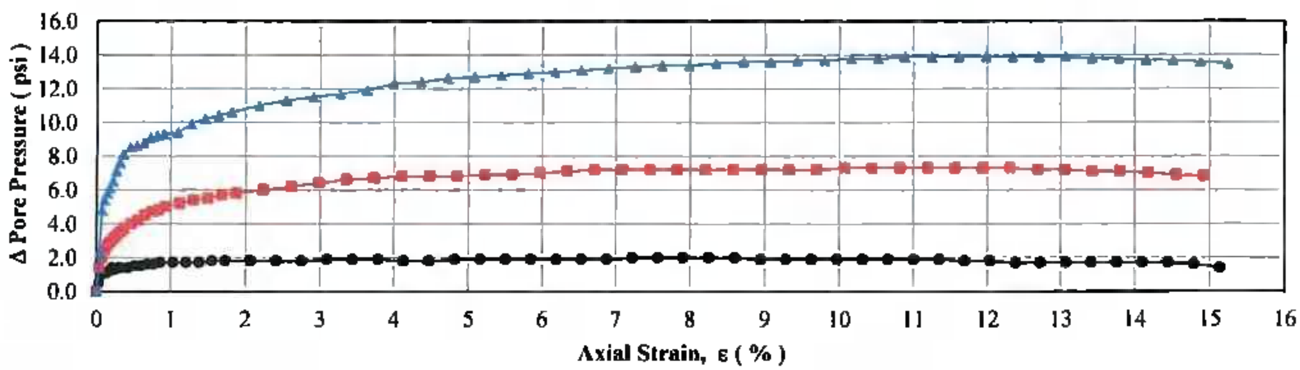
Excel Geotechnical Testing, Inc. "Excellence in Testing" 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	**Project Name:** Winyah Generating Station **Project No:** 618 **Site Sample ID:** SPT-107A, S-03 (14.0-16.0') **Lab Sample No:** 13K151

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	6.2	7.8	1.6	71.4	15.1
2	8.5	10.2	1.7	77.3	12.3
3	13.4	17.6	4.2	83.8	13.8

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	6.2	7.8	1.6	71.4	15.1
2	8.4	10.6	2.2	76.8	14.9
3	13.0	17.5	4.5	83.5	15.2

Notes:

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

2-2-14 NJR

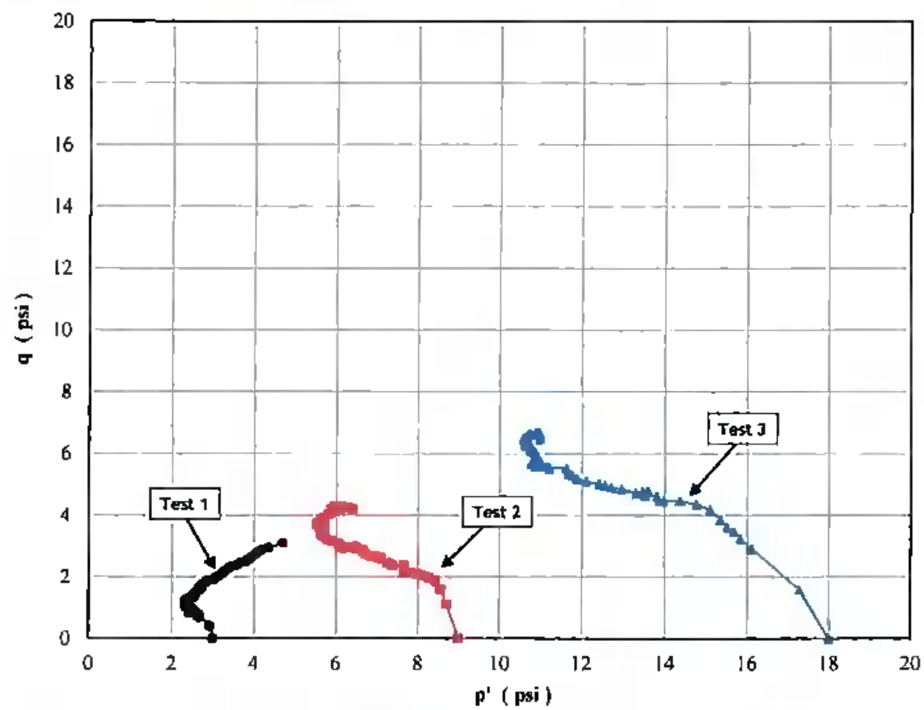
Excel Geotechnical Testing, Inc.
"Excellence In Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-107A, S-03 (14.0-16.0')
Lab Sample No:	13K151

ASTM D 4767

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

Figure 2



Test Specimen No	Initial Conditions							Axial	Specimen
	Height	Diameter	Moisture Content	Dry Unit Weight	B Parameter	Initial Pore Pressure (u_i)	Consolidation Pressure (σ'_c)	Strain	Quality Bad to Good
	(in)	(in)	(%)	(pcf)	(-)	(psi)	(psi)	(% / min)	(1 to 10)
1	6.46	2.79	71.4	57.4	0.99	70.0	3.0	0.046	6
2	6.25	2.83	89.1	51.1	0.98	70.0	9.0	0.048	7
3	6.55	2.87	91.2	46.8	0.99	70.0	18.0	0.046	4

Specimen No.1
Black silty sand (Gypsum)

Specimen No. 2
Black silty sand (Gypsum)

Specimen No. 3
Gray silty sand (Gypsum)

Notes:

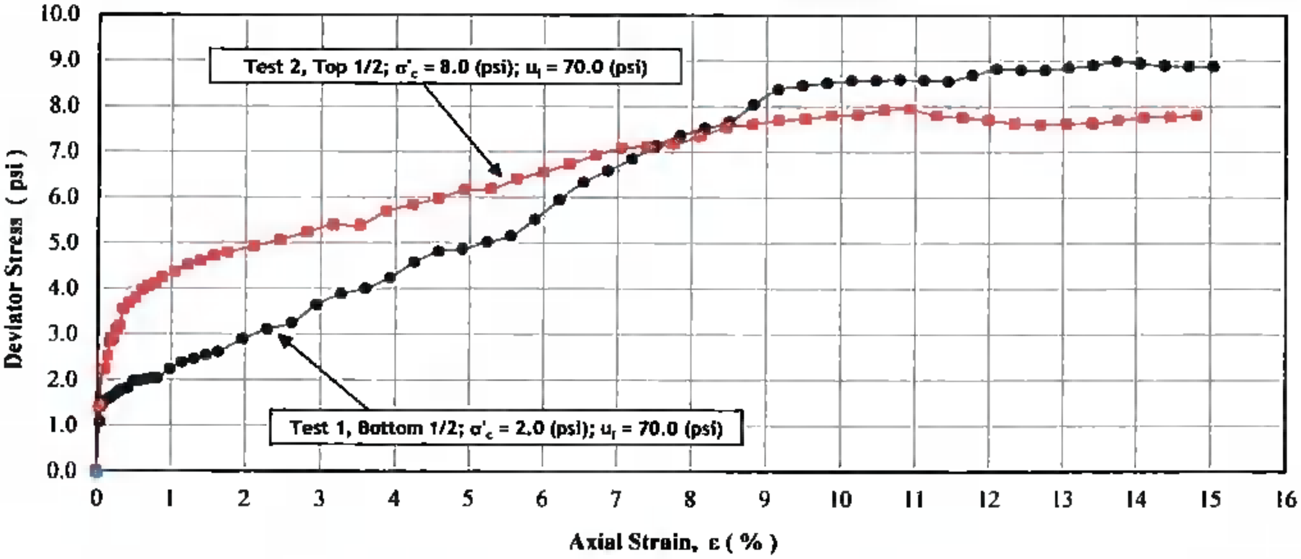
2-2-14
NJK

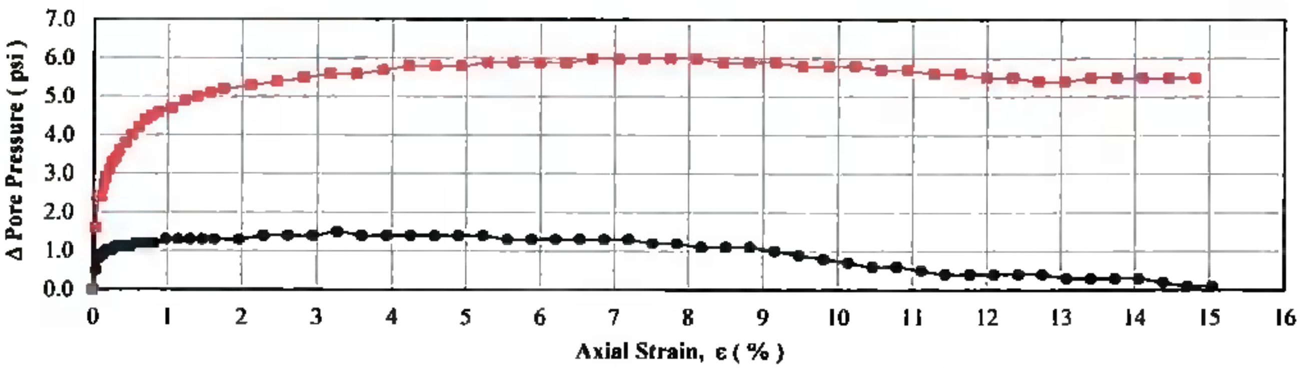
Excel Geotechnical Testing, Inc. *"Excellence in Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	**Project Name:** Winyah Generating Station **Project No:** 618 **Site Sample ID:** SPT-107B, S-01 (8.0-10.0') **Lab Sample No:** 13K153

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	9.0	10.8	1.8	70.3	13.7
2	7.9	9.2	1.3	75.7	10.9

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	8.9	10.9	2.0	70.1	15.0
2	7.8	9.3	1.5	75.5	14.8

Notes:

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

2-2-14
NSR

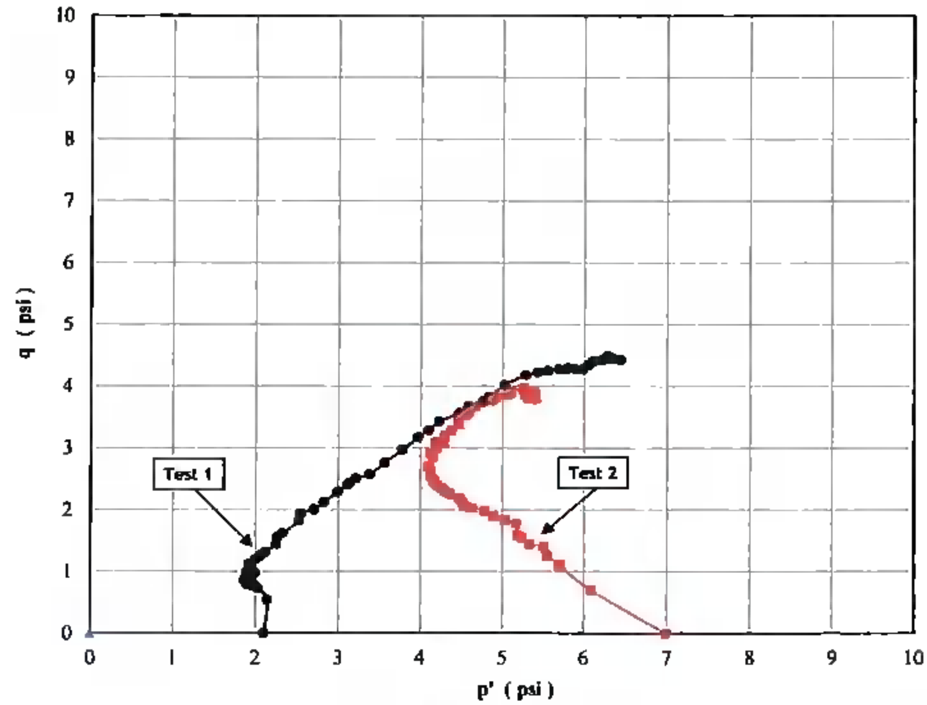
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Site Sample ID:	SPT-107B, S-01 (8.0-10.0')
Lab Sample No:	13K153

ASTM D 4767

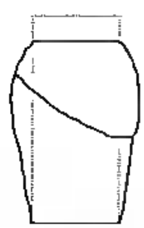
CONSOLIDATED-UNDRAINED (CU) TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENTS

Figure 2



Test Specimen No.	Initial Conditions							Axial Strain	Specimen Quality Bad to Good
	Height (in.)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	Initial Pore Pressure (u_i) (psi)	Consolidation Pressure (σ'_c) (psi)	(% / min)	(1 to 10)
1	6.35	2.80	60.6	65.0	1.00	70.0	2.1	0.047	7
2	6.46	2.83	75.1	55.8	1.00	70.0	7.0	0.046	7

Specimen No.1
Black sandy silt (Gypsum)



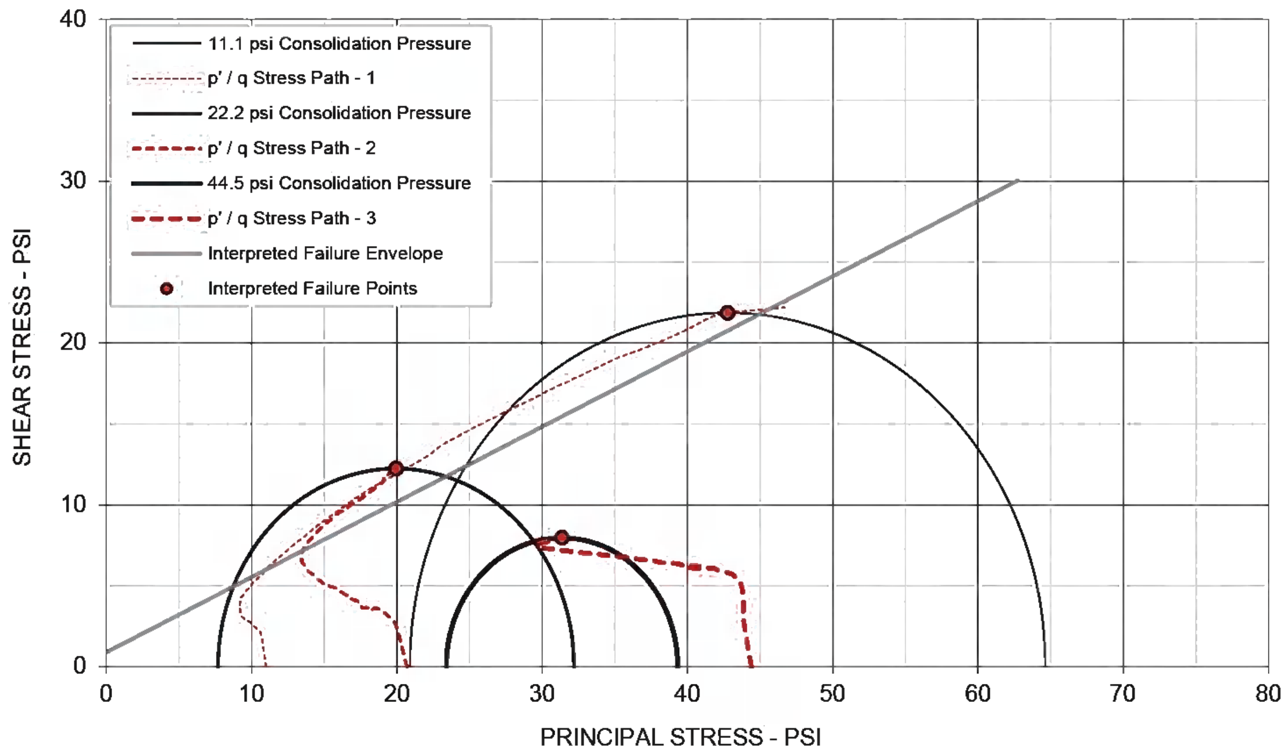
Specimen No. 2
Black sandy silt (Gypsum)

Specimen No. 3
NA

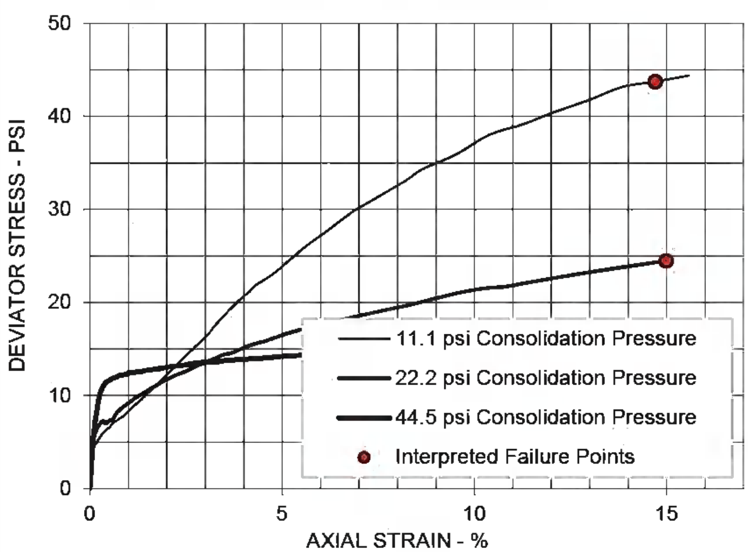
Notes:

2-2-14
NJR

ICU TRIAXIAL COMPRESSION TEST



EFFECTIVE STRESS PARAMETERS	ϕ ' = 24.9 deg	c' = 0.9 psi



SPECIMEN NO.	1	2	3
INITIAL			
Moisture Content - %	18.2	18.2	30.3
Dry Density - pcf	109.8	108.3	92.2
Diameter - inches	2.84	2.85	2.86
Height - inches	5.78	6.00	5.96
AT TEST			
Final Moisture - %	18.7	19.2	22.7
Dry Density - pcf	109.8	108.3	92.3
Calculated Diameter - in.	2.84	2.86	2.86
Height - inches	5.78	6.02	5.97
Effect. Consol. Stress - psi	11.1	22.2	44.5
Failure Stress - psi	43.73	24.49	15.93
Total Pore Pressure - psi	30.2	55.0	62.0
Strain Rate - inches/min.	0.00040	0.00040	0.00040
Failure Strain - %	14.7	15.0	15.1
σ_1' Failure - psi	64.63	32.19	39.33
σ_3' Failure - psi	20.90	7.70	23.40

TEST DESCRIPTION

ISOTROPICALLY CONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION

SAMPLE TYPE: Shelby Tube

DESCRIPTION: Color USCS Description, USCS Symbol

SAMPLE: GSB 11 18' - 20'

ASSUMED SPECIFIC GRAVITY: 2.7

LL: 36 PL: 13 PI: 23 Percent -200: 38.765

REMARKS: XXXXX

PROJECT INFORMATION

PROJECT: Winyah

LOCATION:

PROJECT NO: EN165065

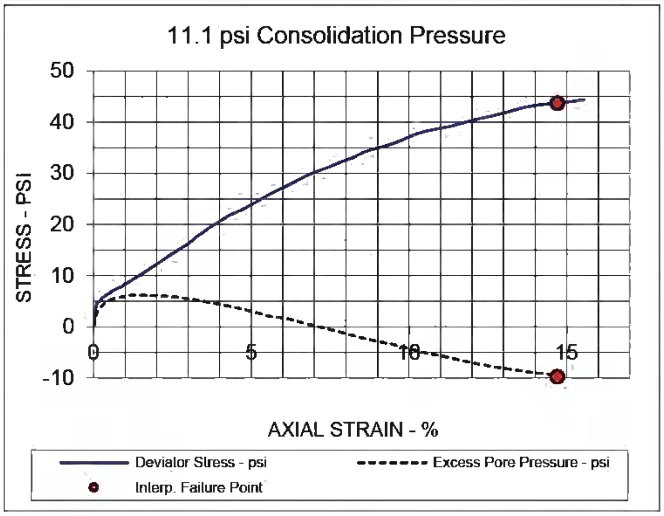
CLIENT:SCE&G

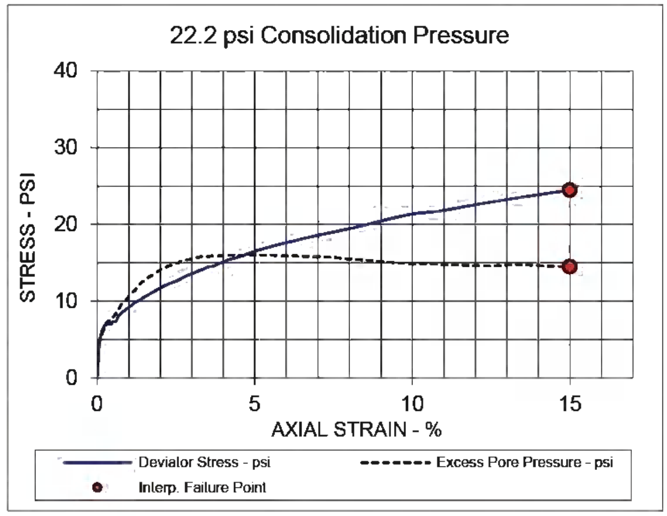
DATE: 05/09/2016

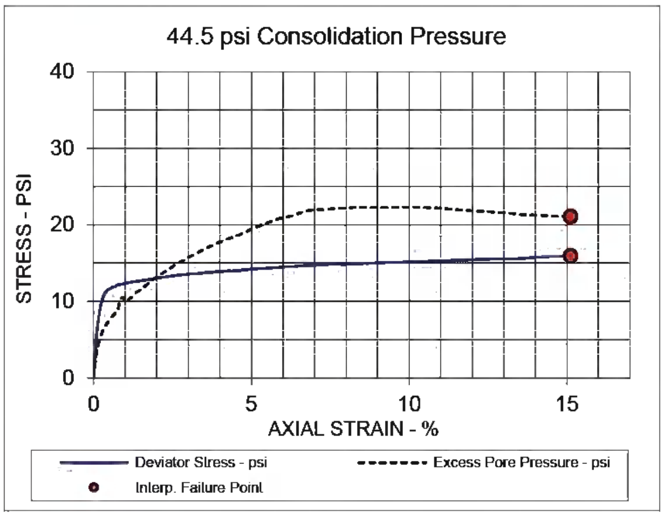
1450 Fifth St W
North Charleston, SC

Terracon

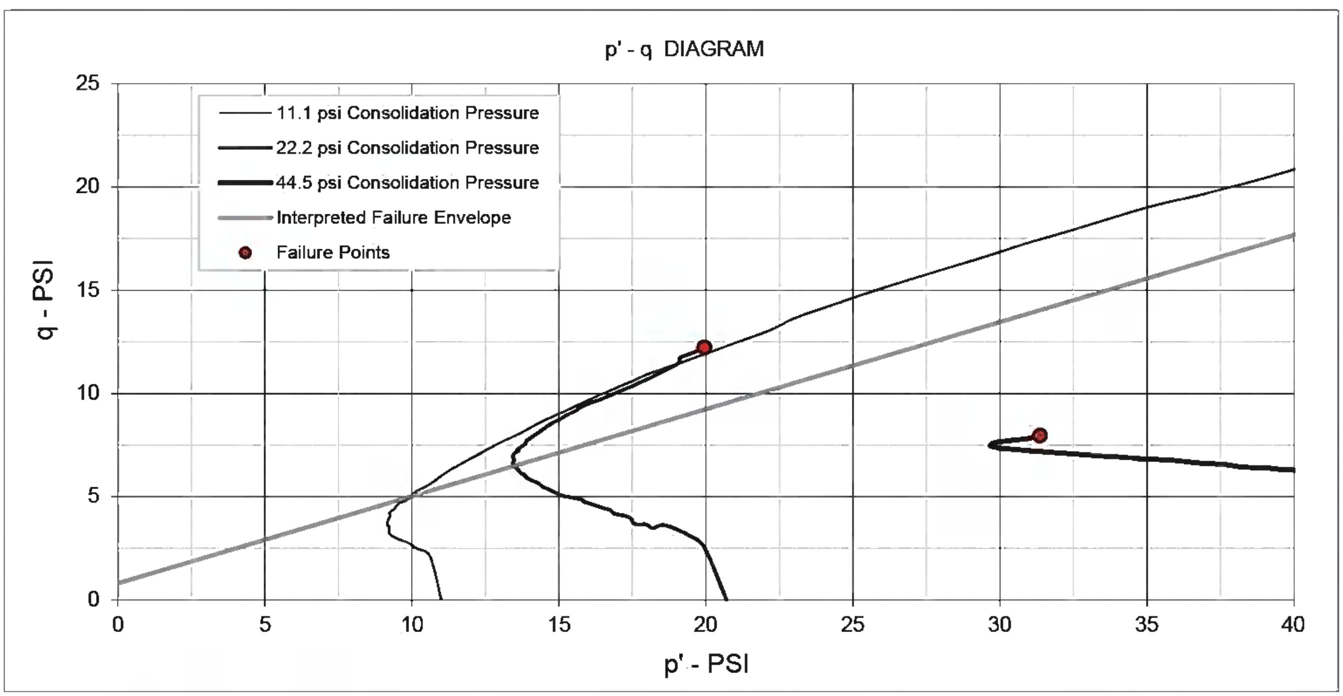
5.9.2016 GSB 11 18-20 CU.xlsx







SPECIMEN FAILURE ILLUSTRATIONS		
1	2	3



EFFECTIVE STRESS PARAMETERS R^2 = 0.68	α = 22.9 deg	a = 0.8 psi
PROJECT: Winyah	ISOTROPICALLY CONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST	
PROJECT NO: EN165065	1450 Fifth St W	Terracon
DESCRIPTION: Color USCS Description, USCS Symbol	North Charleston, SC	

Consolidation/Permeability Testing

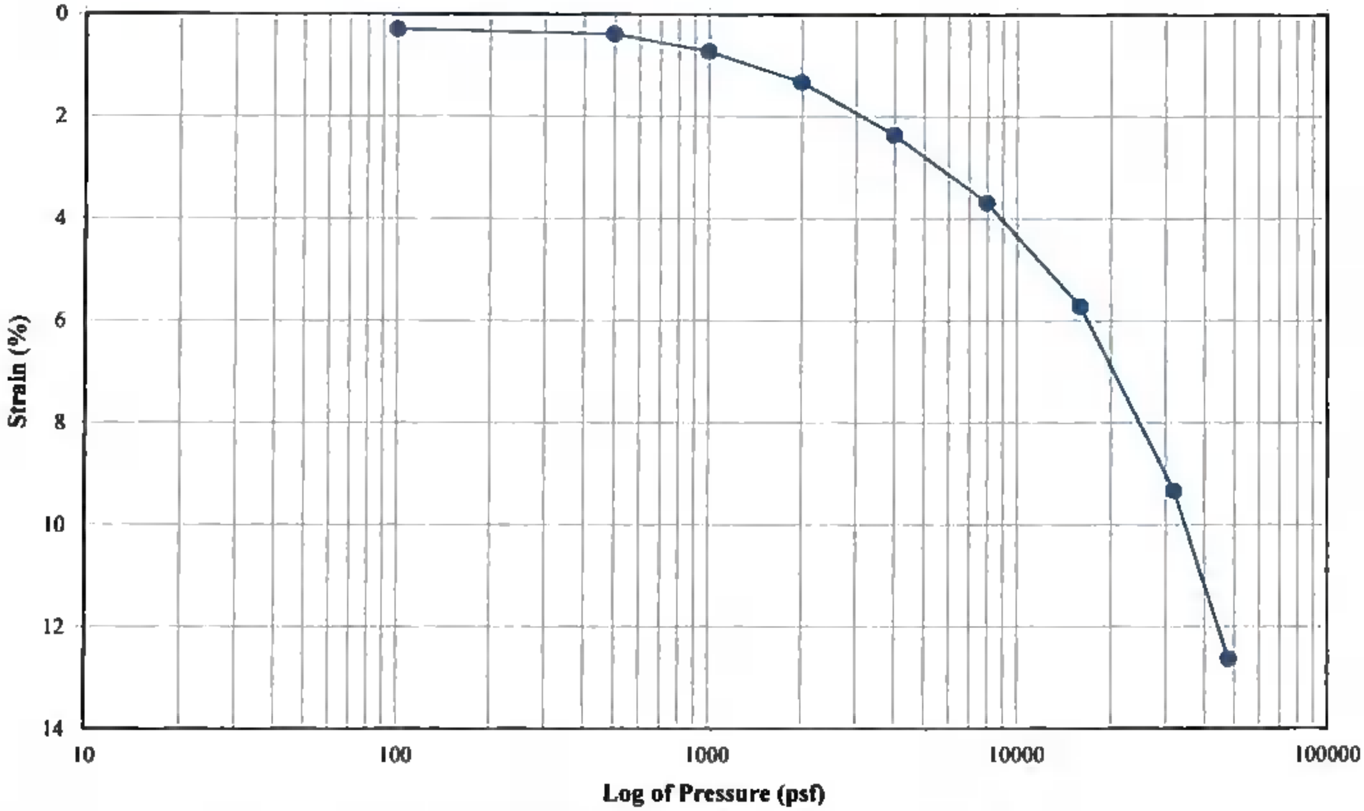
Excel Geotechnical Testing, Inc
"Excellence in Testing"

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

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-106, ST-01 (60.0-62.0')
Lab Sample No: 13J353

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-106, ST-01 (60.0-62.0')	13J353	3	2.54	6.35	73.9	40.7	100	1315	0.29	1	
							500	1475	0.38	2	
							1000	1314	0.72	3	
							2000	1439	1.32	4	
							4000	1369	2.35	5	
							8000	1430	3.68	6	
							16000	1441	5.71	7	
							32000	1489	9.33	8	
							48000	2776	12.63	9	

Notes:

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

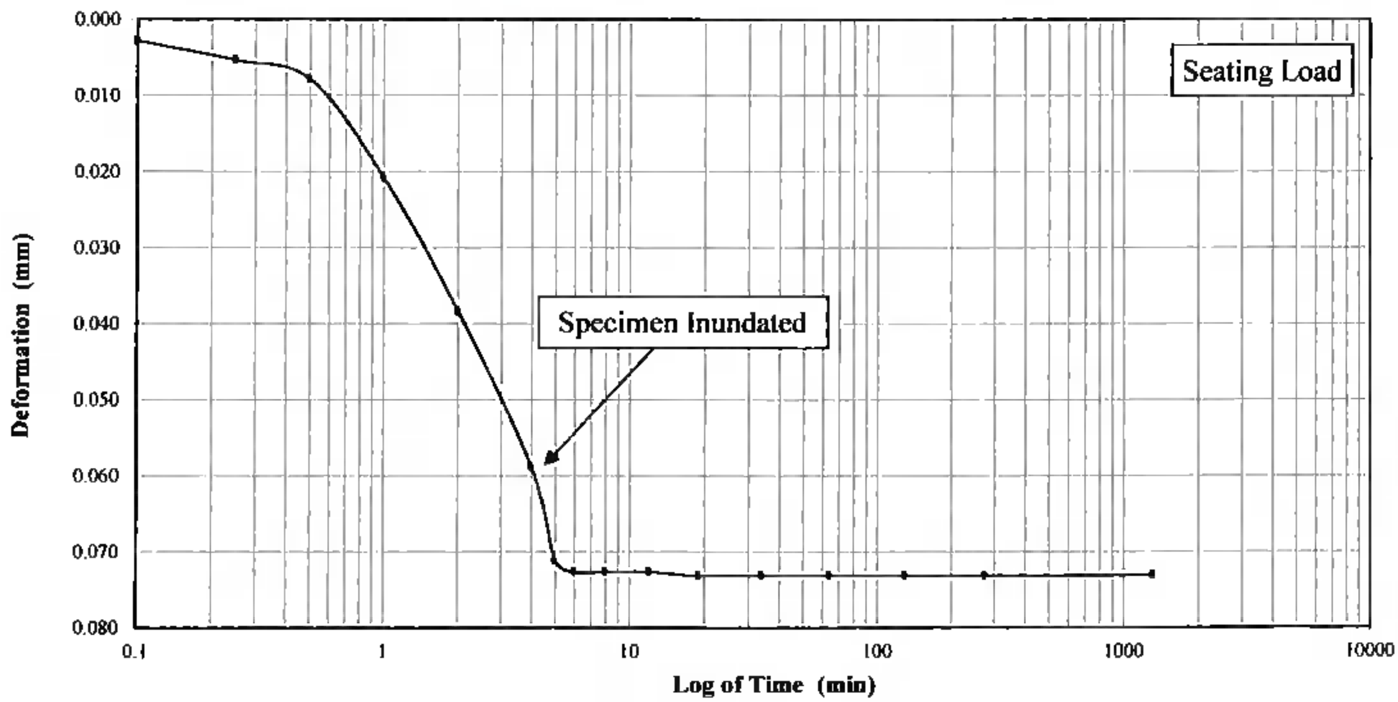
1-04-14
NJR

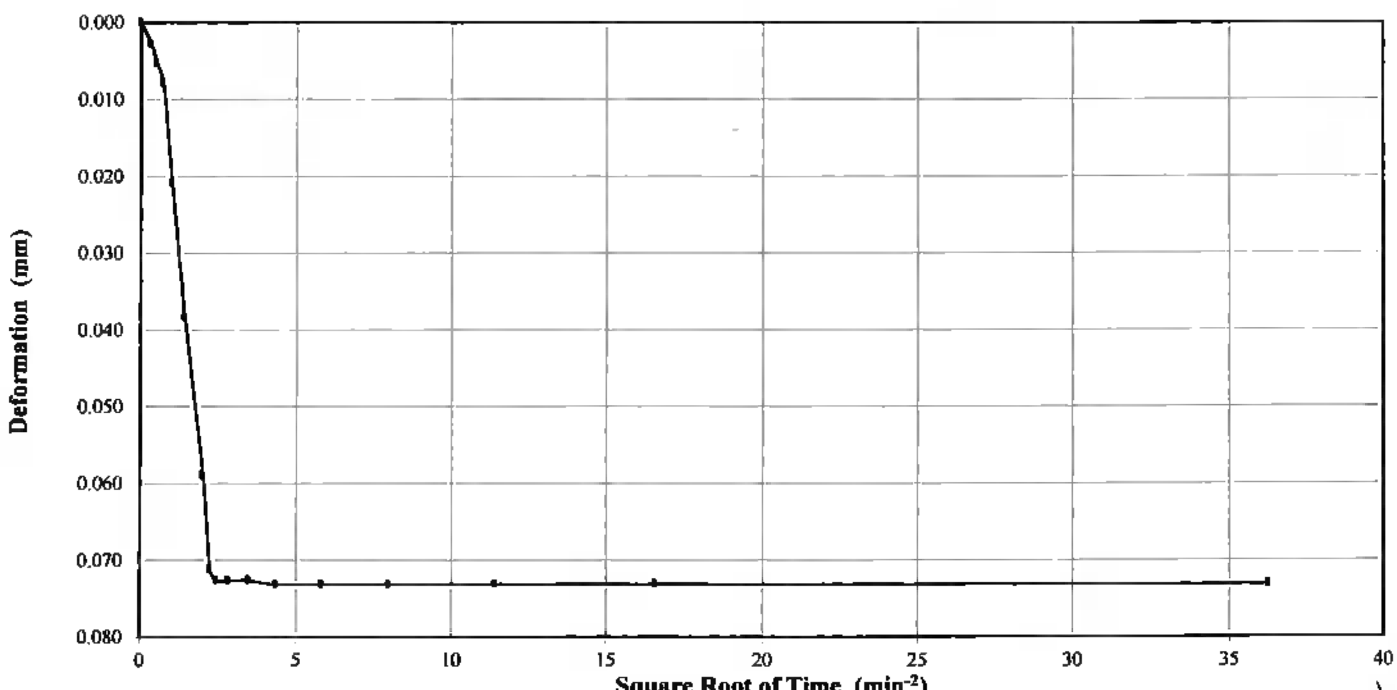
Excel Geotechnical Testing, Inc. *"Excellence in Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-106, ST-01 (60.0-62.0') Lab Sample No: 13J353

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





1-04-14
NSK

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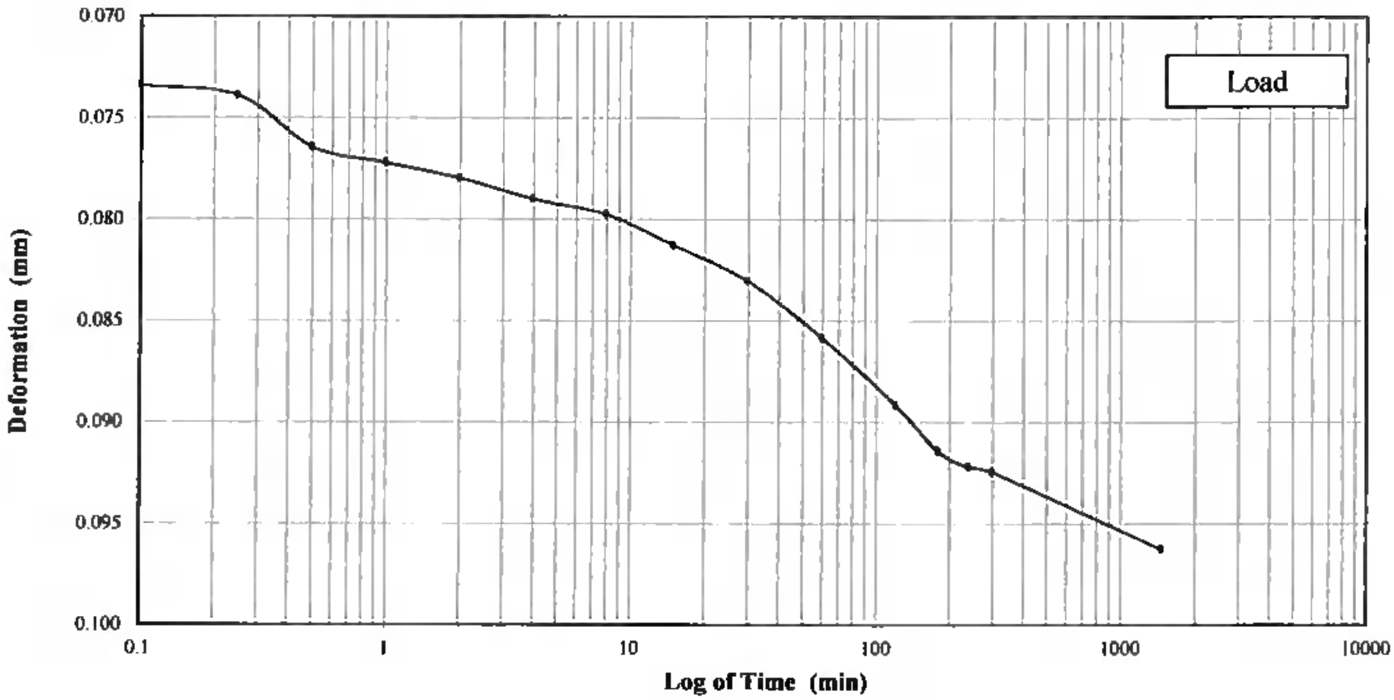
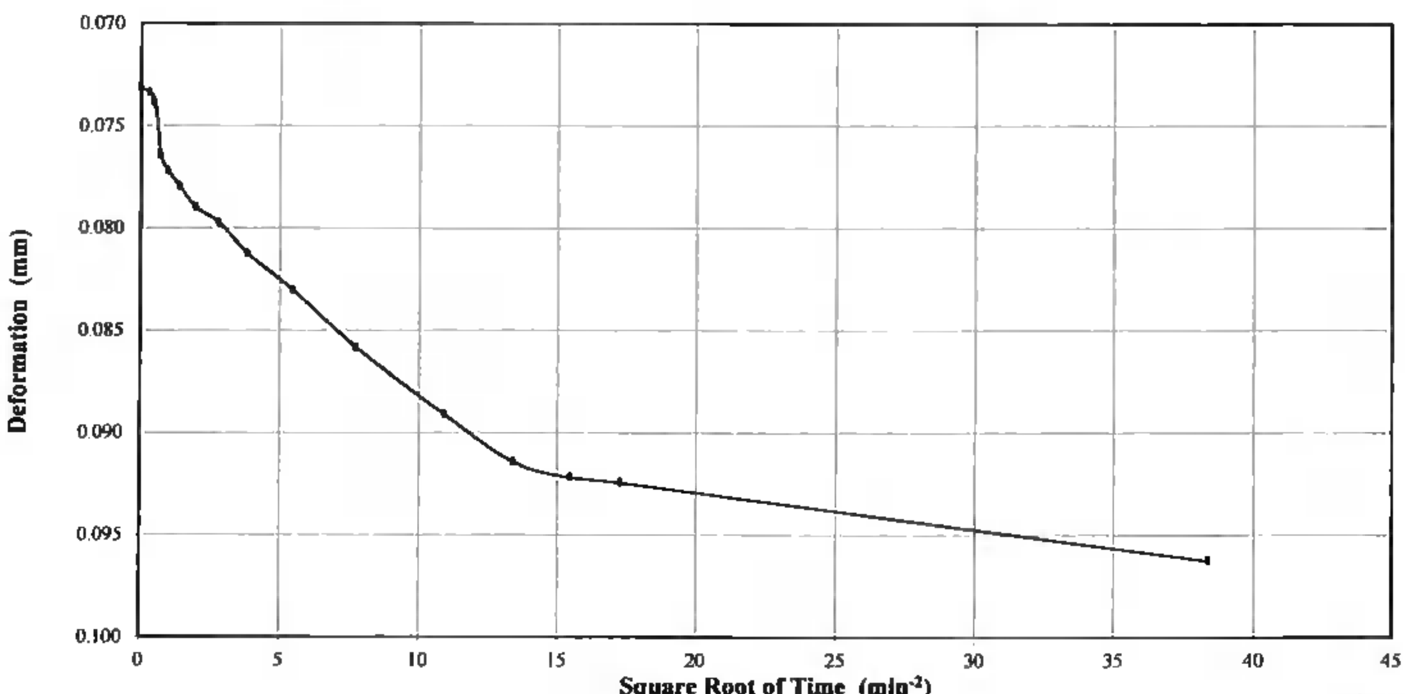
Project No:	618
Client Sample ID:	SPT-106, ST-01 (60.0-62.0')
Lab Sample No:	13J353

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf

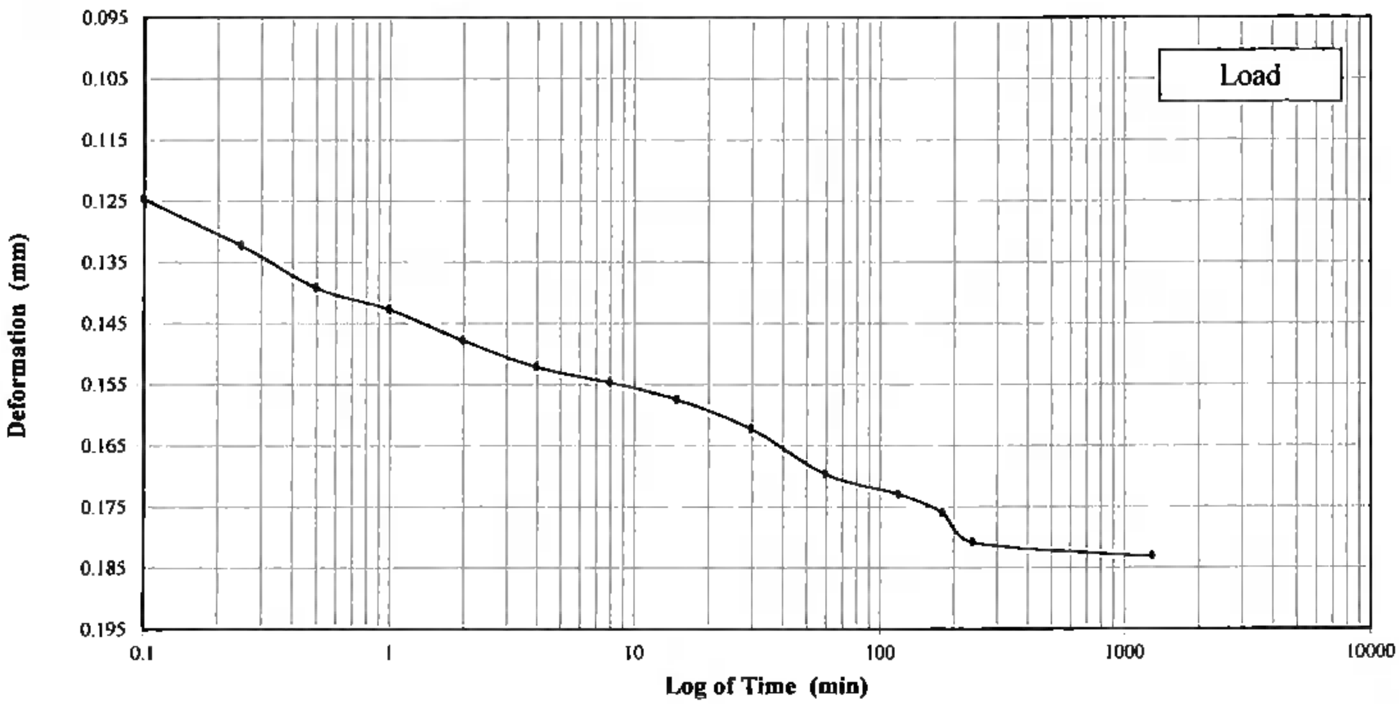


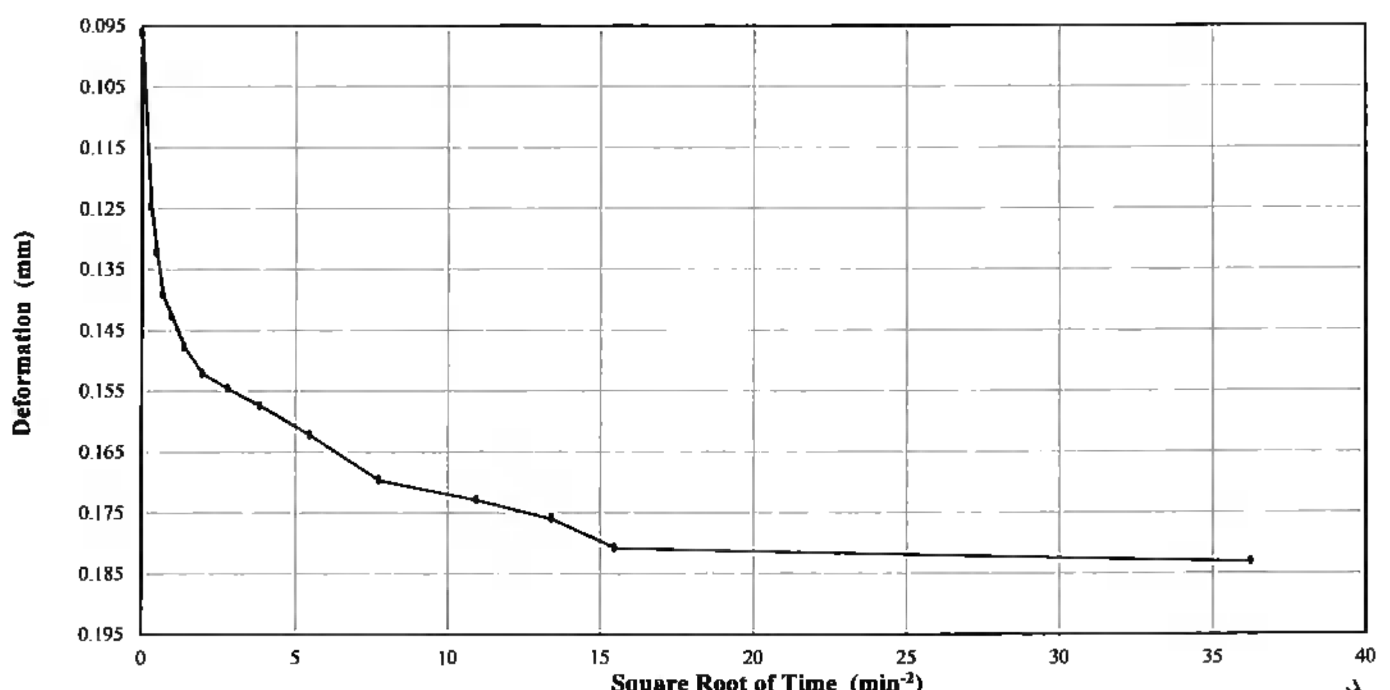


1-04-14
NSR

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-106, ST-01 (60.0-62.0') Lab Sample No: 13J353

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 3 - 1000 psf





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Project No: 618
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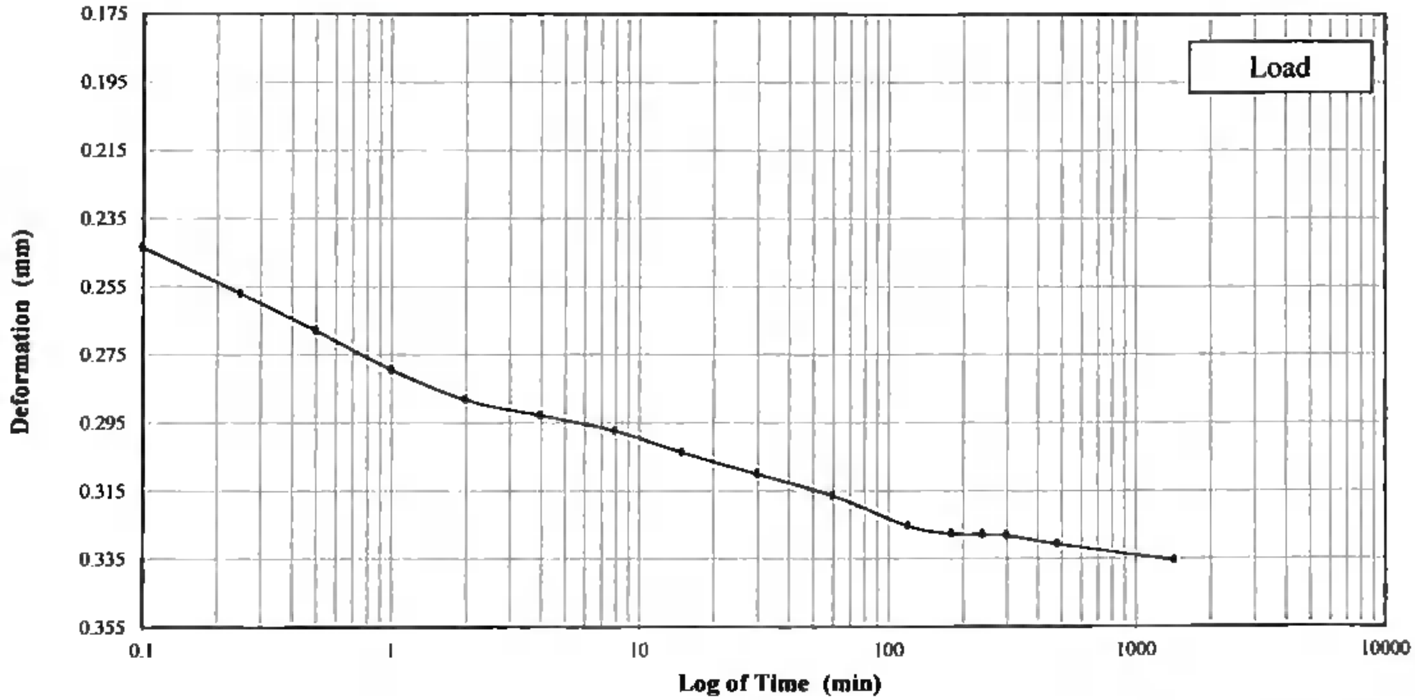
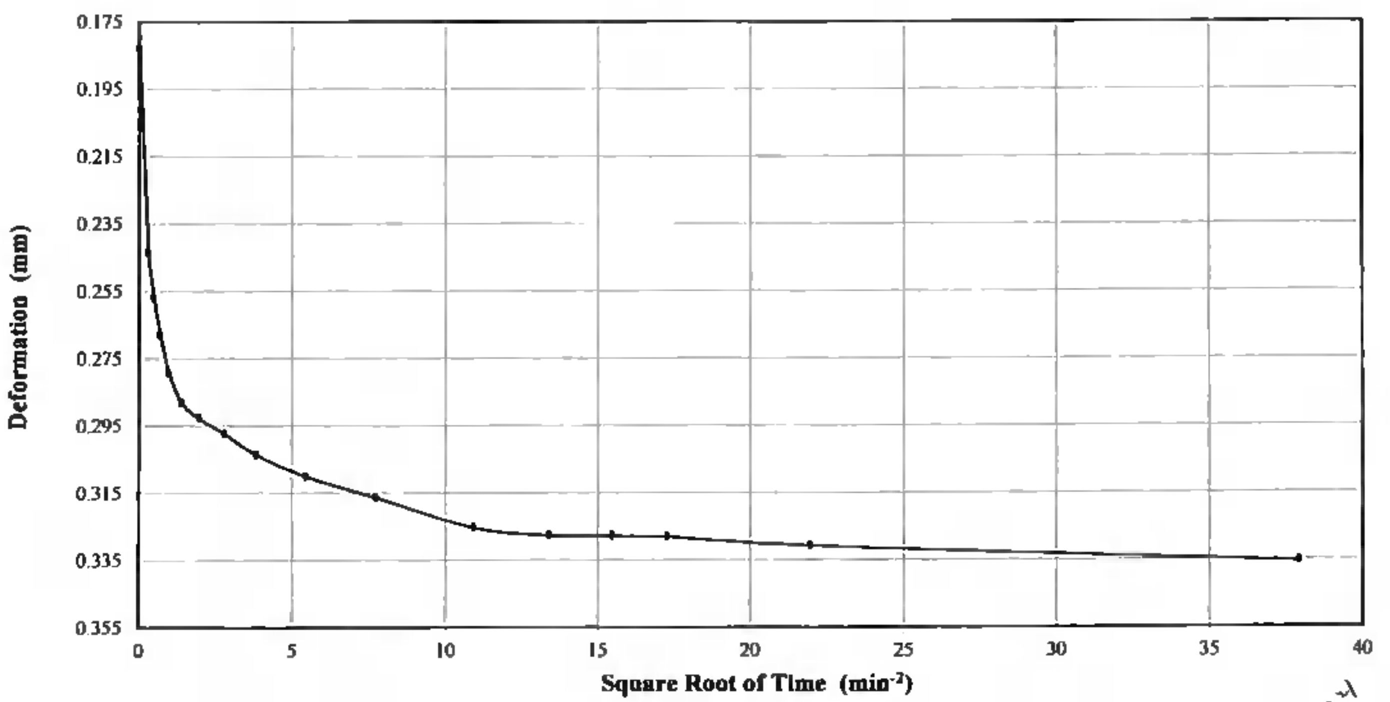
Lab Sample No: 13J353

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf





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953 Forrest Street, Roswell, Georgia 30075
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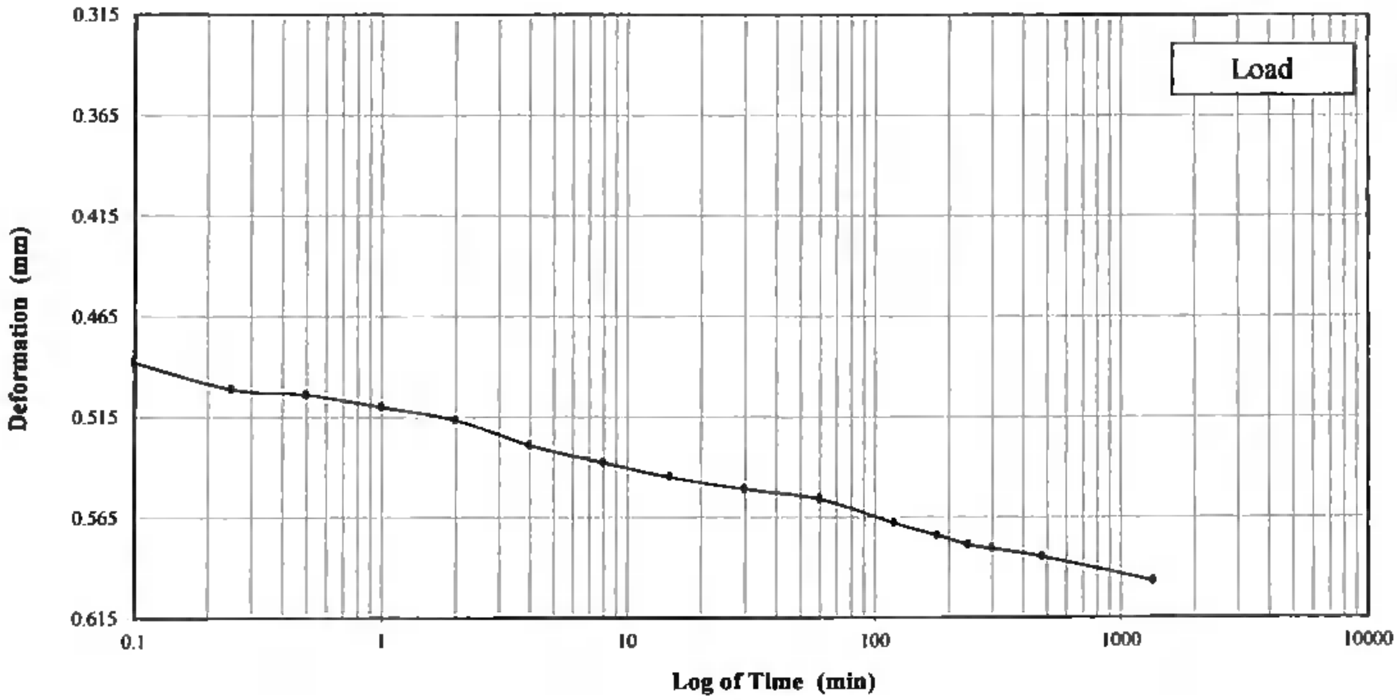
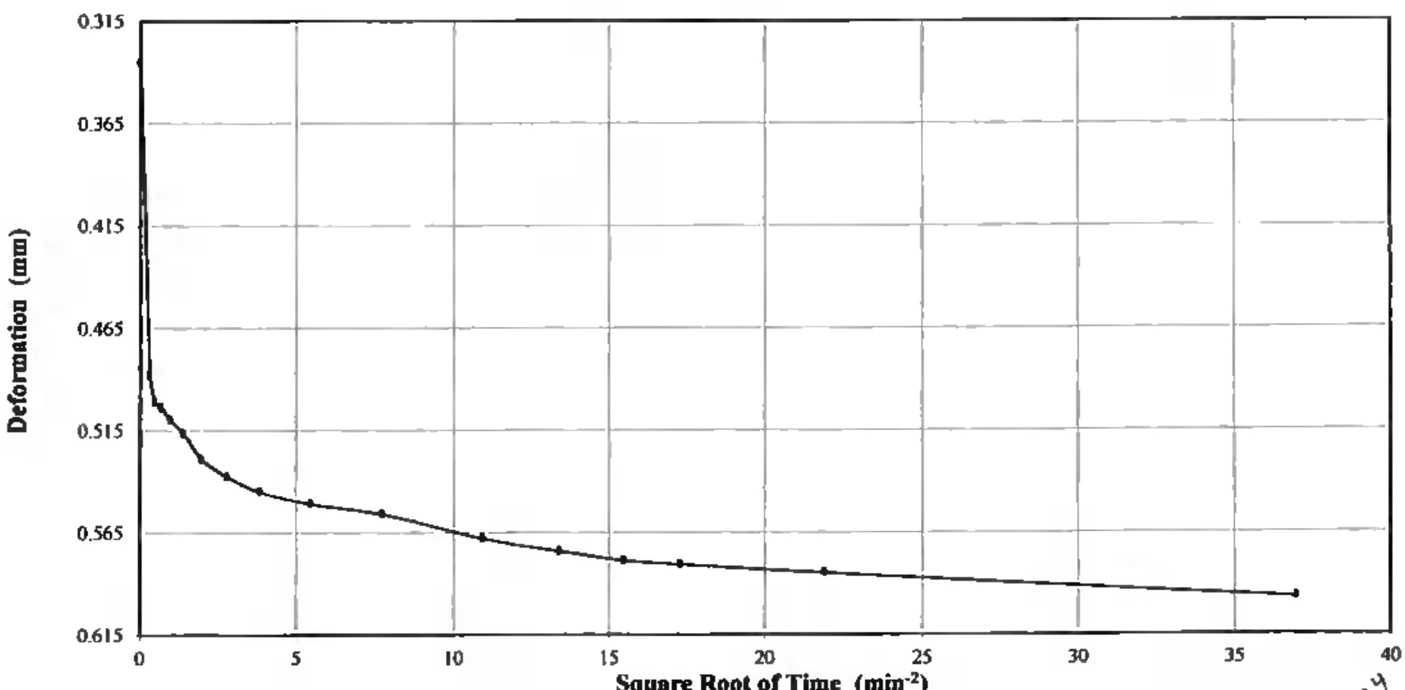
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-106, ST-01 (60.0-62.0')
Lab Sample No: 13J353

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 5 - 4000 psf





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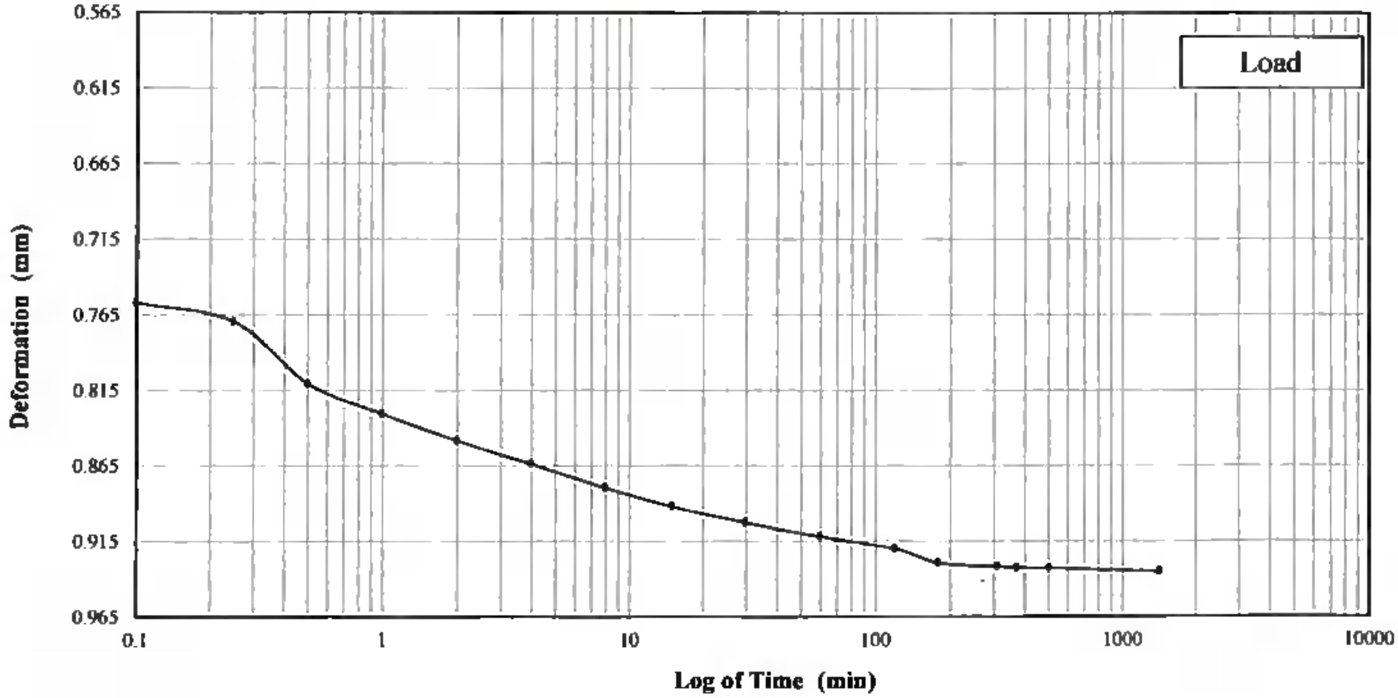
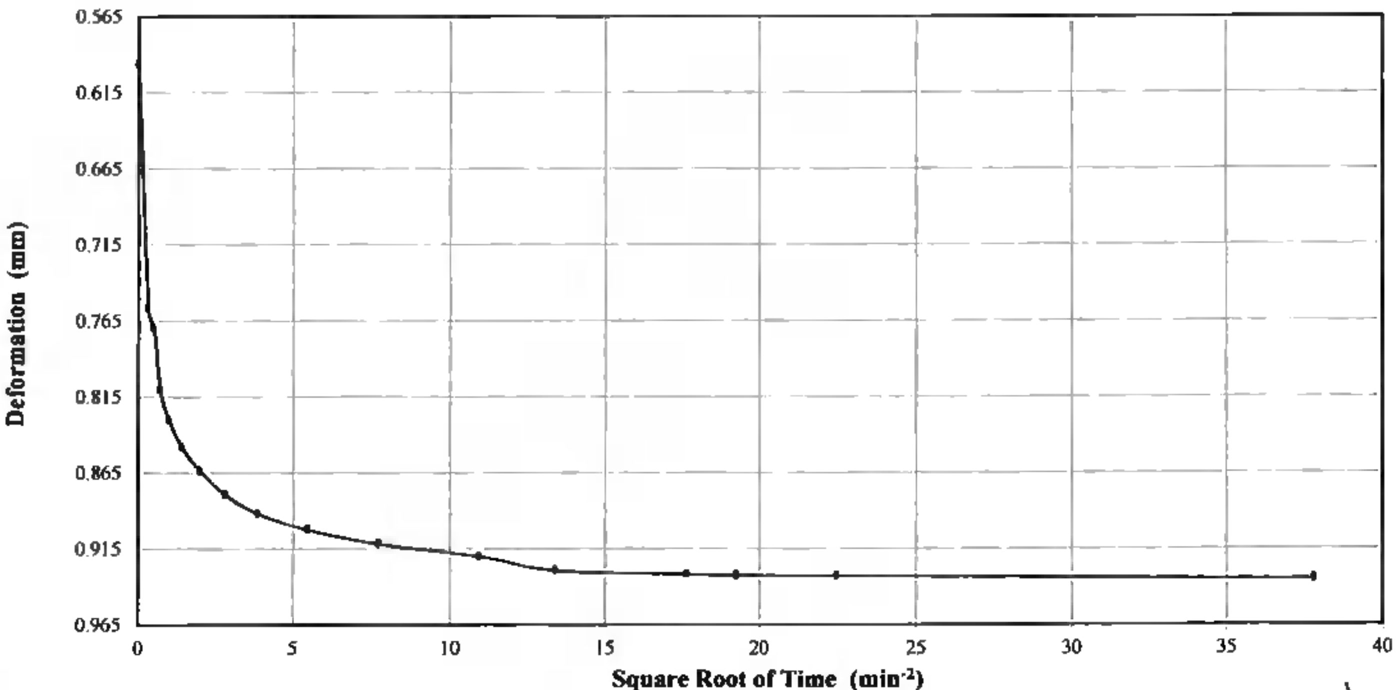
Project No: 618
Client Sample ID: SPT-106, ST-01 (60.0-62.0')
Lab Sample No: 13J353

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 6 - 8000 psf





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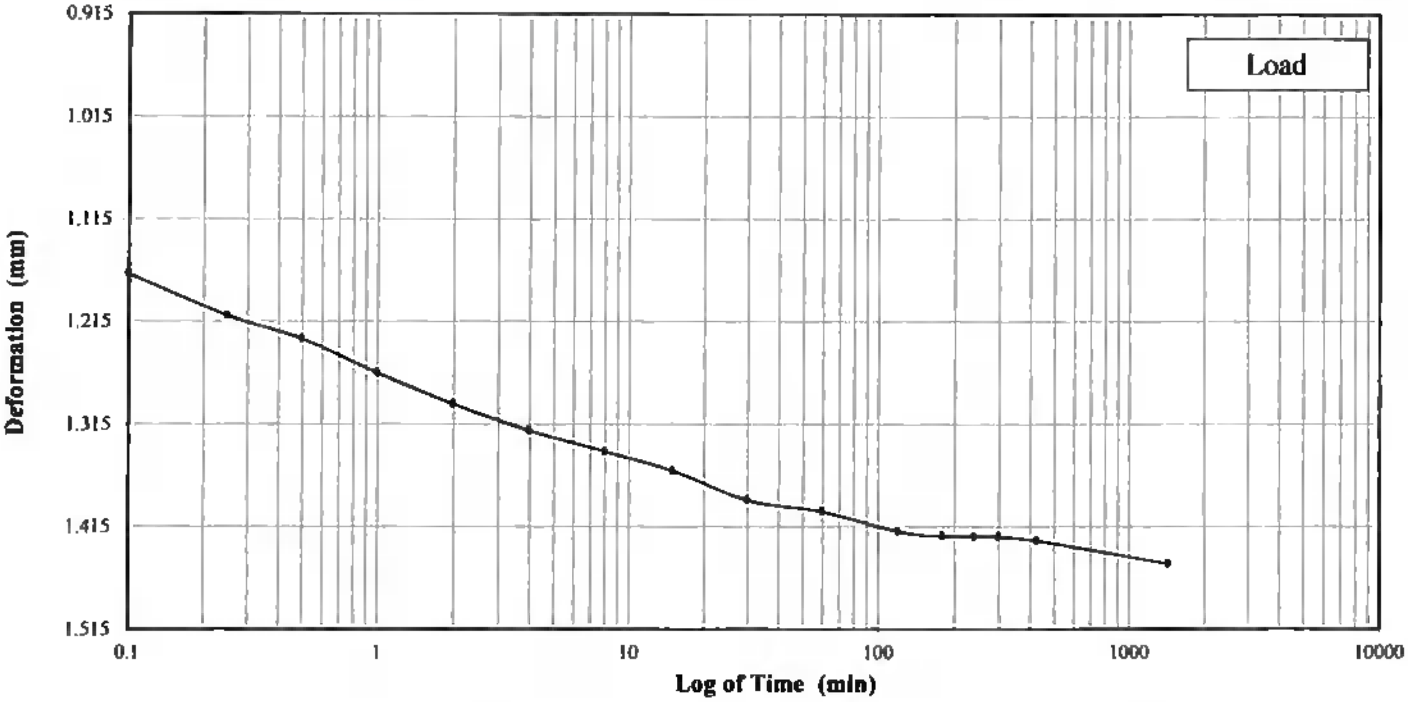
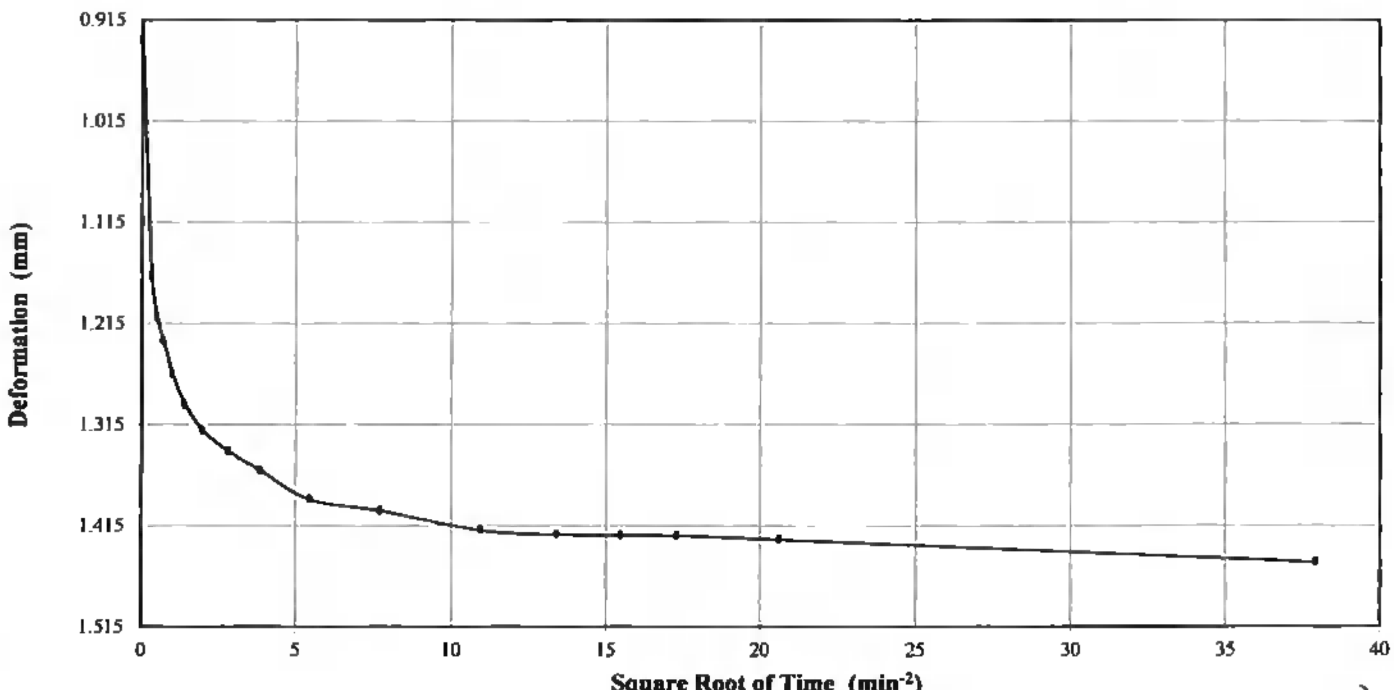
Excel Geotechnical Testing, Inc. *"Excellence in Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-106, ST-01 (60.0-62.0') Lab Sample No: 13J353

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** **Figure 7 - 16000 psf**





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"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
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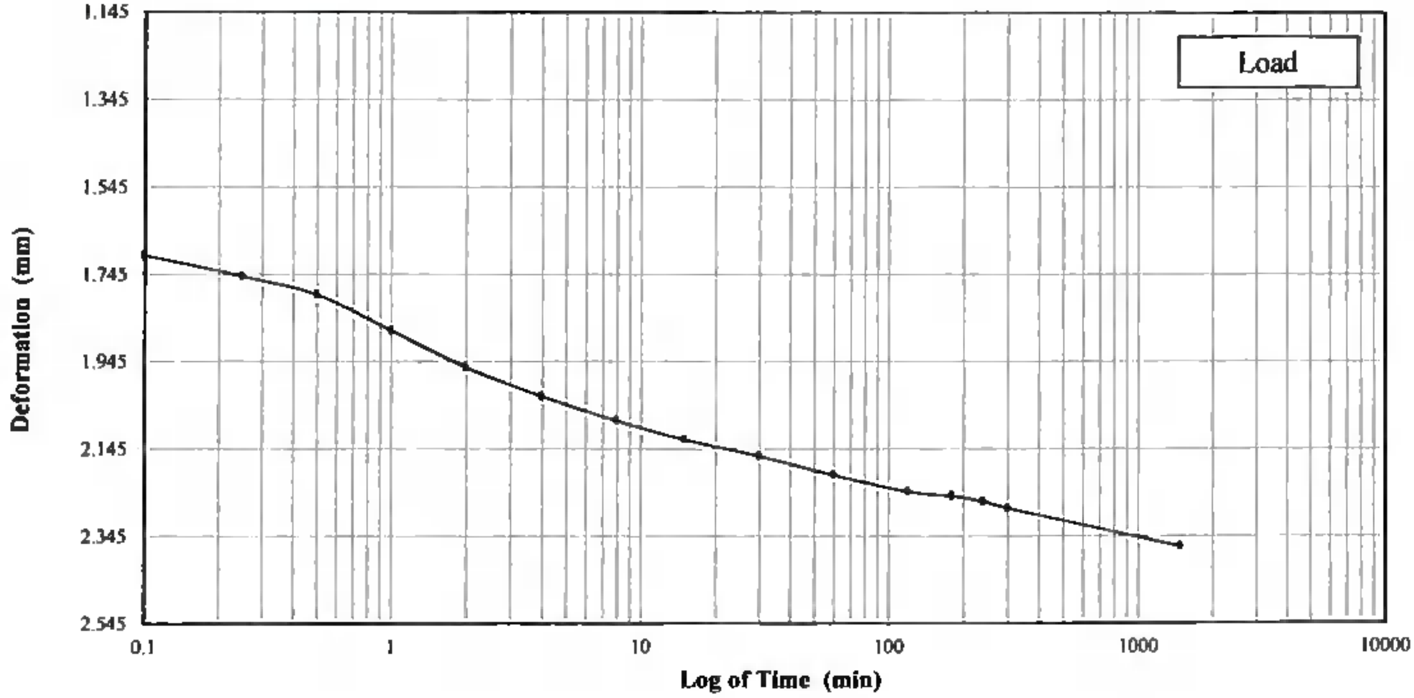
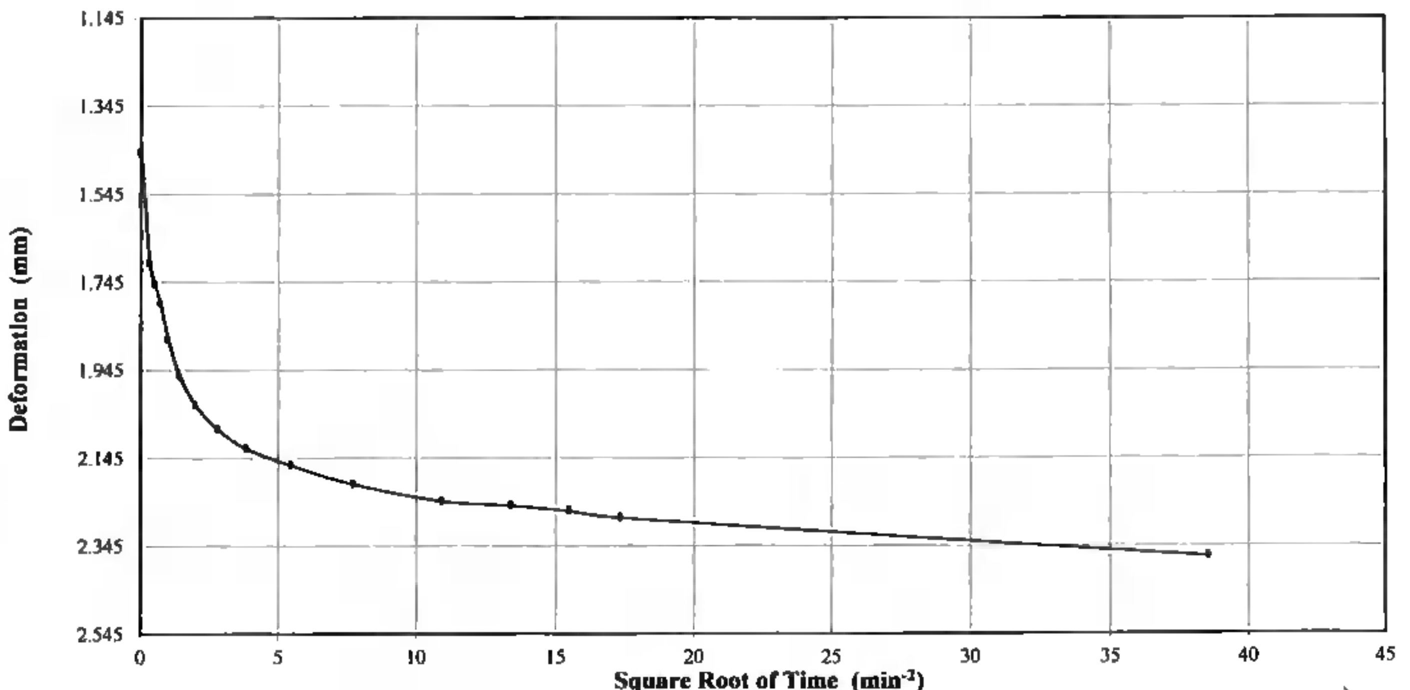
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106, ST-01 (60.0-62.0')
Lab Sample No:	13J353

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 8 - 32000 psf





1-04-14
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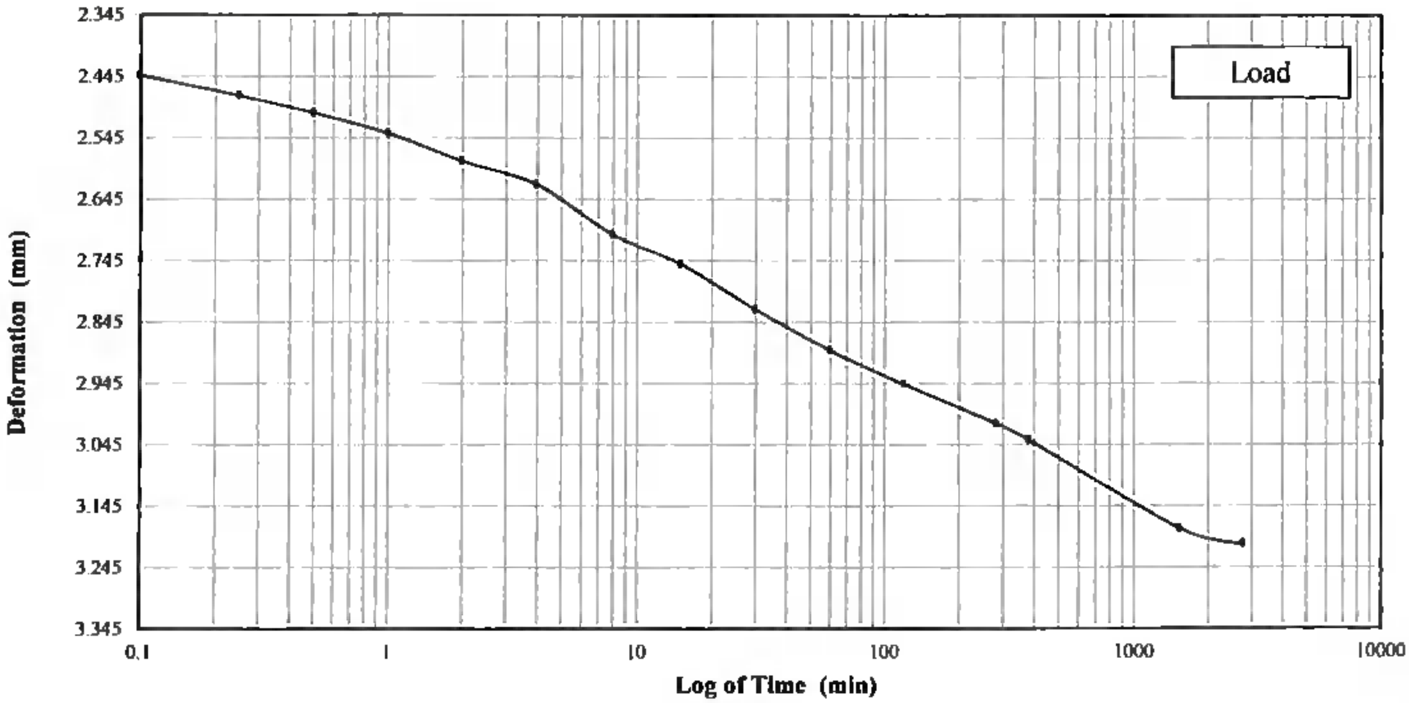
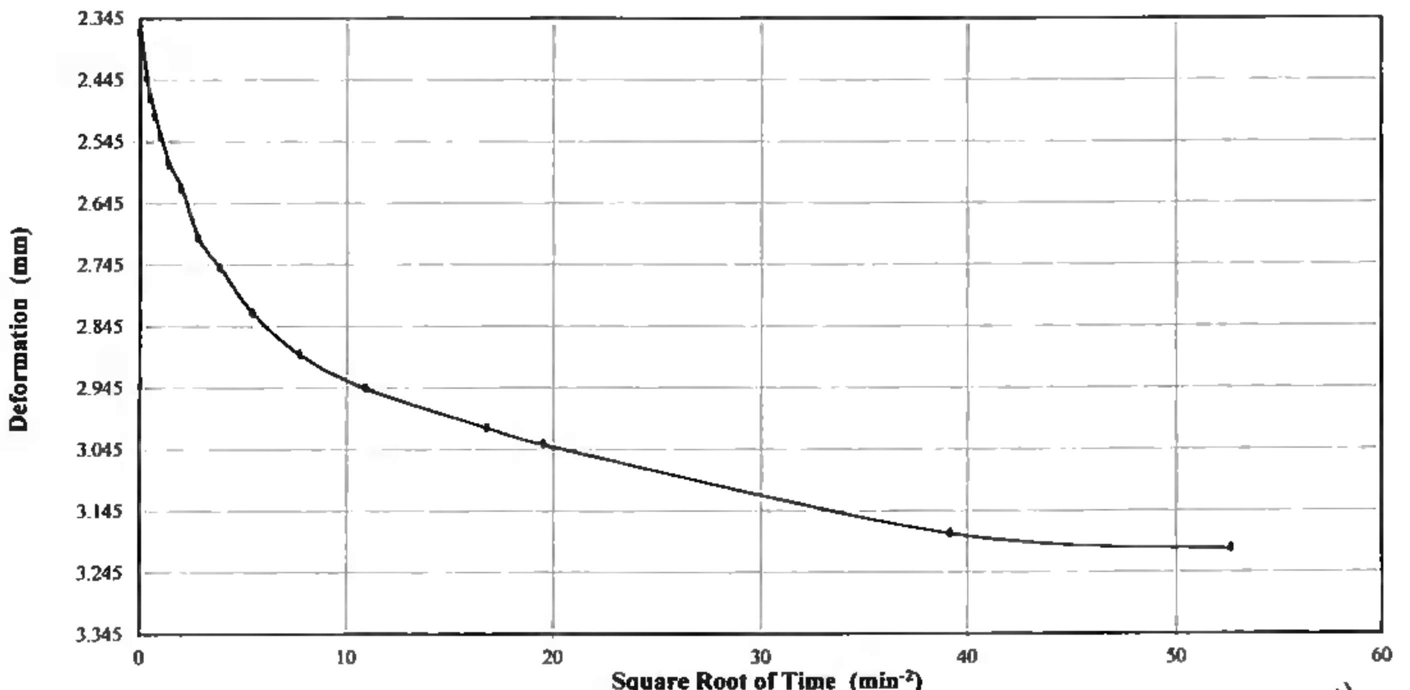
Lab Sample No: 13J353

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 9 - 48000 psf





1-04-14
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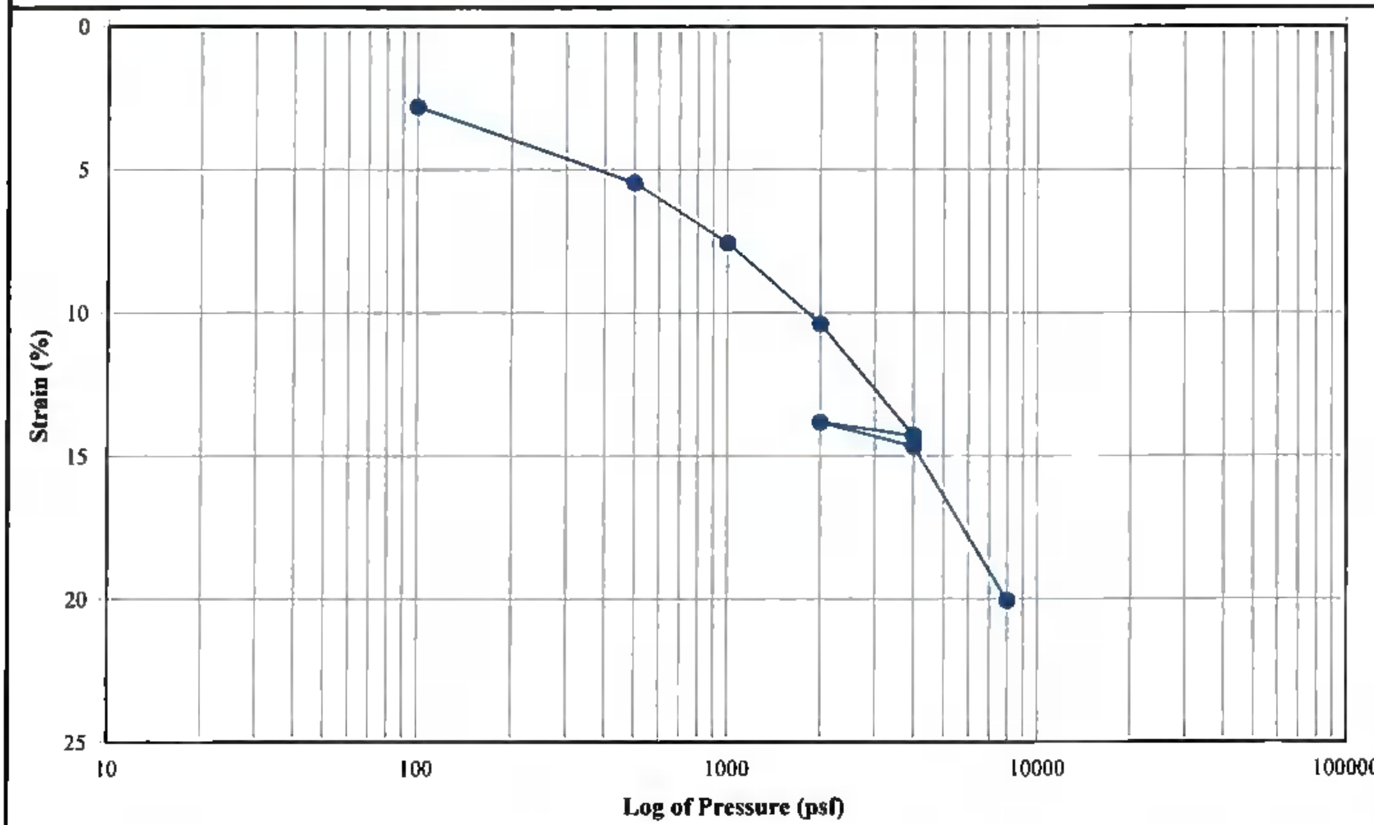
Excel Geotechnical Testing, Inc
"Excellence in Testing"

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-106A, S-03 (16.0-18.0')
Lab Sample No: 13K143

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-106A, S-03 (16.0-18.0')	13K143	6	2.54	6.35	50.9	83.5	100	862	2.81	1	
							500	1266	5.46	2	
							1000	1468	7.56	3	
							2000	1405	10.39	4	
							4000	1454	14.29	5	
							2000	175	13.82	6	
							4000	1238	14.66	7	
							8000	4330	20.06	8	

Notes:

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

2-3-14
NJR

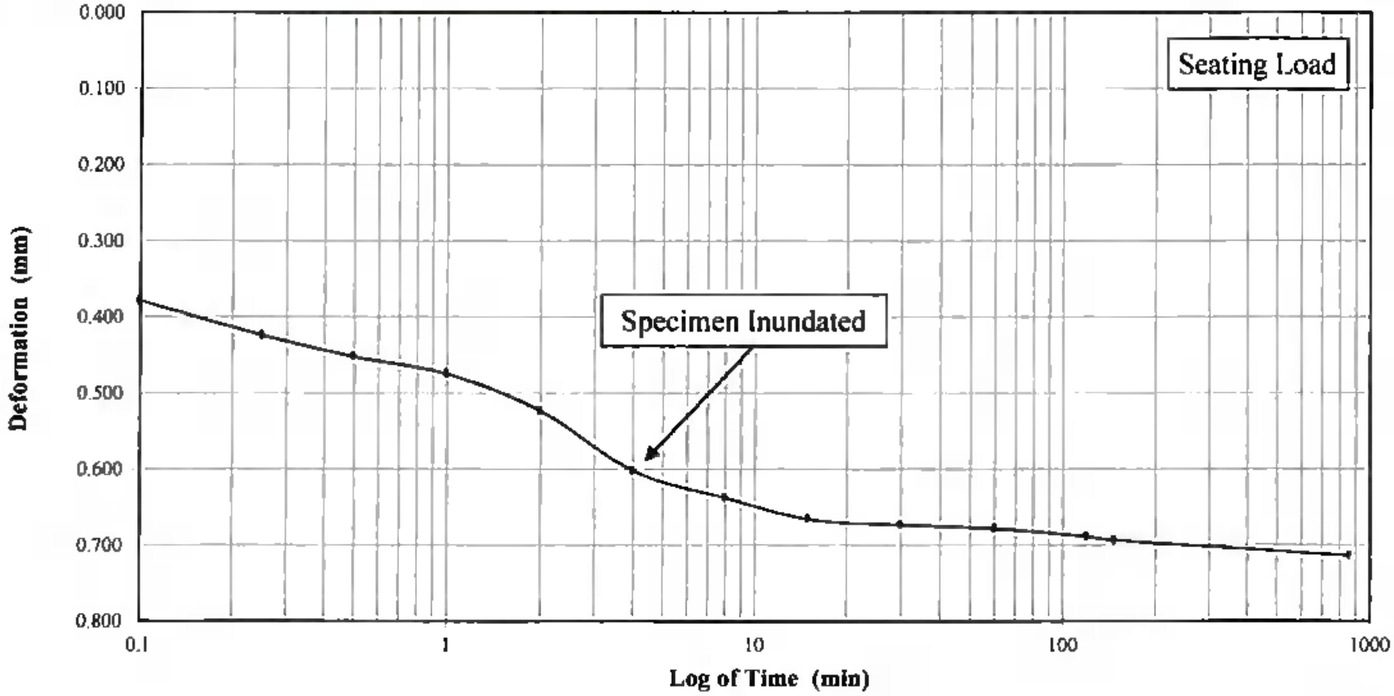
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

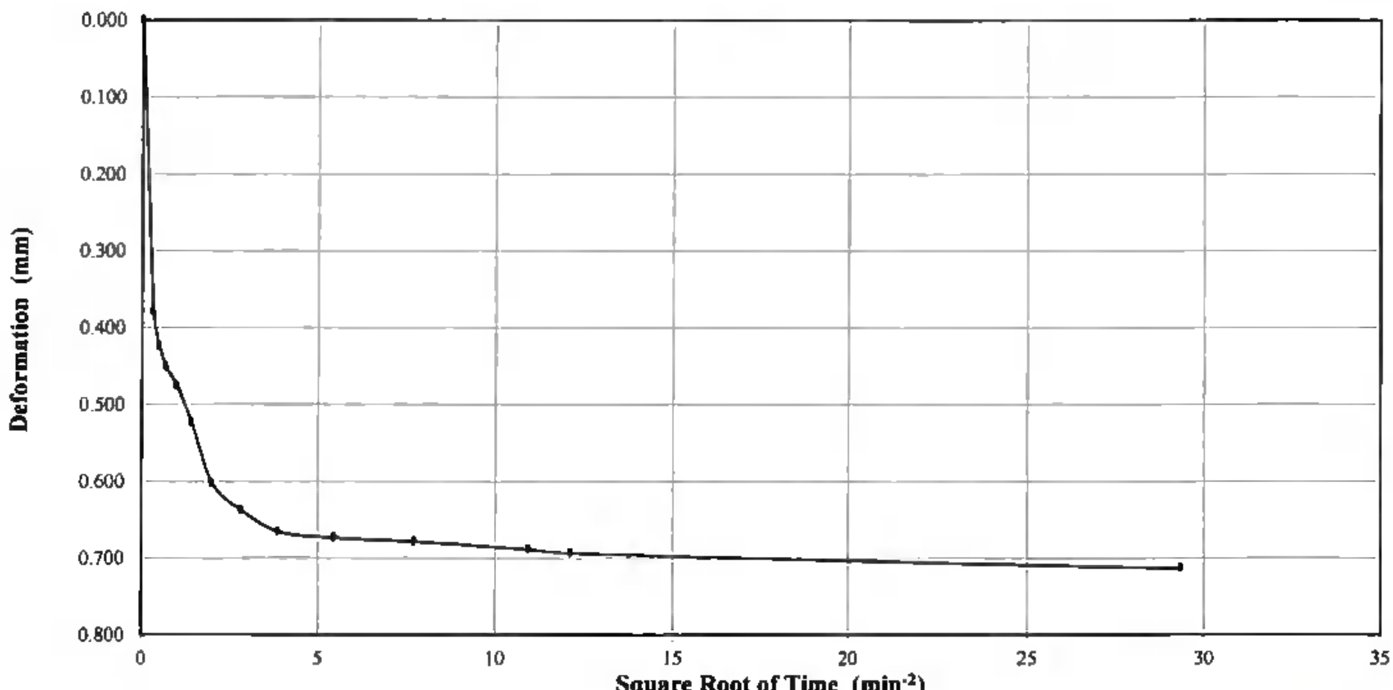
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106A, S-03 (16.0-18.0')
Lab Sample No:	13K143

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





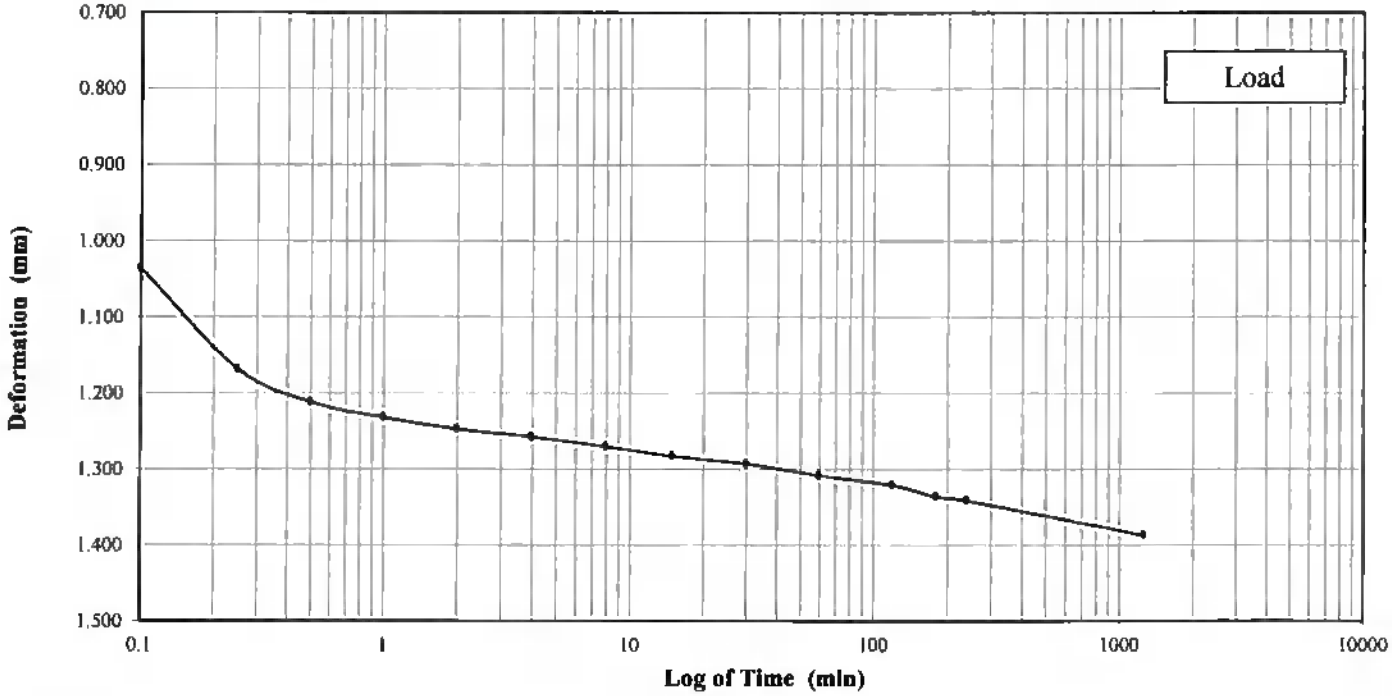
2-3-14
NSR

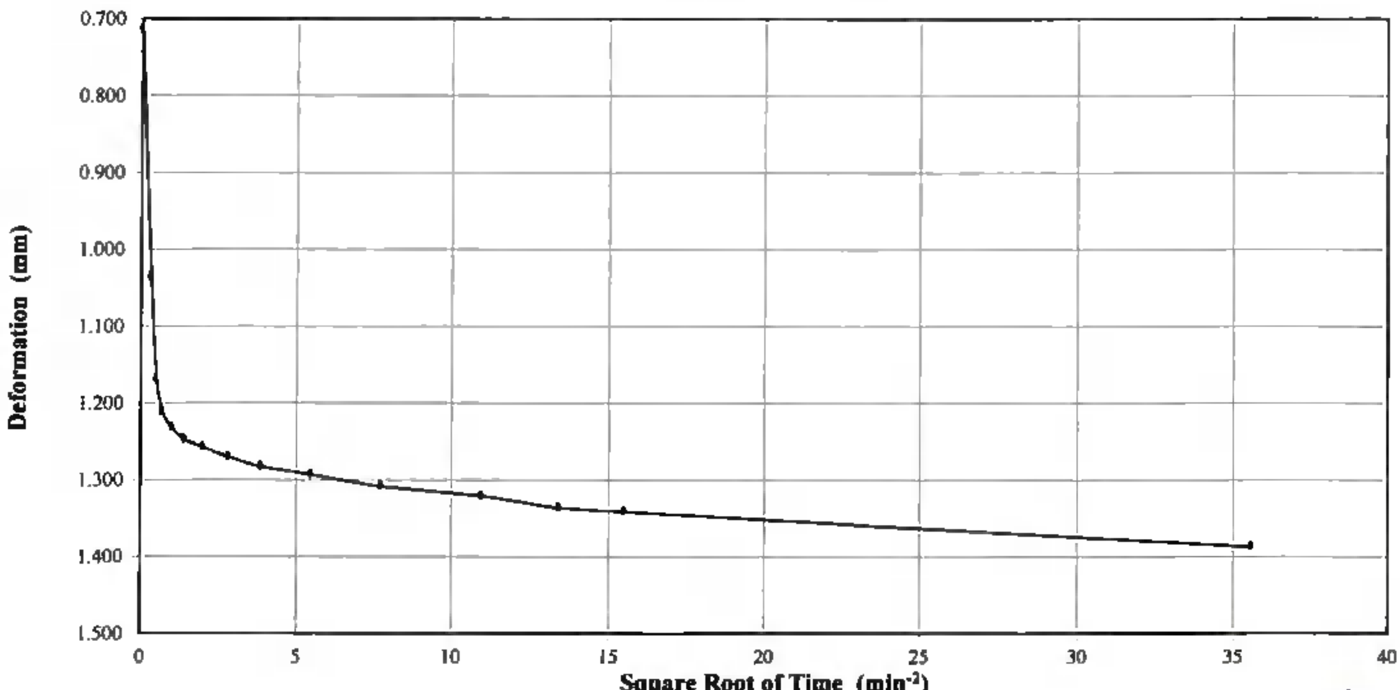
Excel Geotechnical Testing, Inc. *"Excellence in Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-106A, S-03 (16.0-18.0') Lab Sample No: 13K143

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf





2-3-14
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Excel Geotechnical Testing, Inc.
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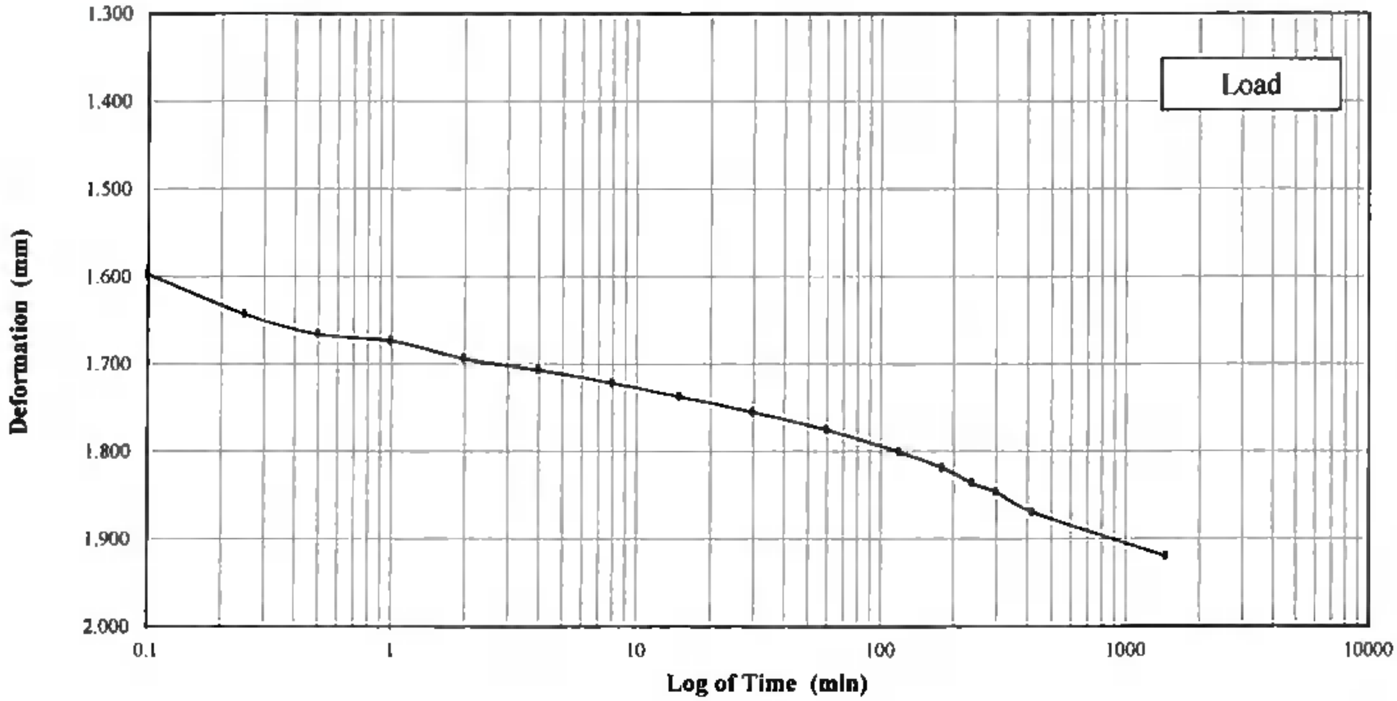
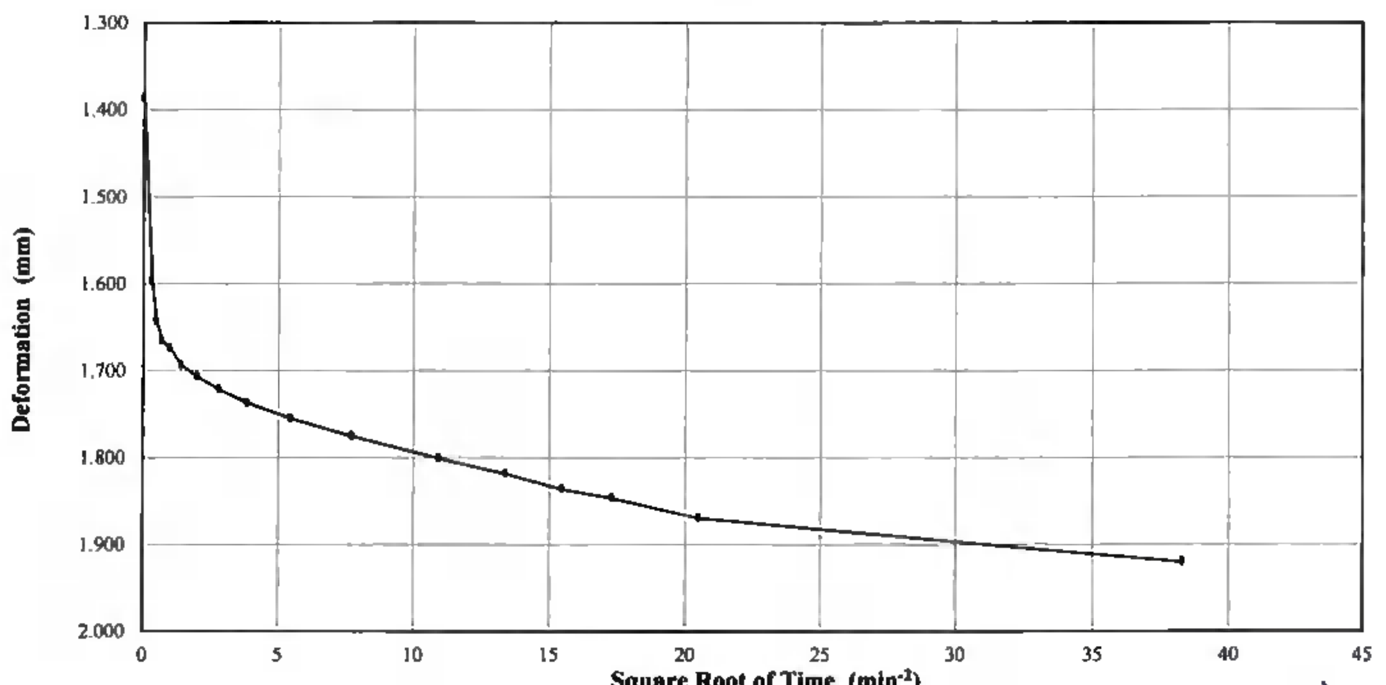
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106A, S-03 (16.0-18.0')
Lab Sample No:	13K143

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** **Figure 3 - 1000 psf**





2-3-14
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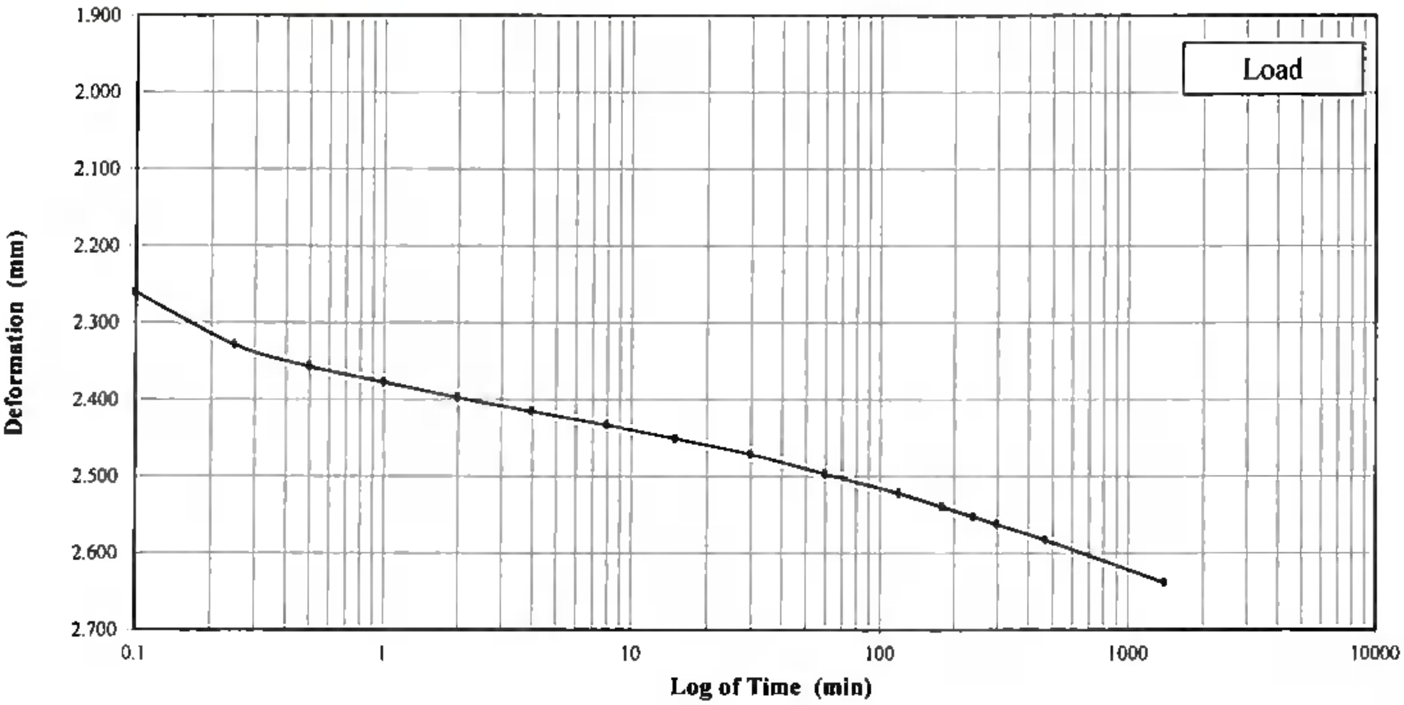
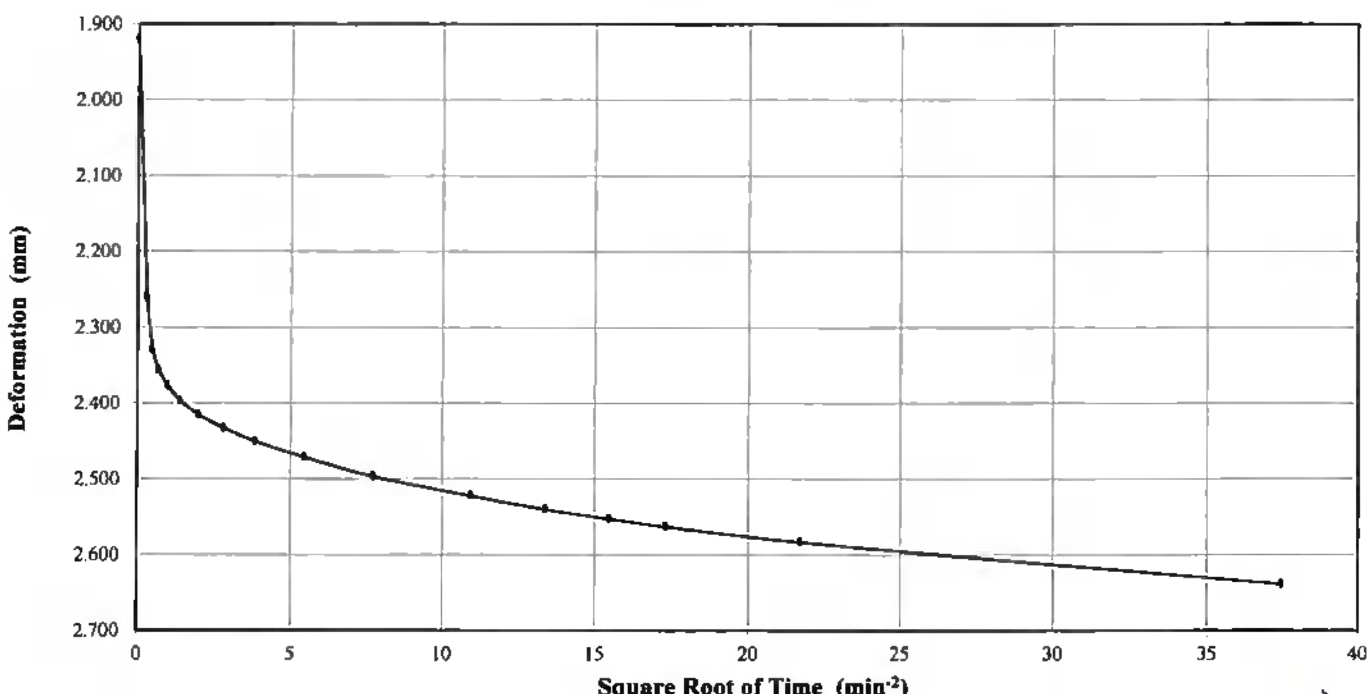
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106A, S-03 (16.0-18.0')
Lab Sample No:	13K143

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf

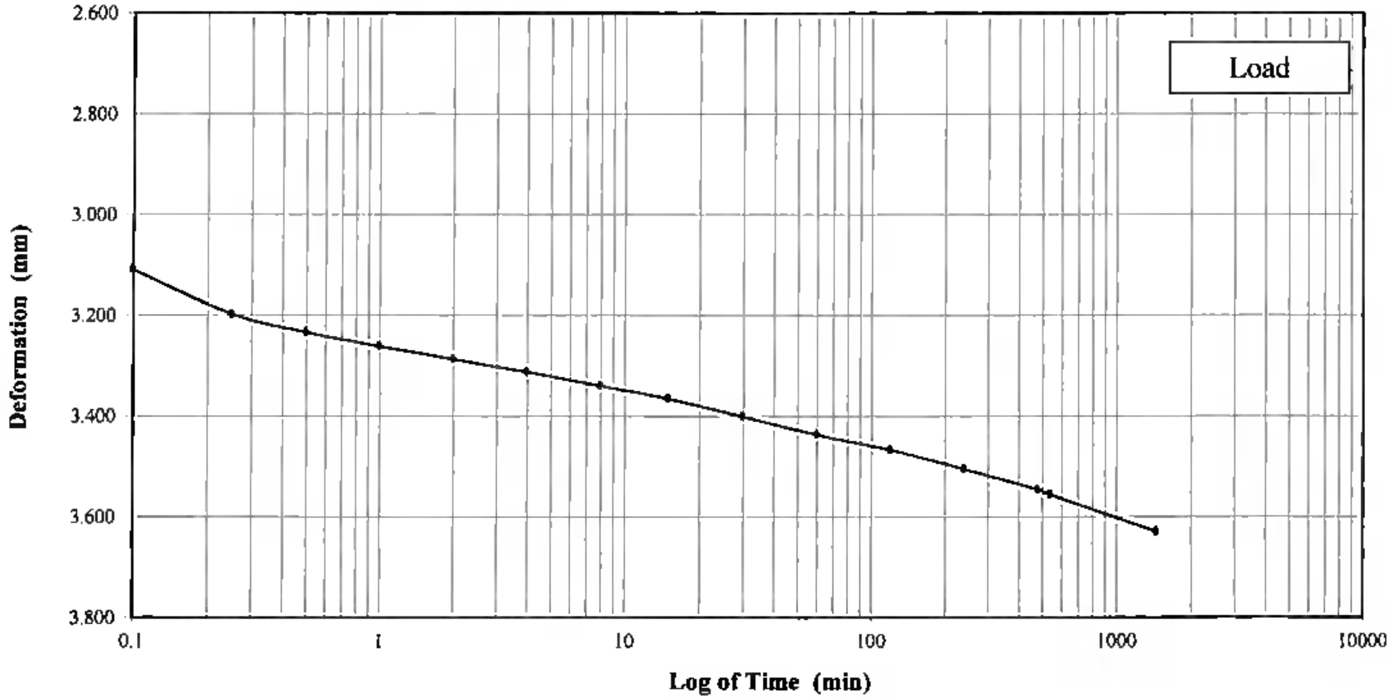


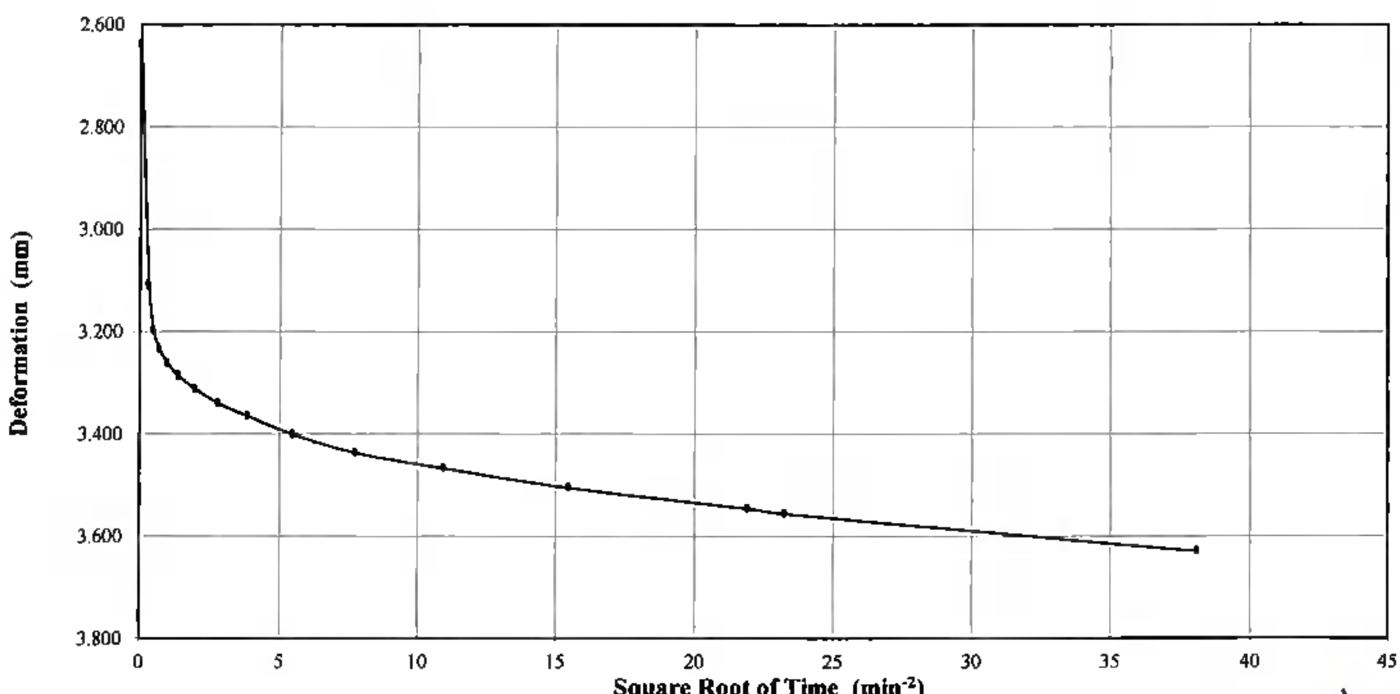


2-3-14
NSR

	Project Name:	Winyah Generating Station
Excel Geotechnical Testing, Inc. *"Excellence in Testing"*	Project No:	618
953 Forrest Street, Roswell, Georgia 30075	Client Sample ID:	SPT-106A, S-03 (16.0-18.0')
Tel: (770) 910 7537 Fax: (770) 910 7538	Lab Sample No:	13K143

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 5 - 4000 psf

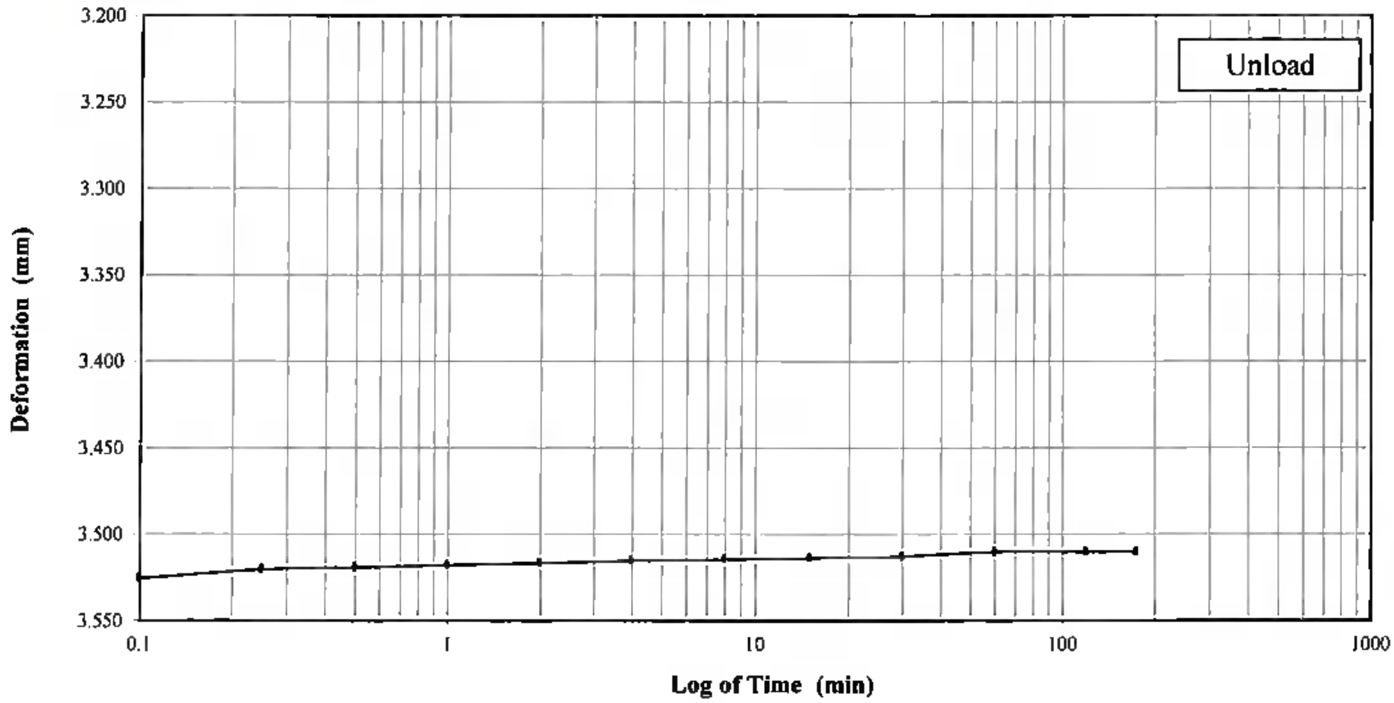


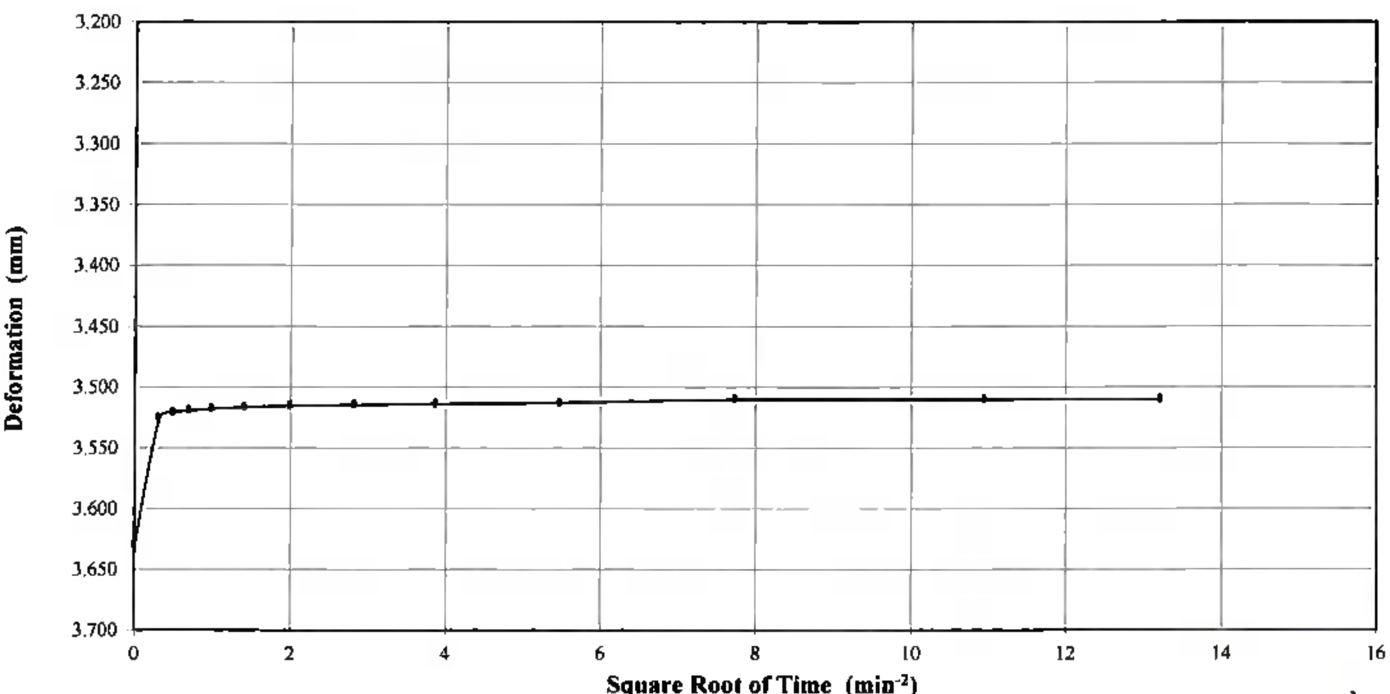


2-3-14
NSR

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-106A, S-03 (16.0-18.0') Lab Sample No: 13K143

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 6 - 2000 psf





2-5-14 NSR

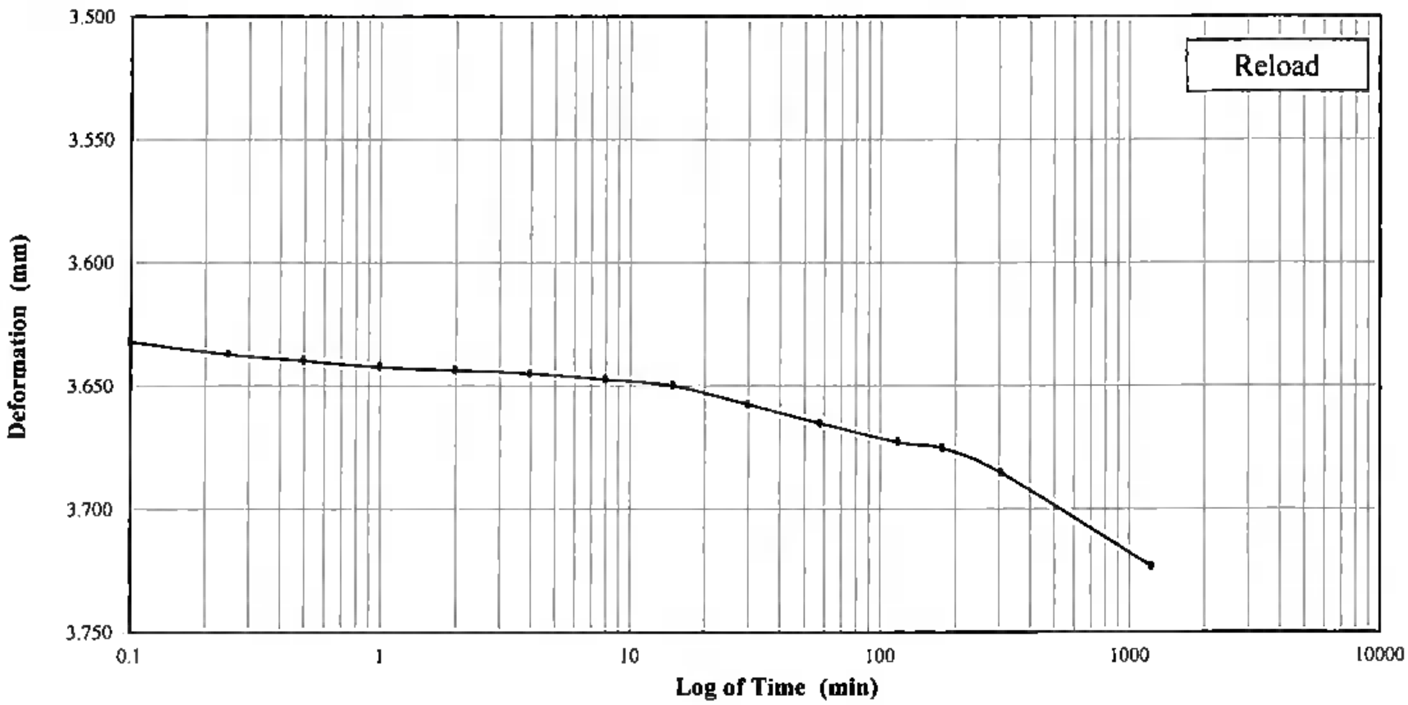
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

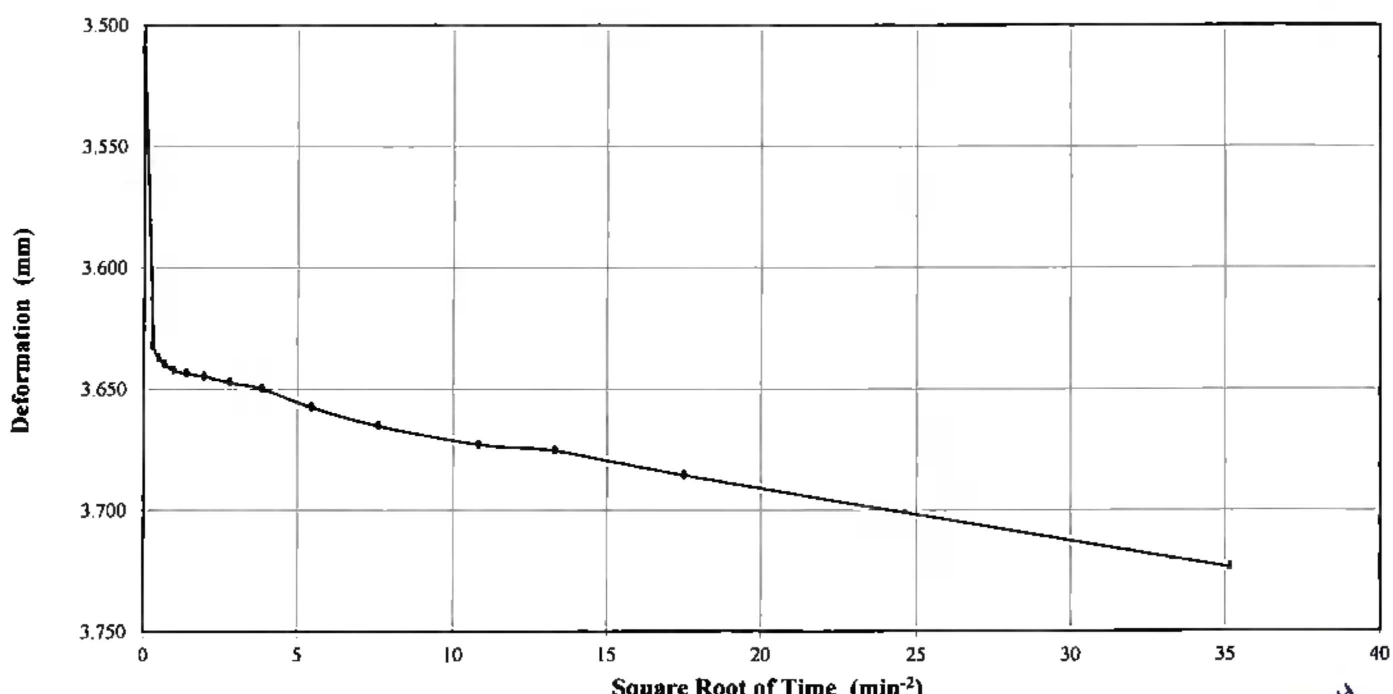
Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-106A, S-03 (16.0-18.0')
Lab Sample No: 13K143

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 7 - 4000 psf





2-3-14
NSR

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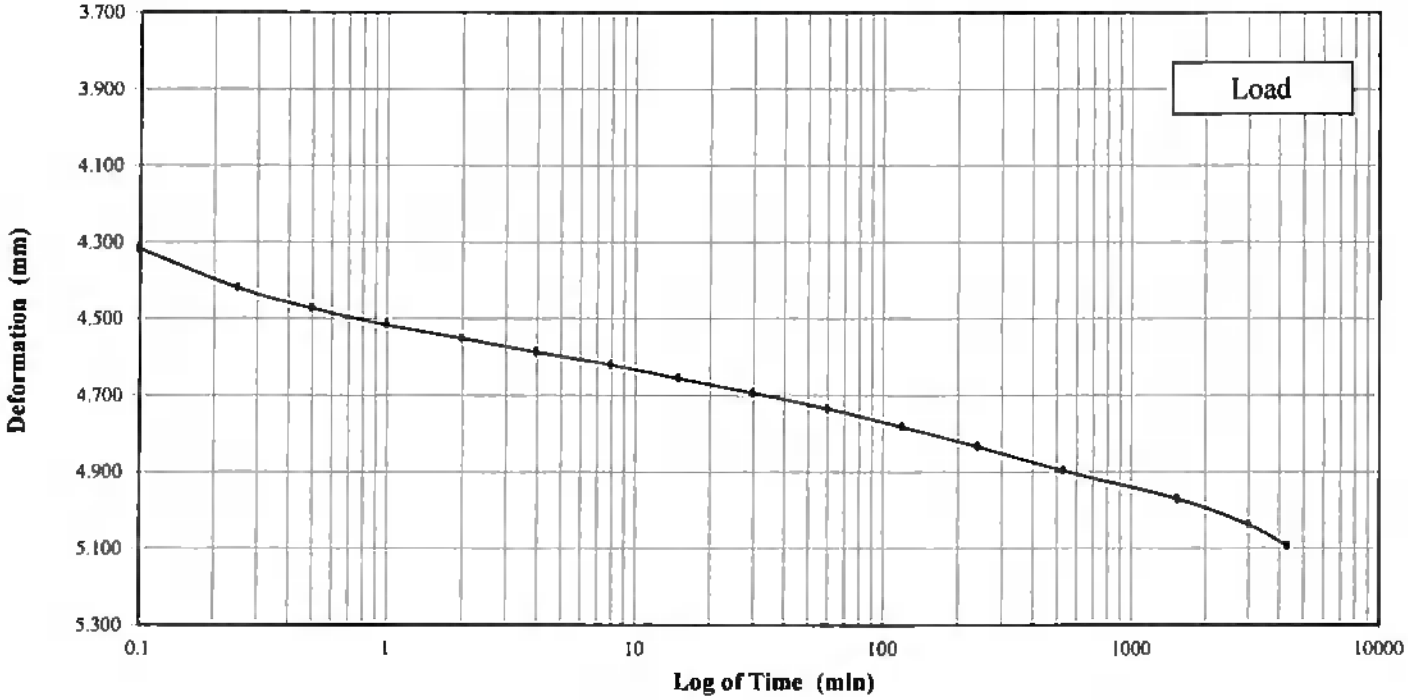
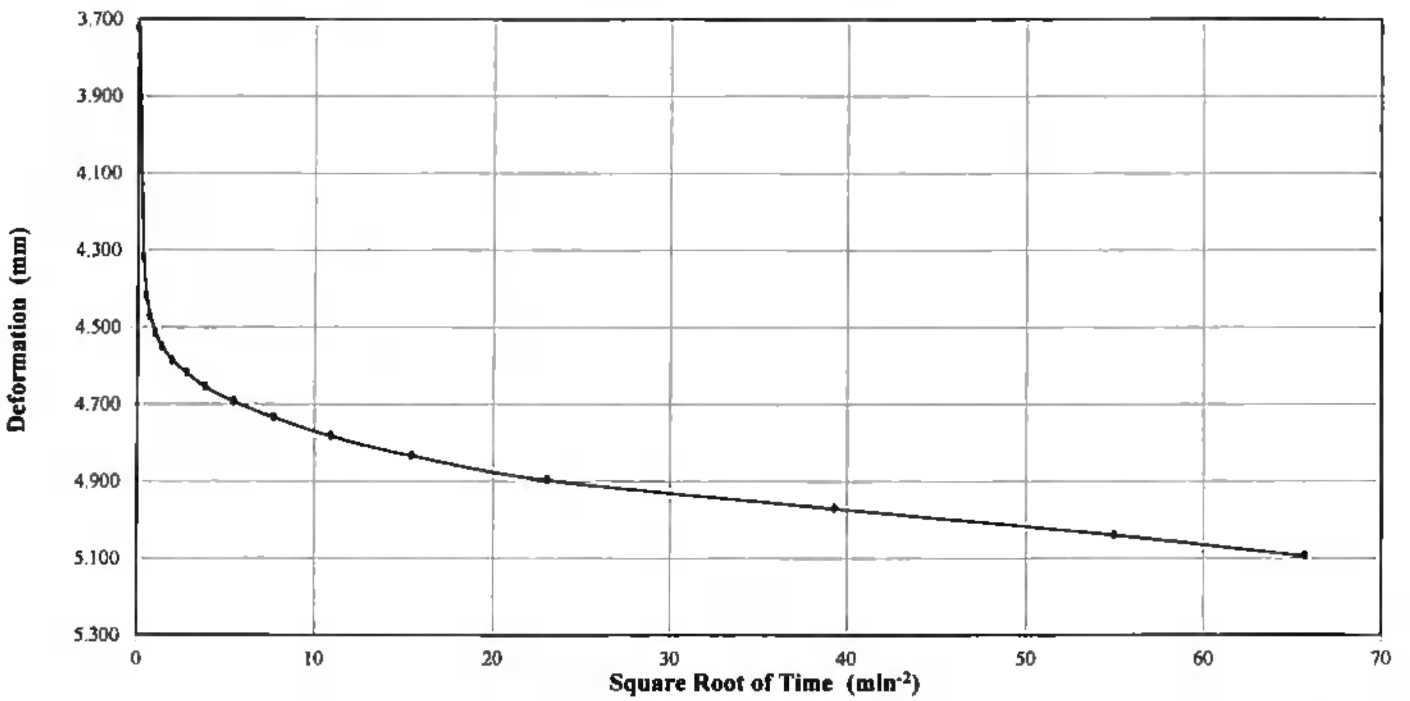
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106A, S-03 (16.0-18.0')
Lab Sample No:	13K143

ASTM D 2435 — **ONE-DIMENSIONAL CONSOLIDATION TEST** — Figure 8 - 8000 psf





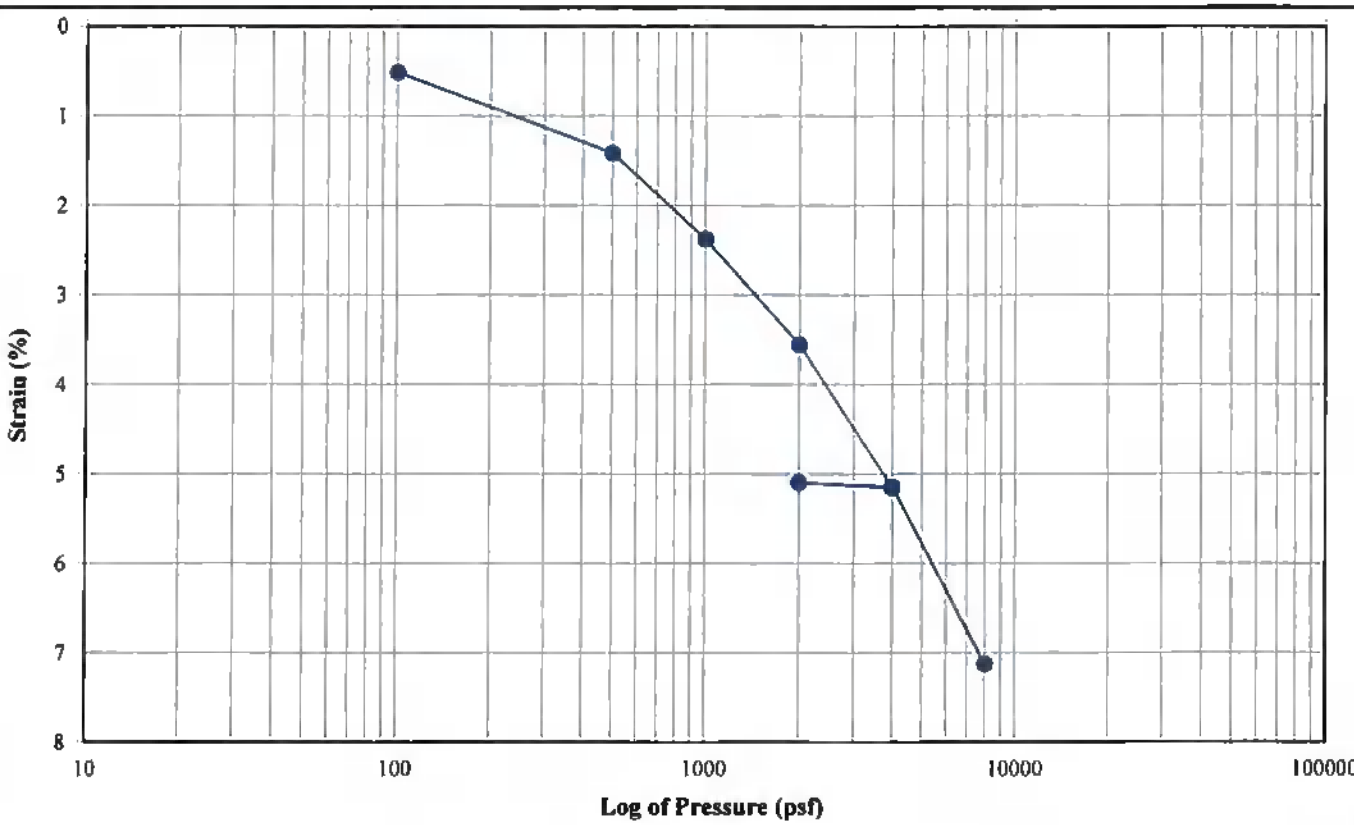
2-3-14
NSR

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-106B, S-01 (6.0-8.0')
Lab Sample No: 13K145

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST



Client Sample ID	Lab Sample No.	Specimen Quality 1-10 (Bad to Good)	Test Specimen Initial Conditions: Height (cm)	Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)	Consolidation Pressure (psf)	Pressure Increment Duration (min)	Accumu.[1] Vertical Strain (%)	Figure No.	Remarks
SPT-106B, S-01 (6.0-8.0')	13K145	7	2.54	6.35	72.6	48.2	100	1252	0.52	1	
							500	1444	1.42	2	
							1000	1266	2.37	3	
							2000	1463	3.55	4	
							4000	1420	5.16	5	
							2000	1458	5.09	6	
							4000	180	5.14	7	
							8000	1231	7.12	8	

Notes:

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

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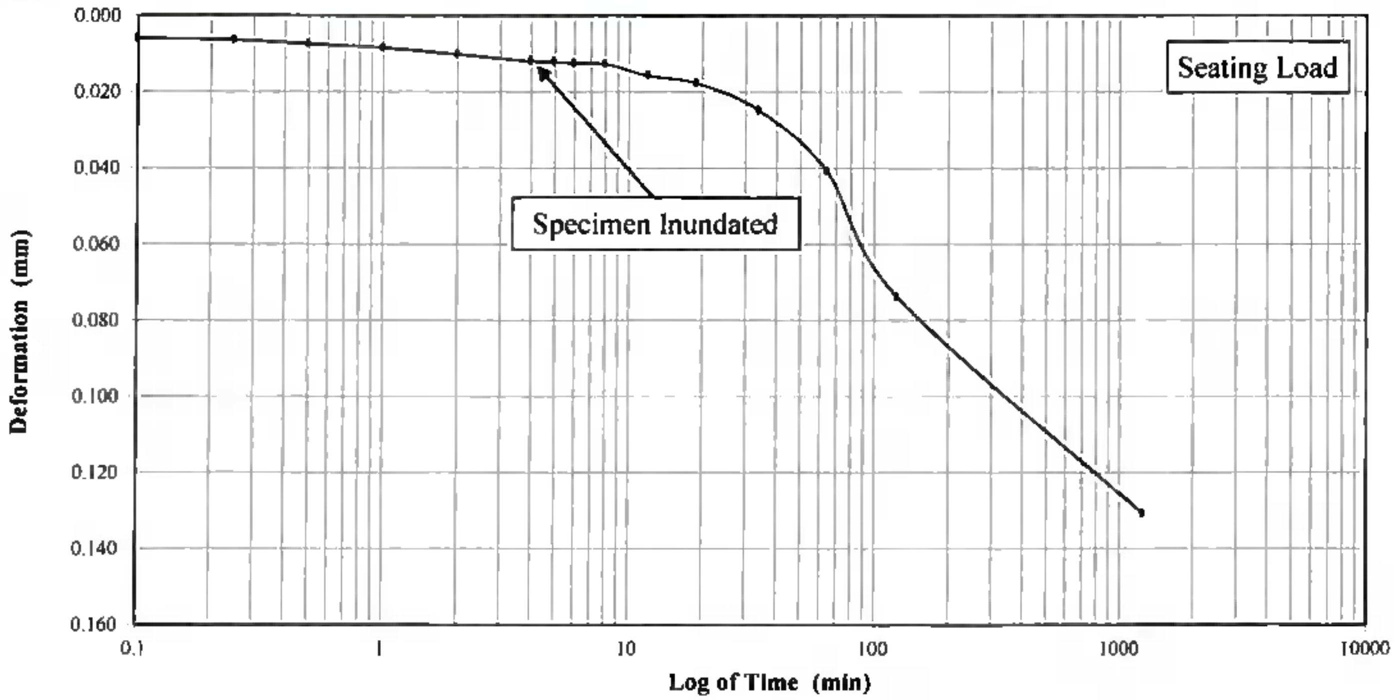
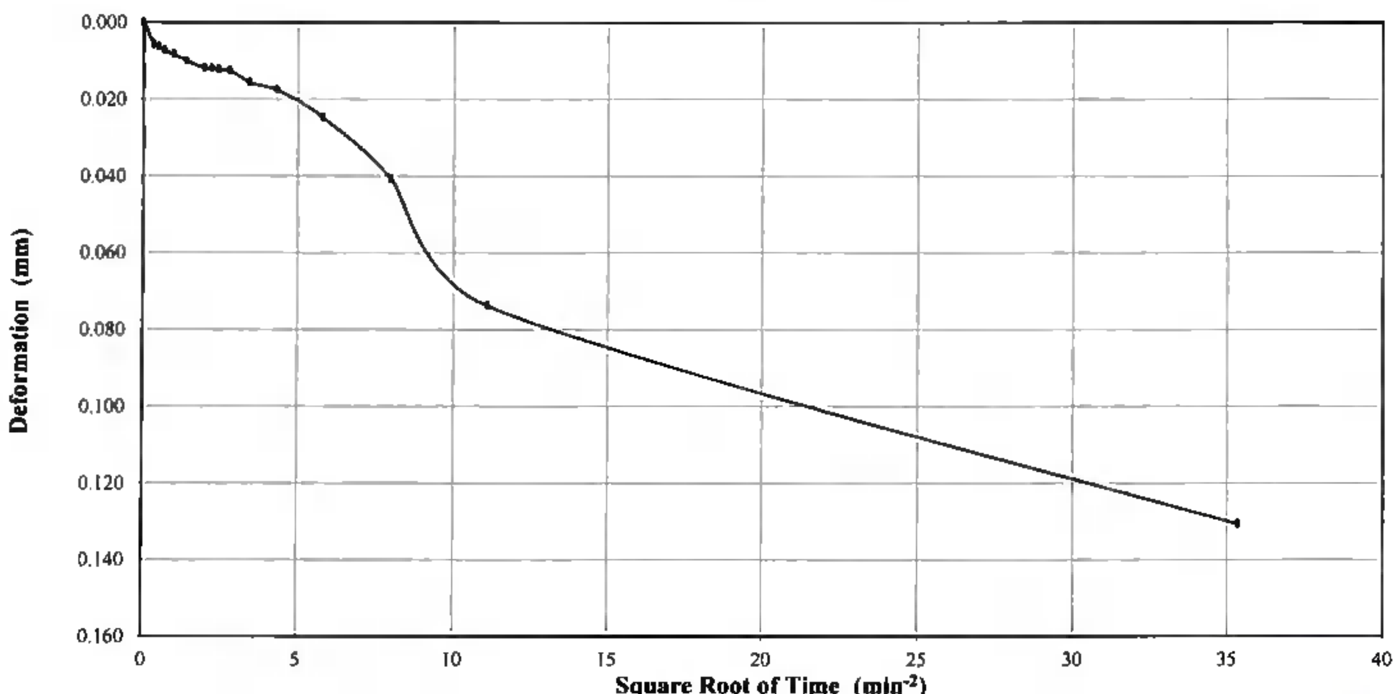
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106B, S-01 (6.0-8.0')
Lab Sample No:	13K145

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





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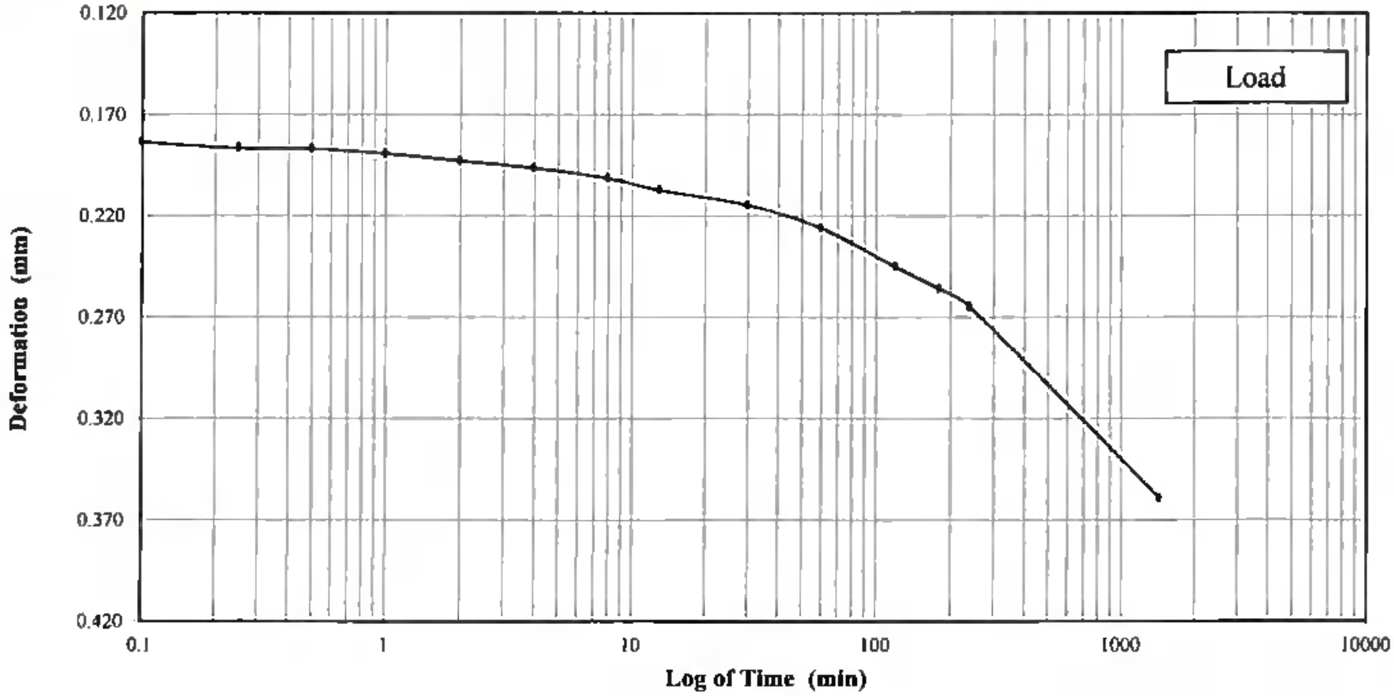
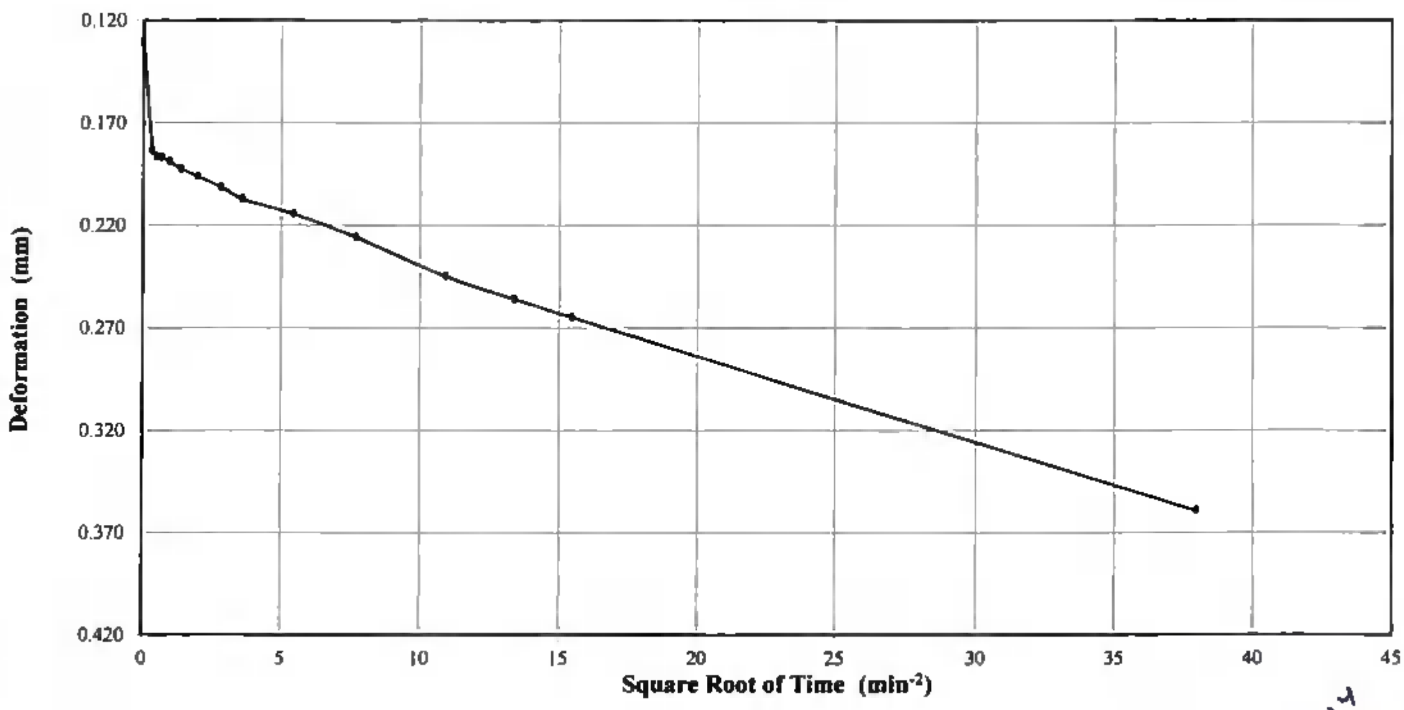
Excel Geotechnical Testing, Inc. *"Excellence In Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-106B, S-01 (6.0-8.0') Lab Sample No: 13K145

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** **Figure 2 - 500 psf**





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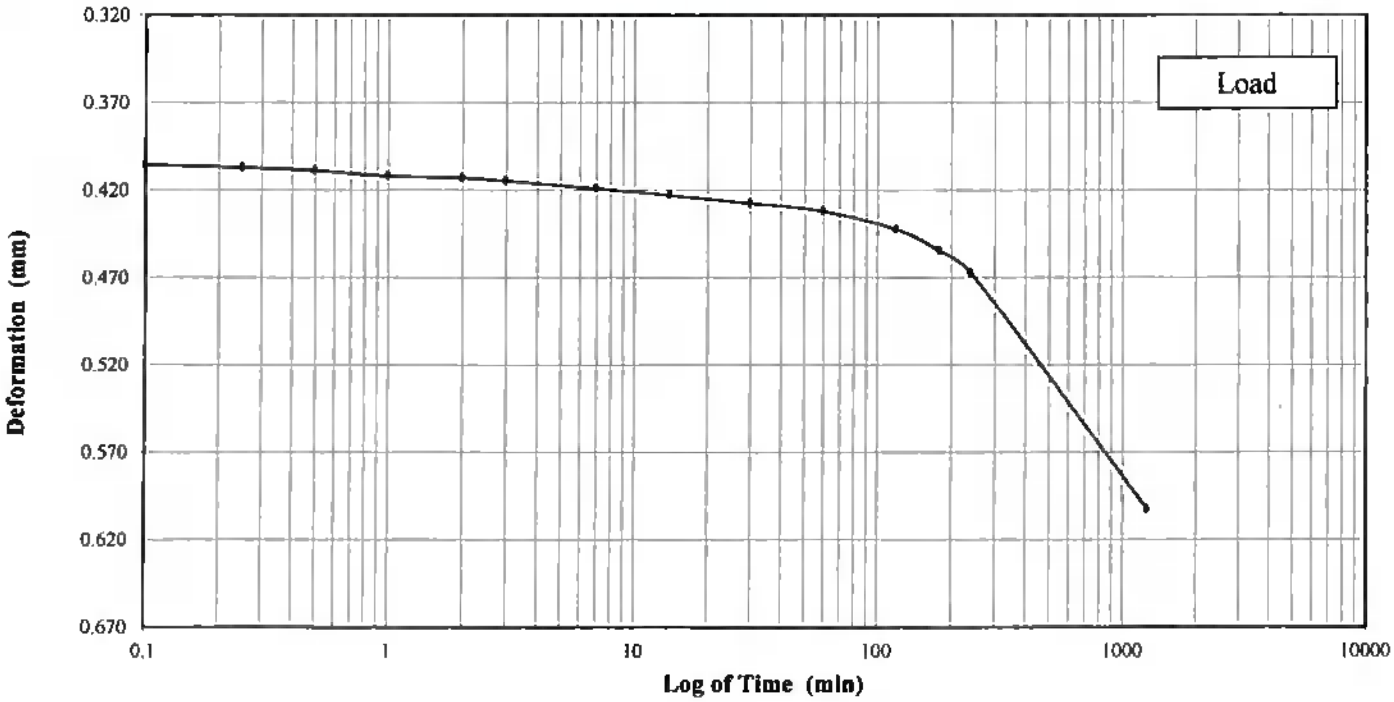
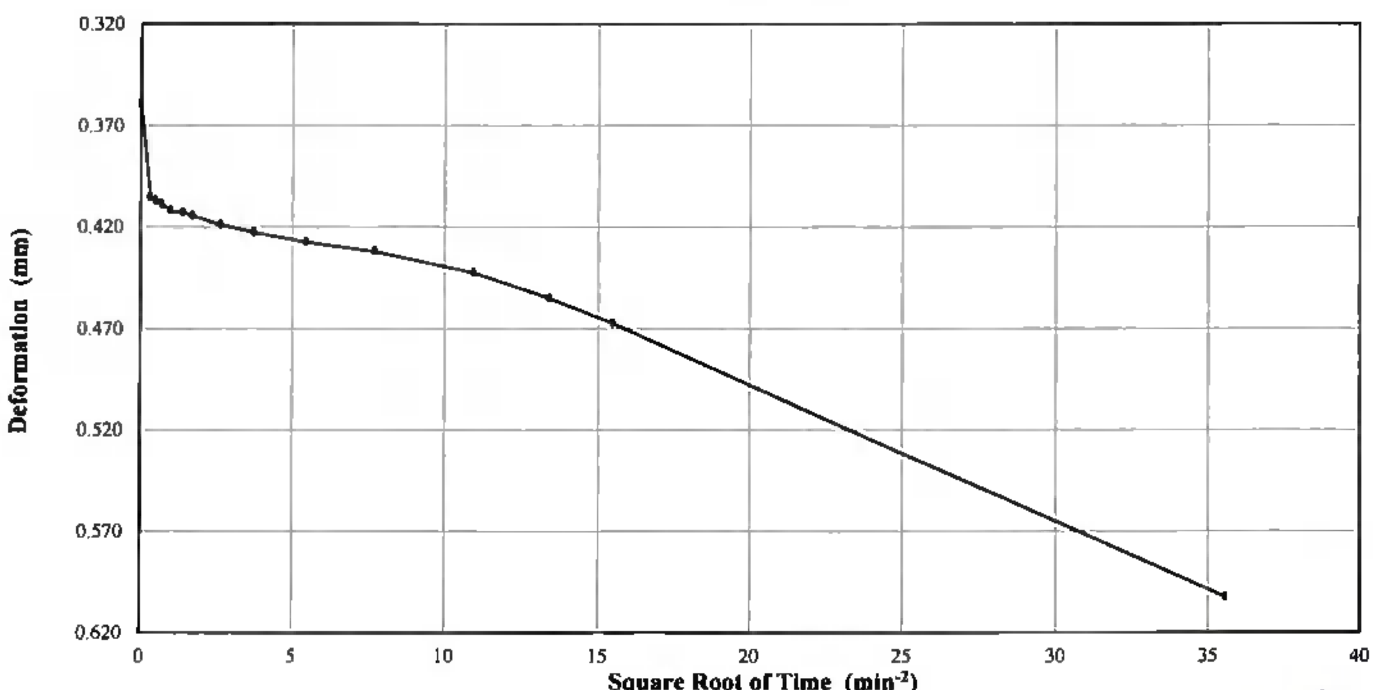
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-106B, S-01 (6.0-8.0')
Lab Sample No: 13K145

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 3 - 1000 psf





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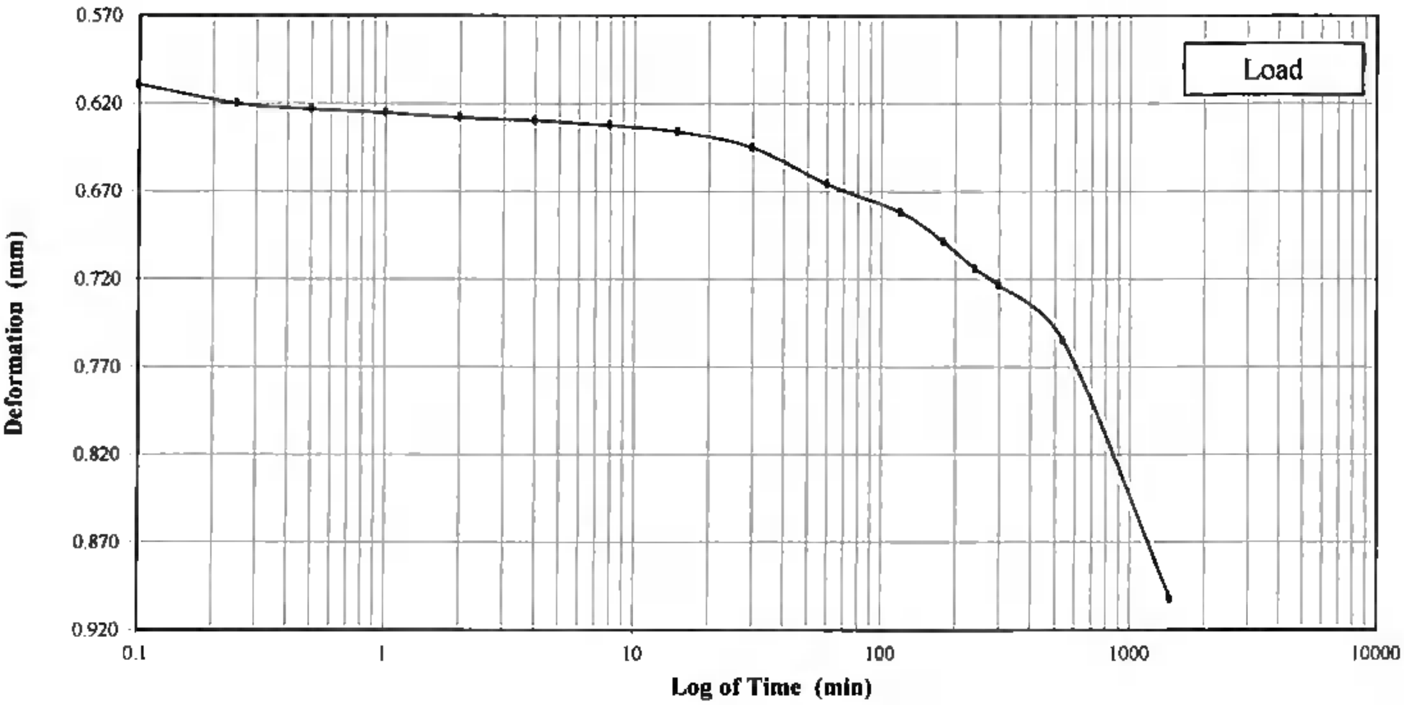
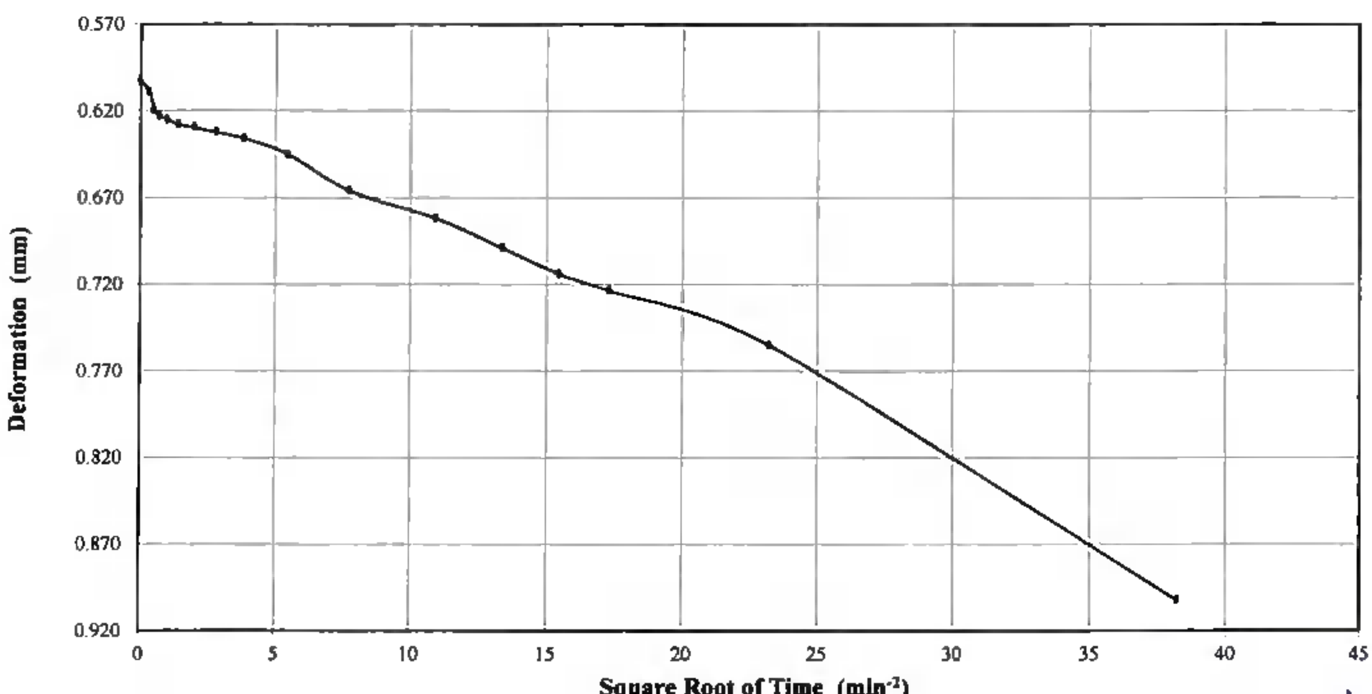
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106B, S-01 (6.0-8.0')
Lab Sample No:	13K145

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 4 - 2000 psf

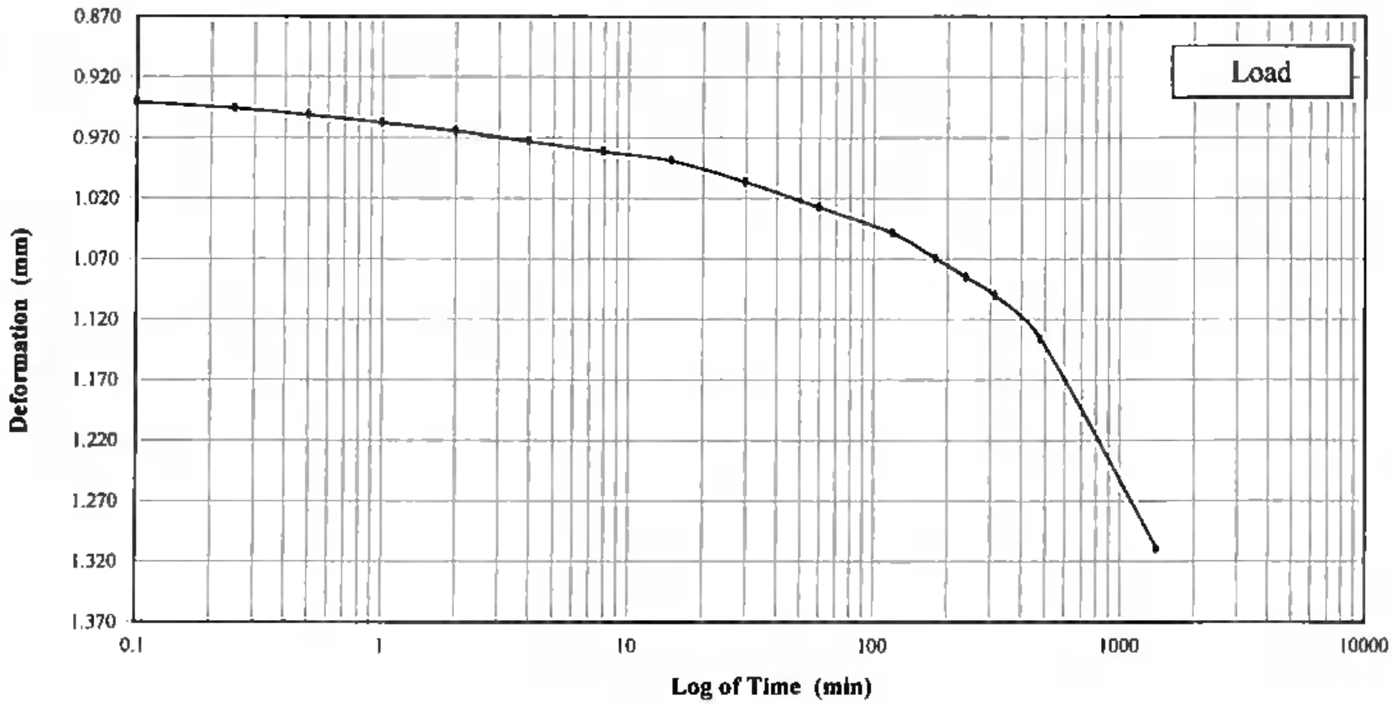


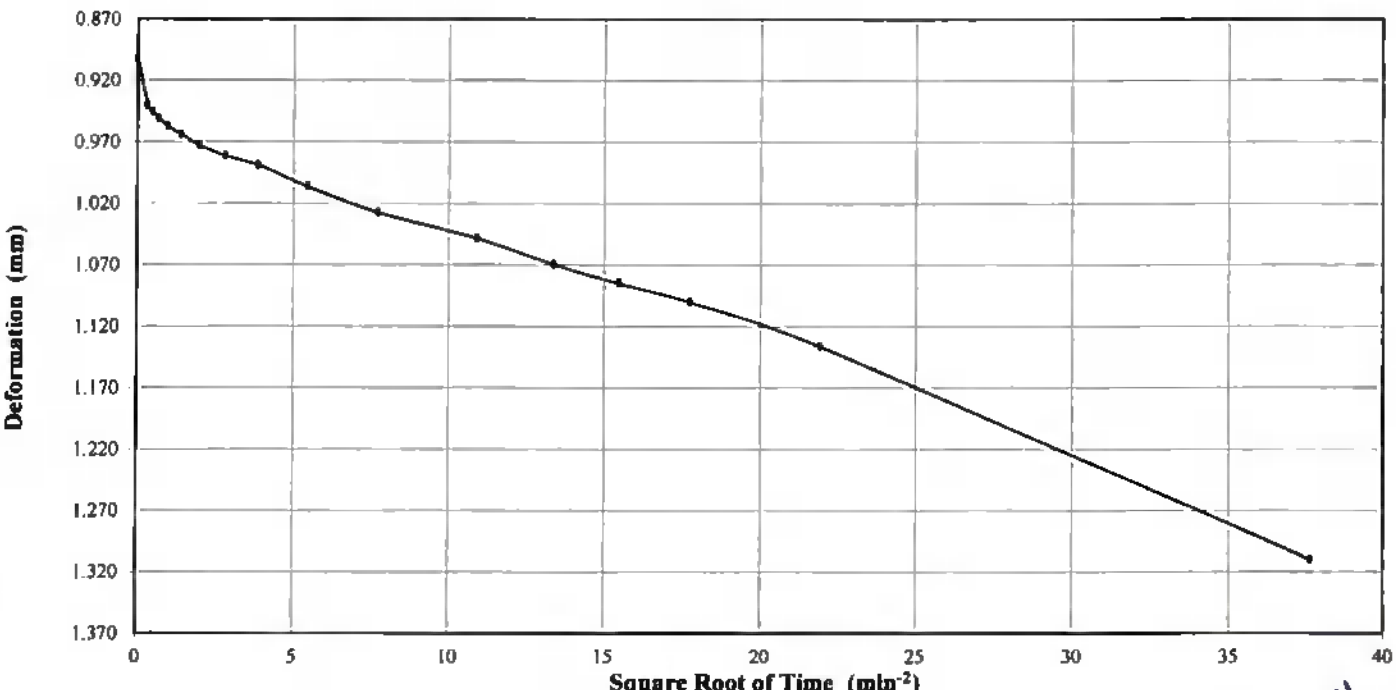


2-3-14
NSK

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-106B, S-01 (6.0-8.0')
	Lab Sample No:	13K145

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 5 - 4000 psf





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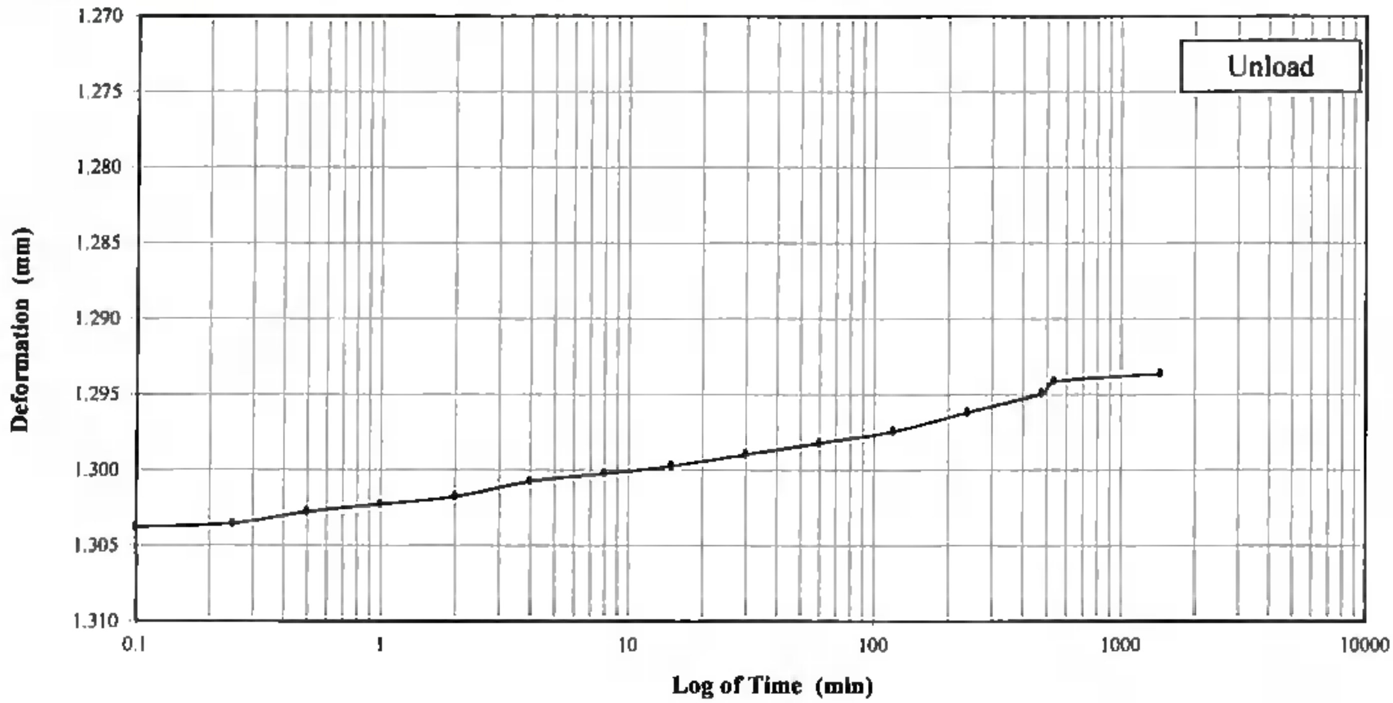
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

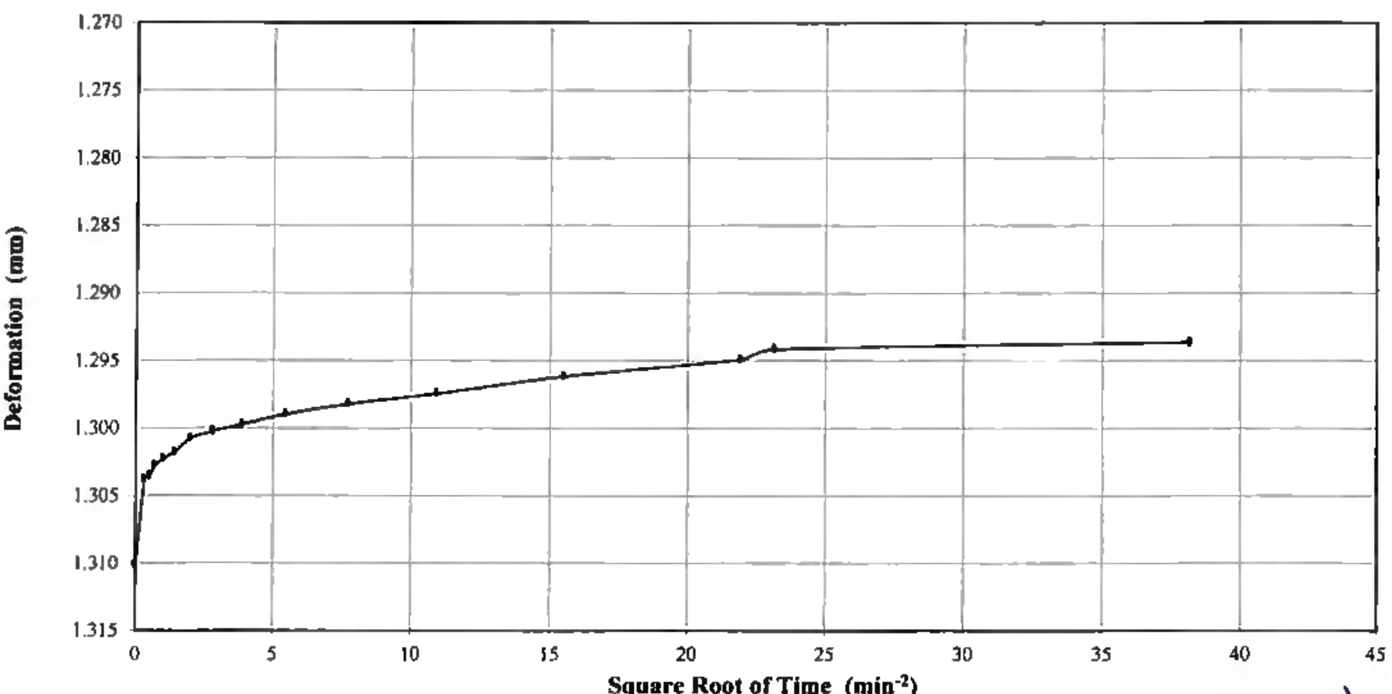
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106B, S-01 (6.0-8.0')
Lab Sample No:	13K145

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 6 - 2000 psf





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953 Forrest Street, Roswell, Georgia 30075
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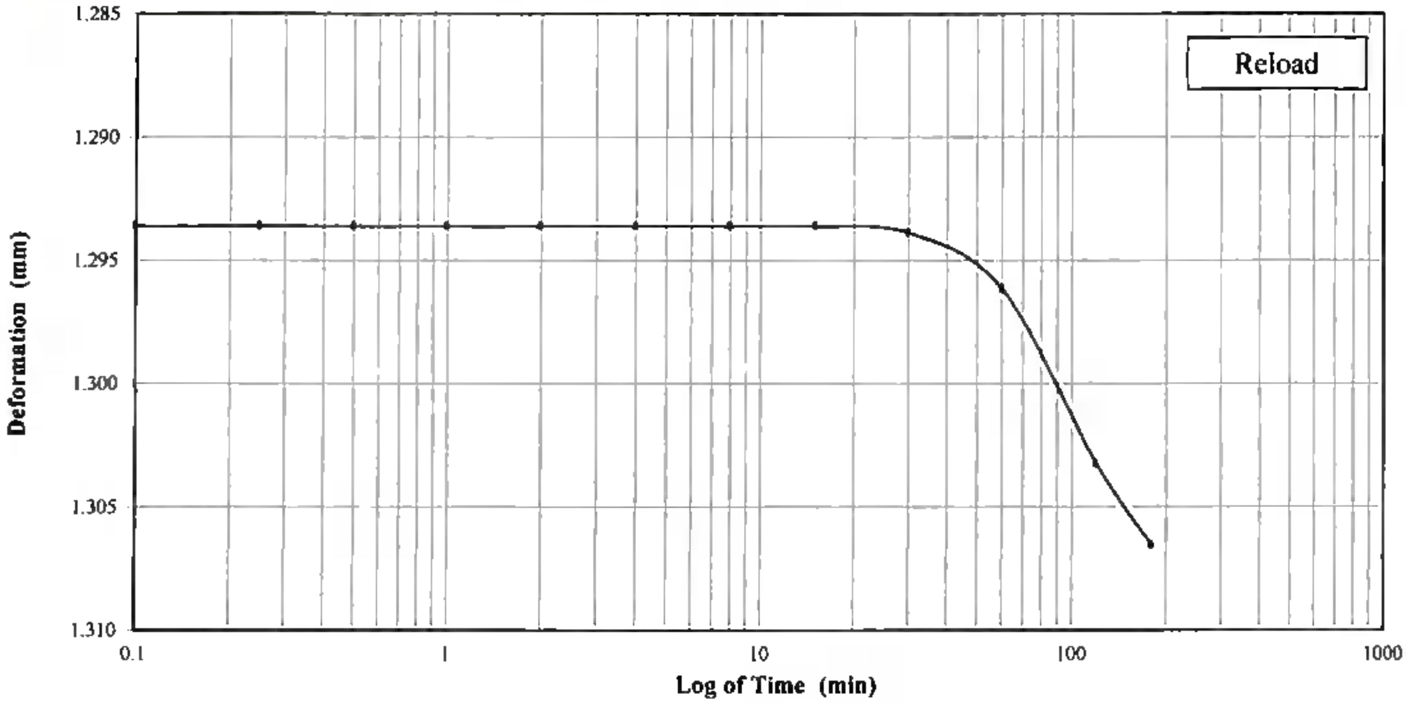
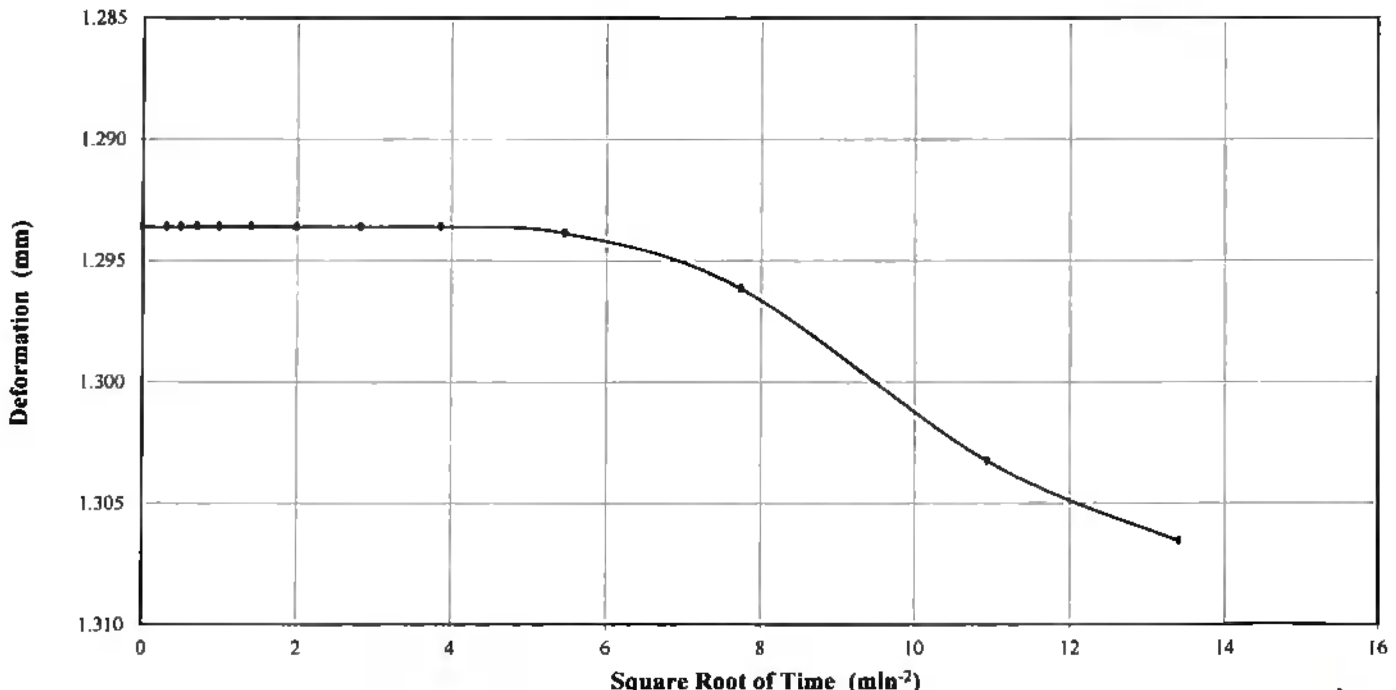
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106B, S-01 (6.0-8.0')
Lab Sample No:	13K145

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 7 - 4000 psf





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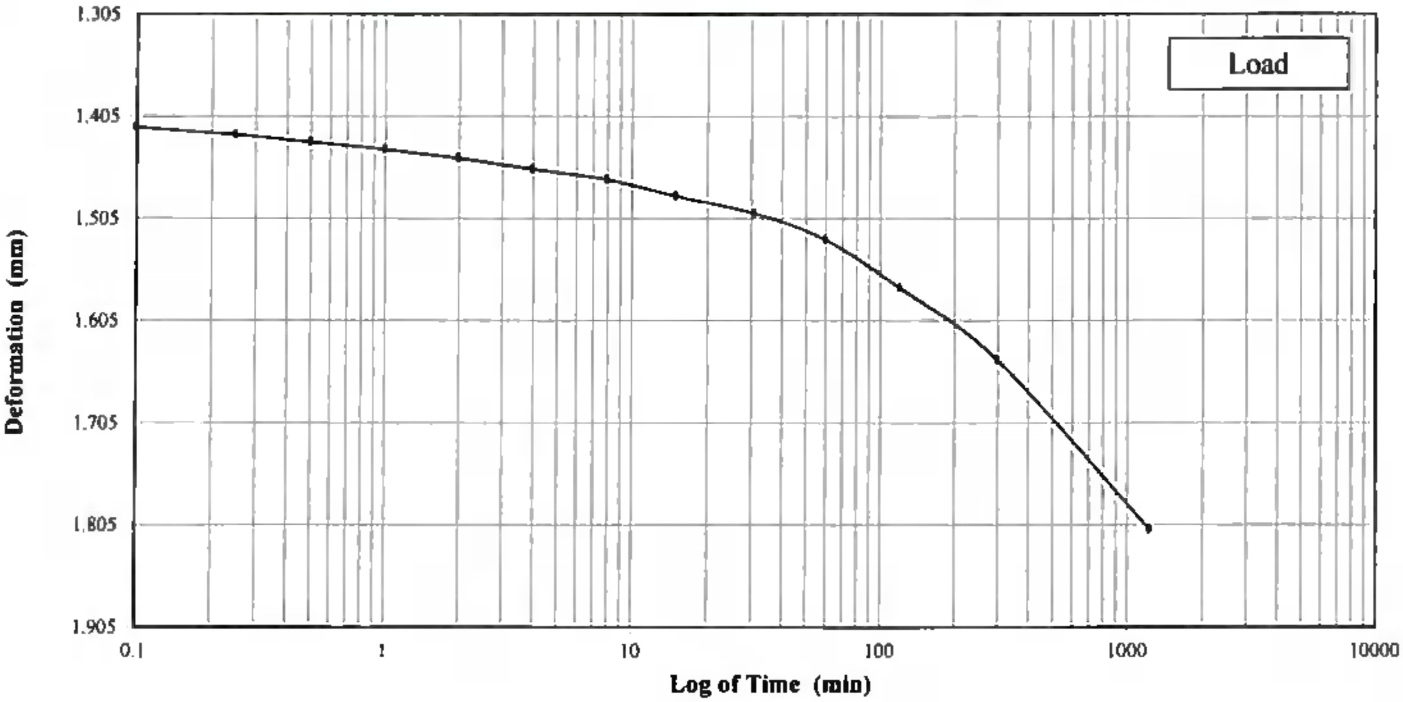
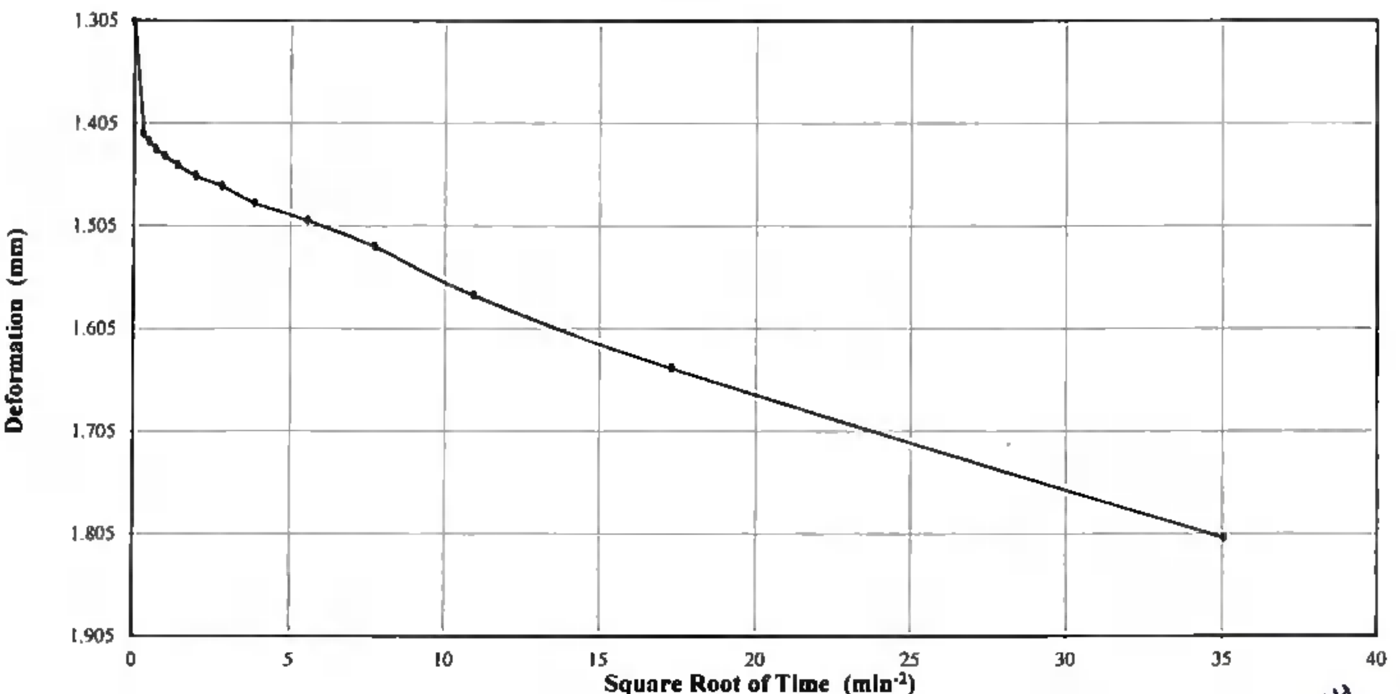
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-106B, S-01 (6.0-8.0')
Lab Sample No:	13K145

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 8 - 8000 psf





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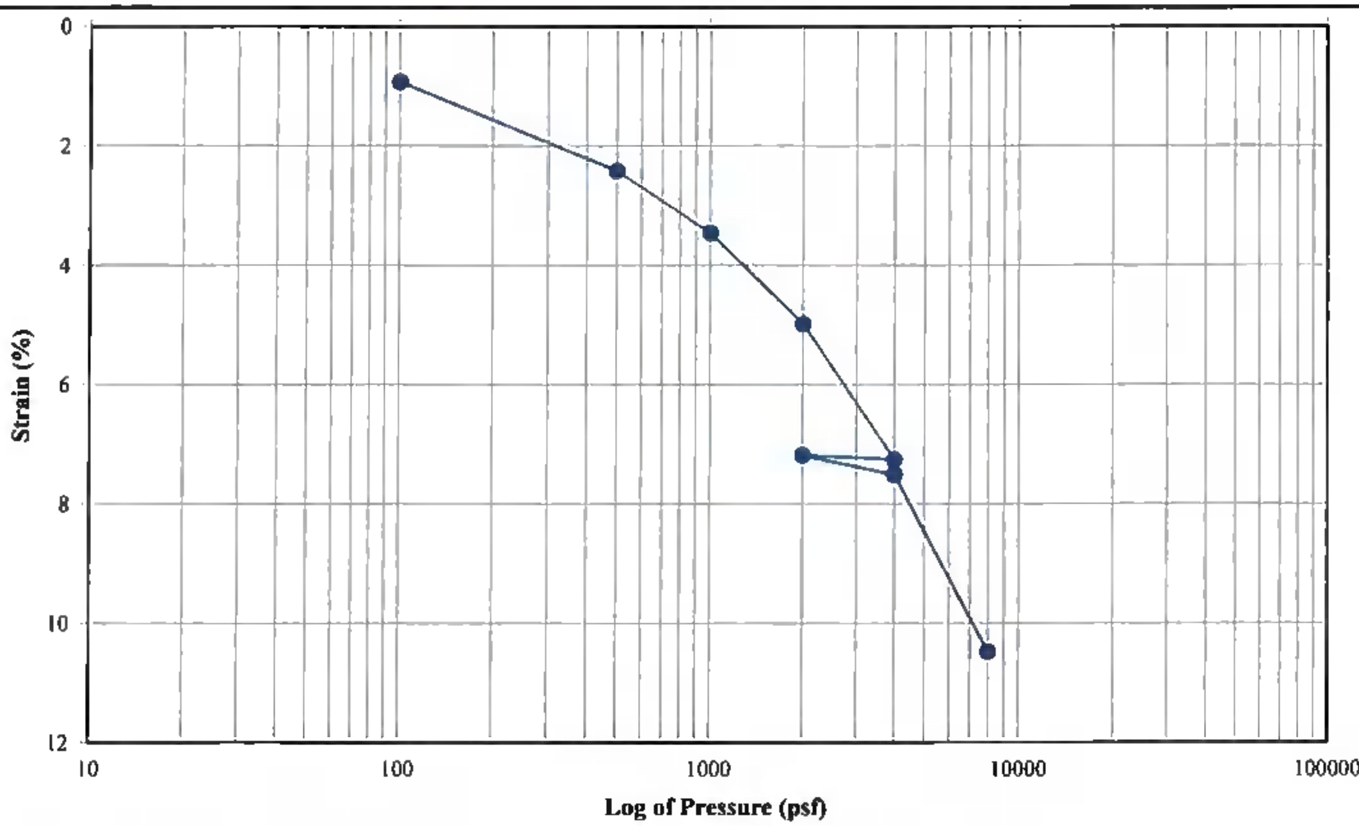
Excel Geotechnical Testing, Inc
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-107A, S-01 (6.0-8.0')
Lab Sample No: 13K149

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-107A, S-01 (6.0-8.0')	13K149	5	2.54	6.35	67.8	52.4	100	1190	0.93	1	
							500	1470	2.42	2	
							1000	1283	3.46	3	
							2000	1462	4.98	4	
							4000	1421	7.26	5	
							2000	1443	7.18	6	
							4000	1425	7.51	7	
							8000	4464	10.48	8	

Notes:

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

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"Excellence In Testing"

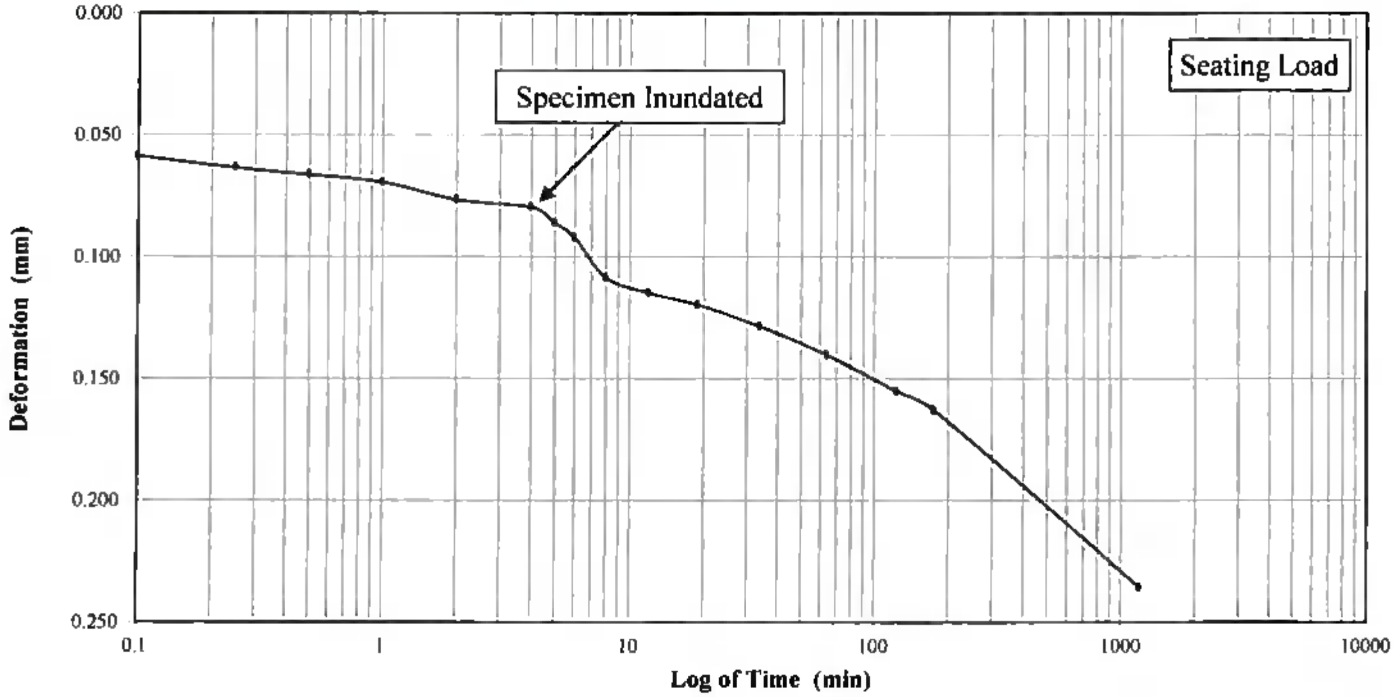
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

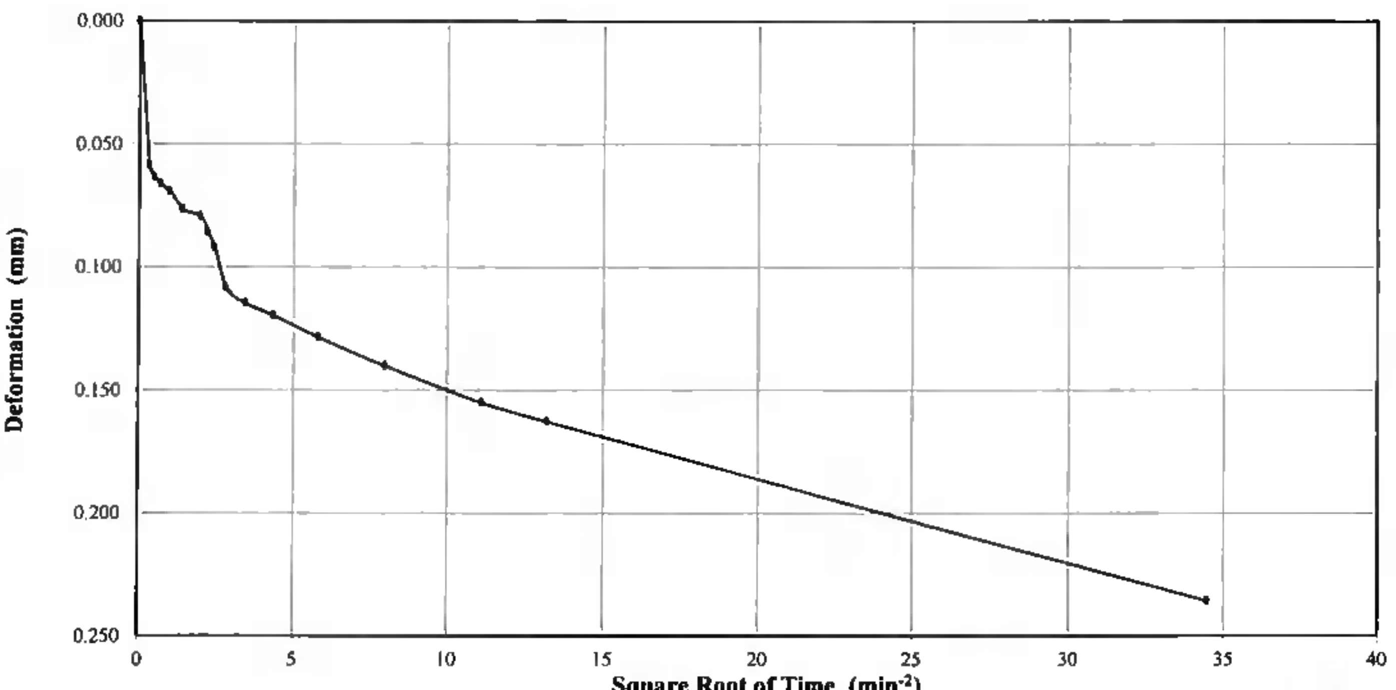
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107A, S-01 (6.0-8.0')
Lab Sample No:	13K149

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





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Excel Geotechnical Testing, Inc.
"Excellence In Testing"

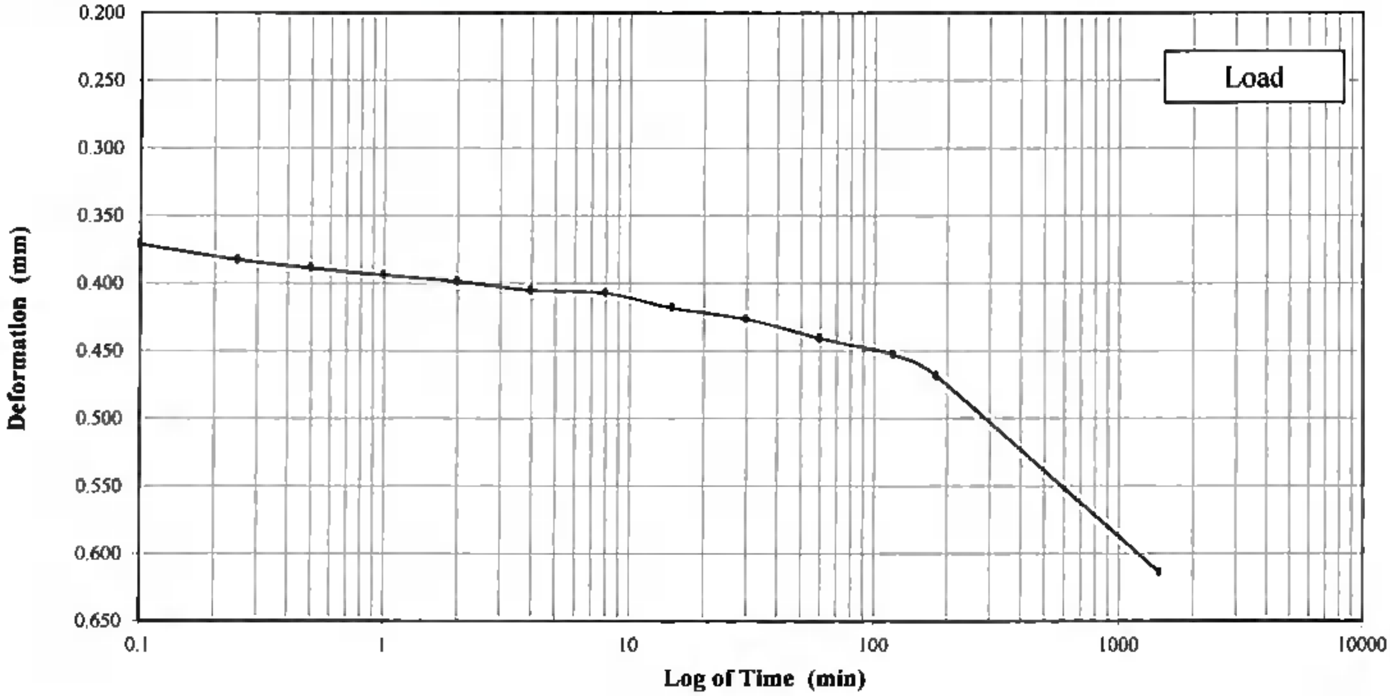
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

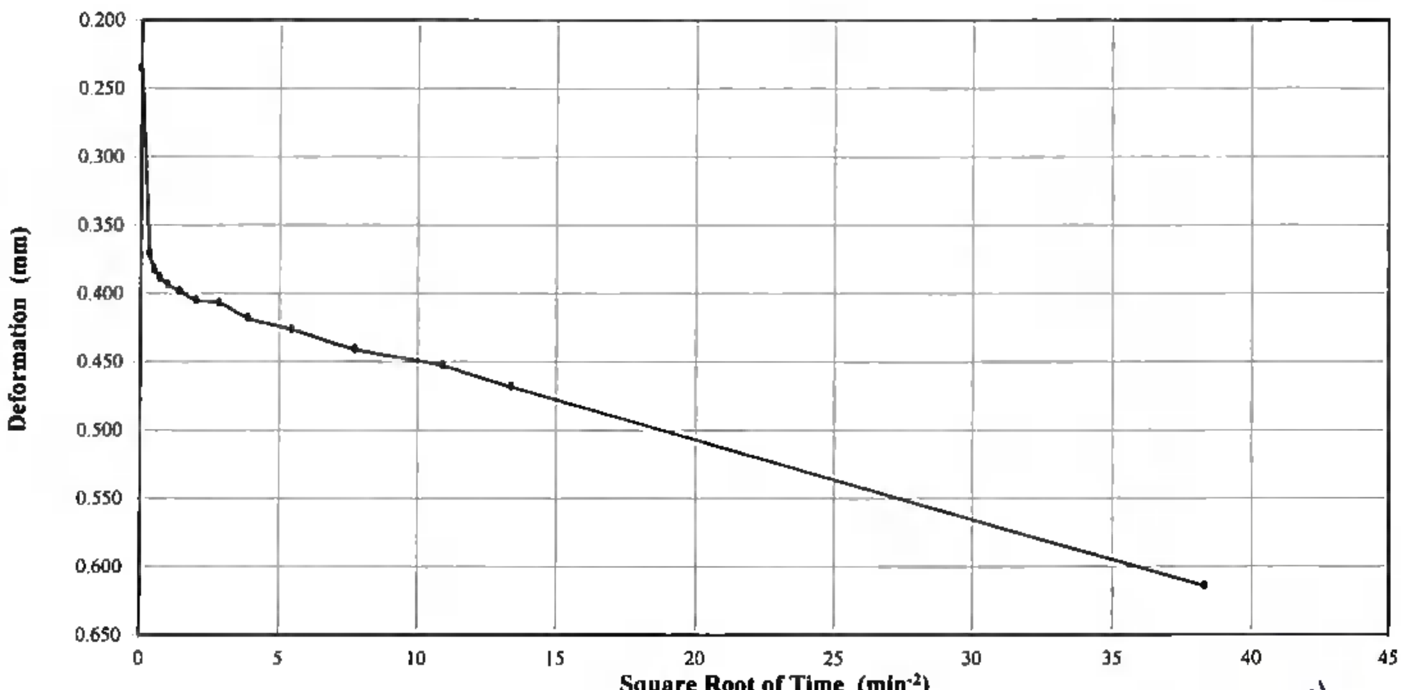
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107A, S-01 (6.0-8.0')
Lab Sample No:	13K149

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf

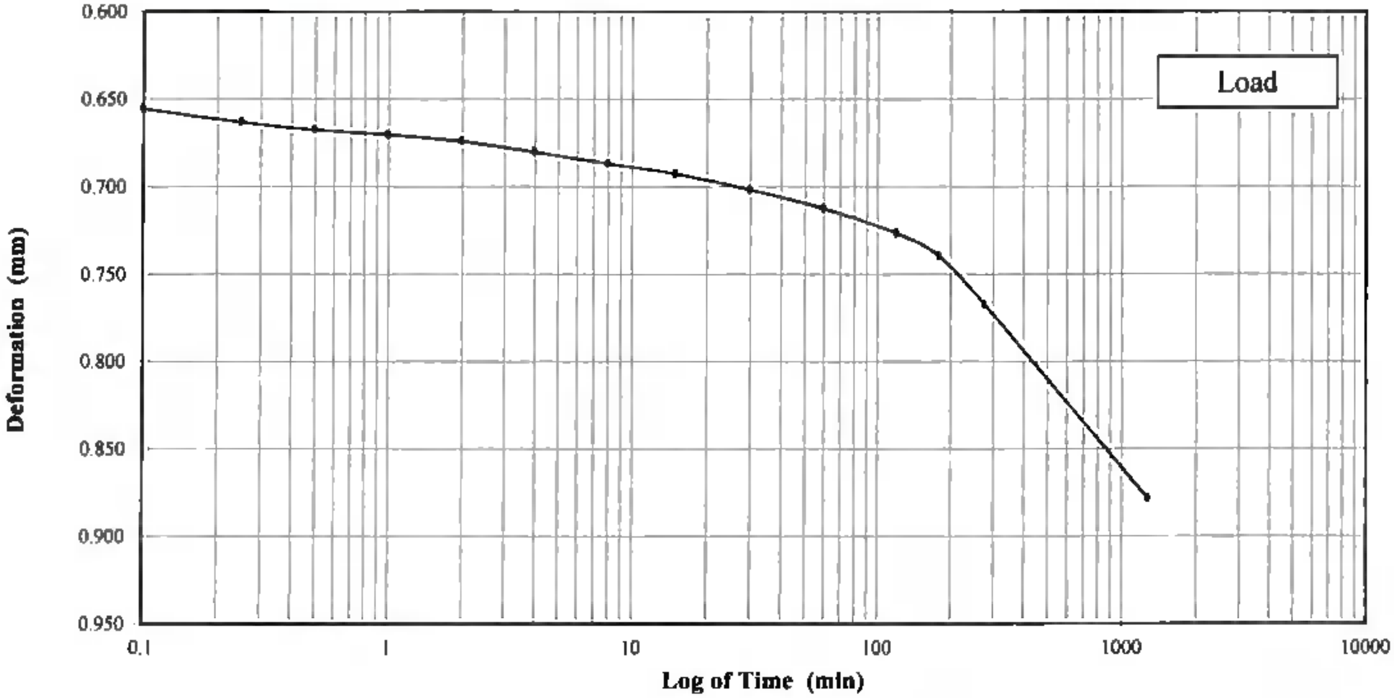


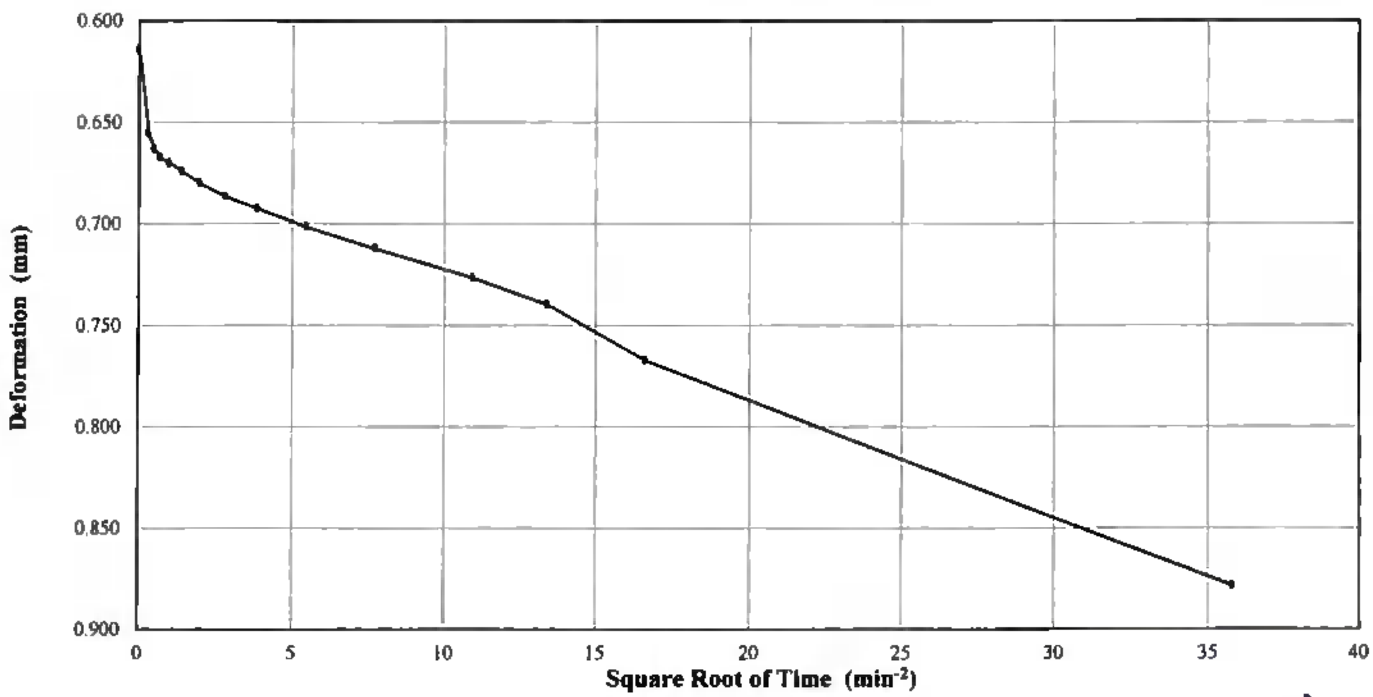


2-3-14
NSR

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-107A, S-01 (6.0-8.0') Lab Sample No: 13K149

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 3 - 1000 psf





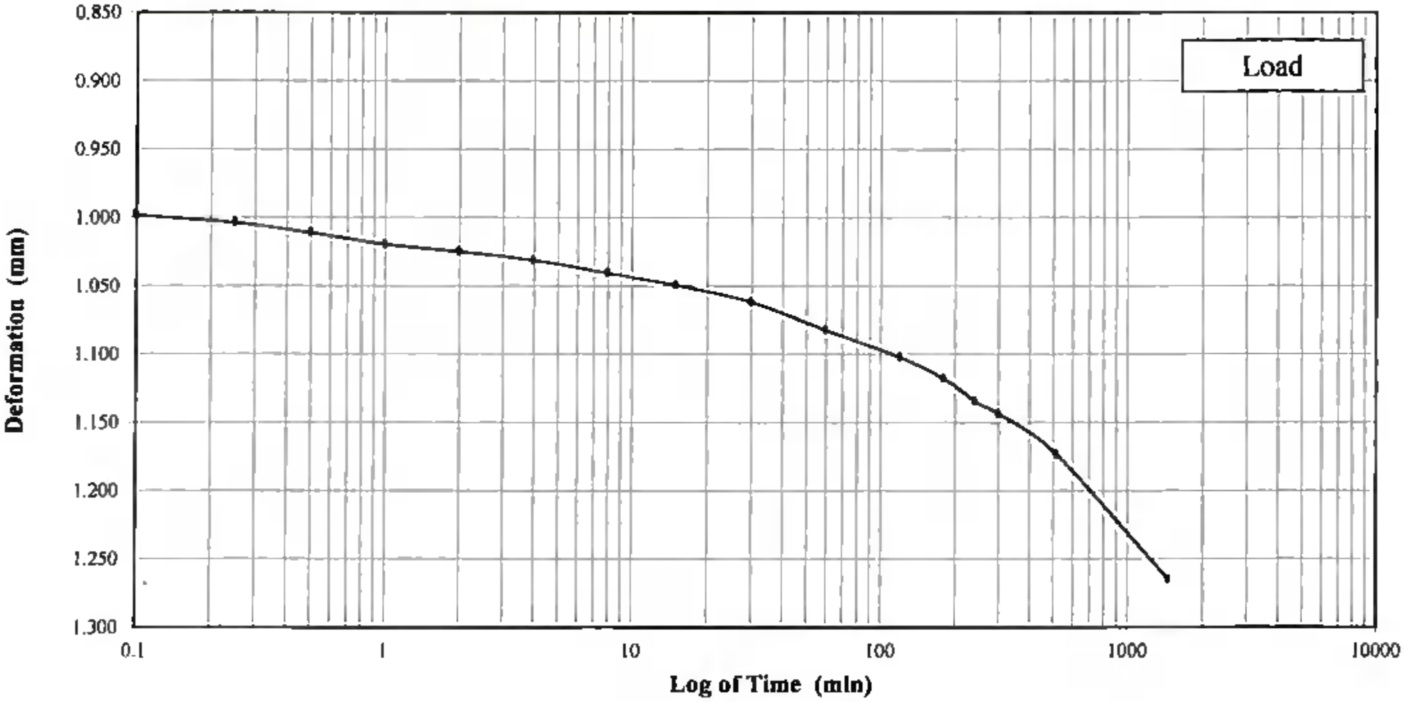
2-3-14 NSR

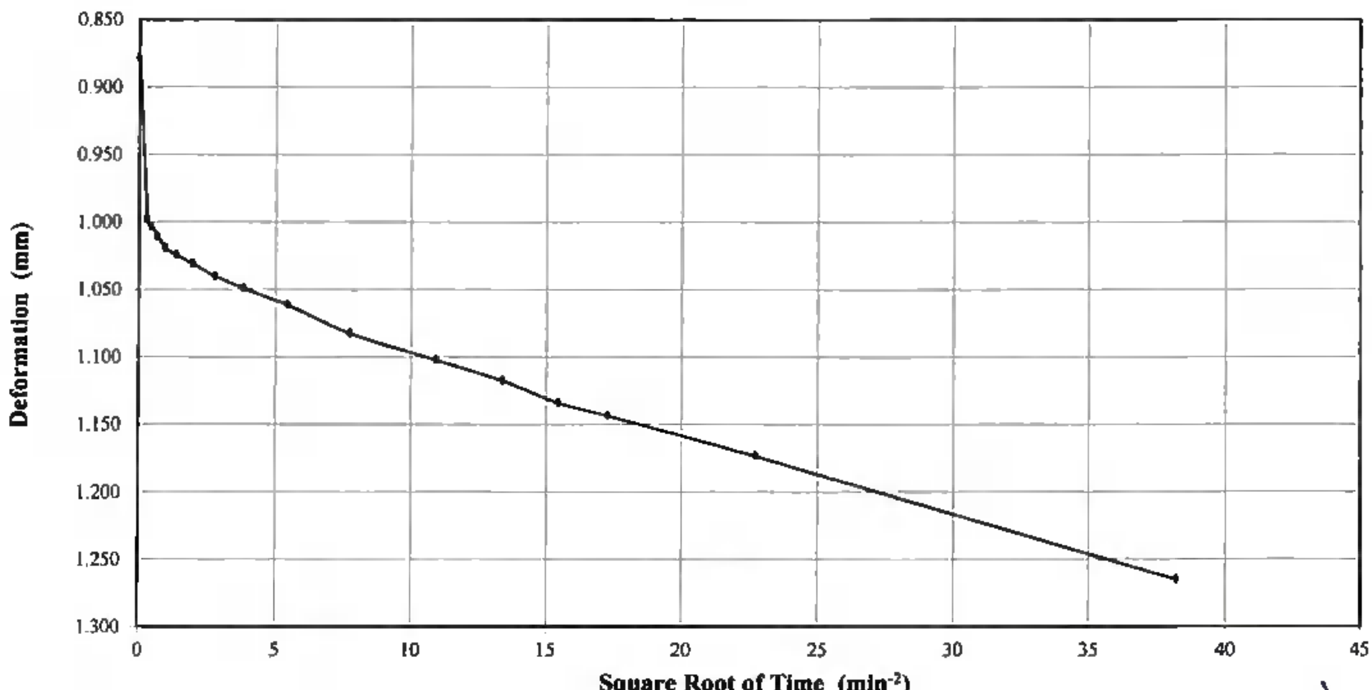
Excel Geotechnical Testing, Inc. *"Excellence In Testing"* 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-107A, S-01 (6.0-8.0') Lab Sample No: 13K149

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 4 - 2000 psf





2-3-14
NJK

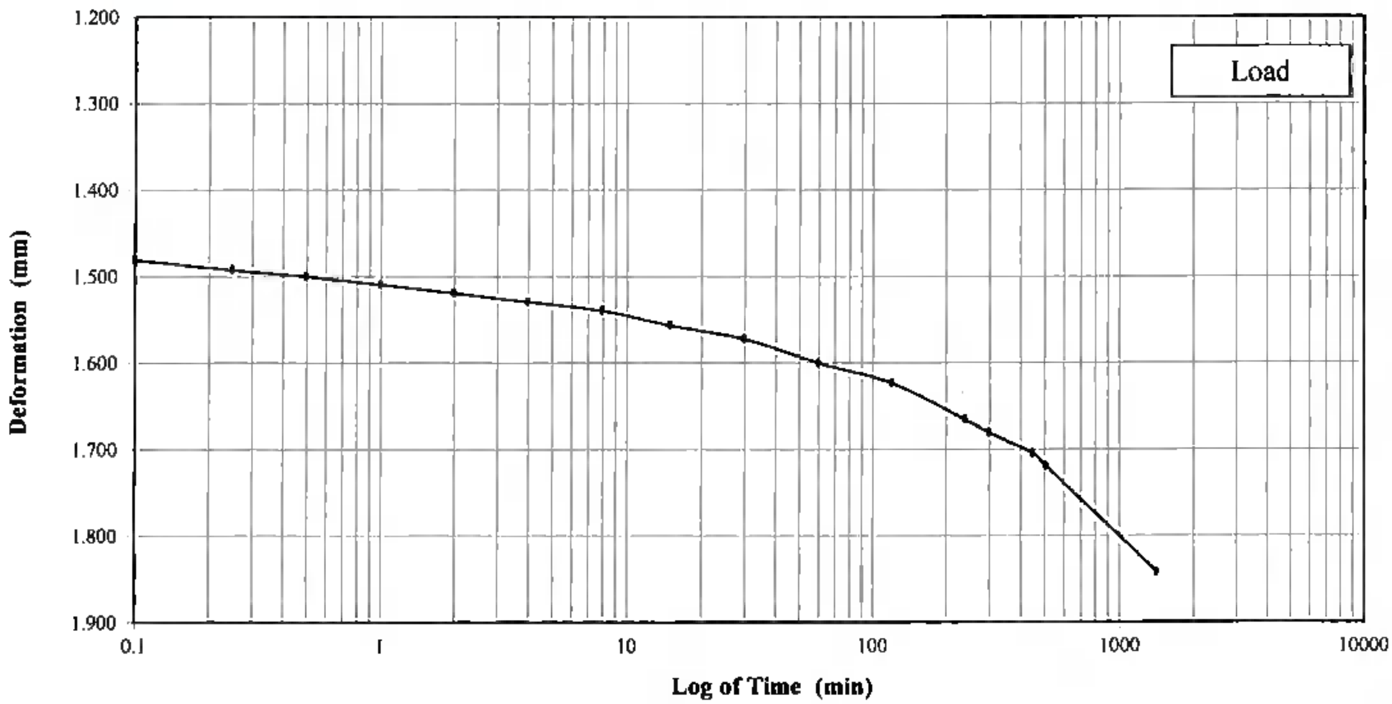
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

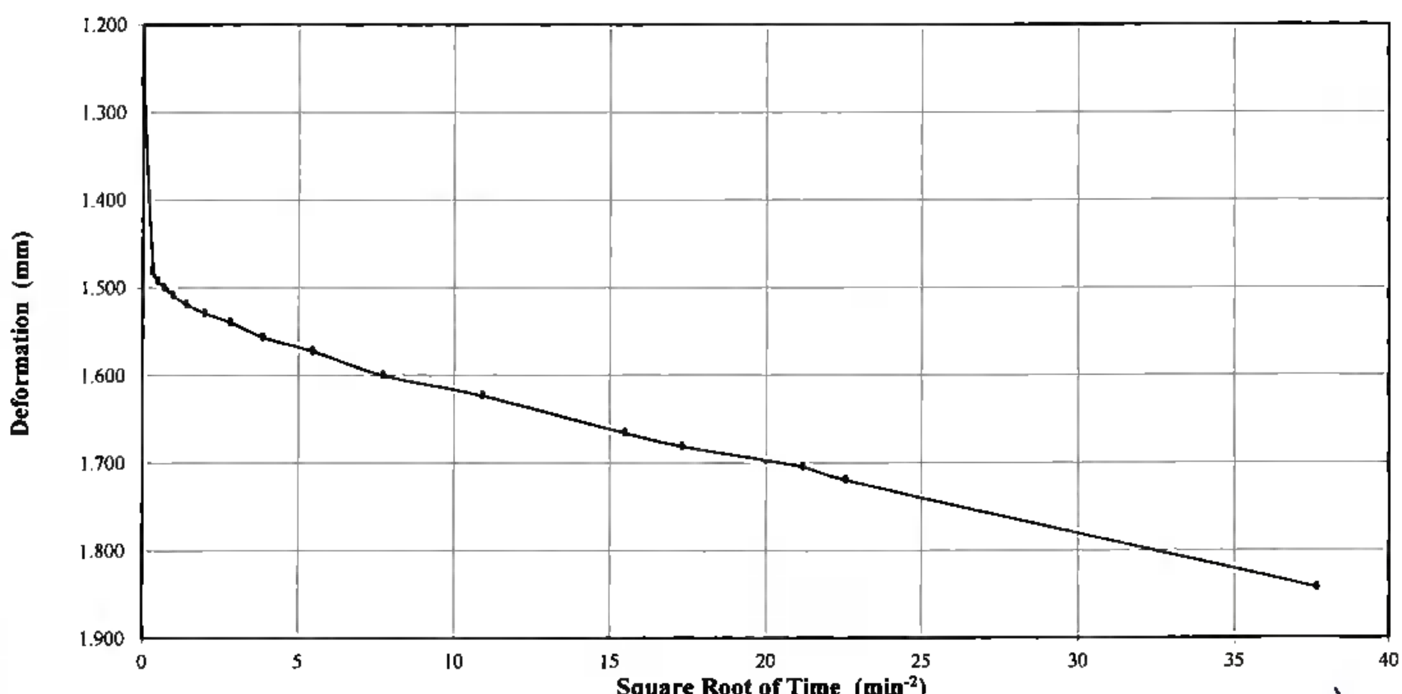
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107A, S-01 (6.0-8.0')
Lab Sample No:	13K149

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 5 - 4000 psf





2-3-14
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Excel Geotechnical Testing, Inc.
"Excellence in Testing"

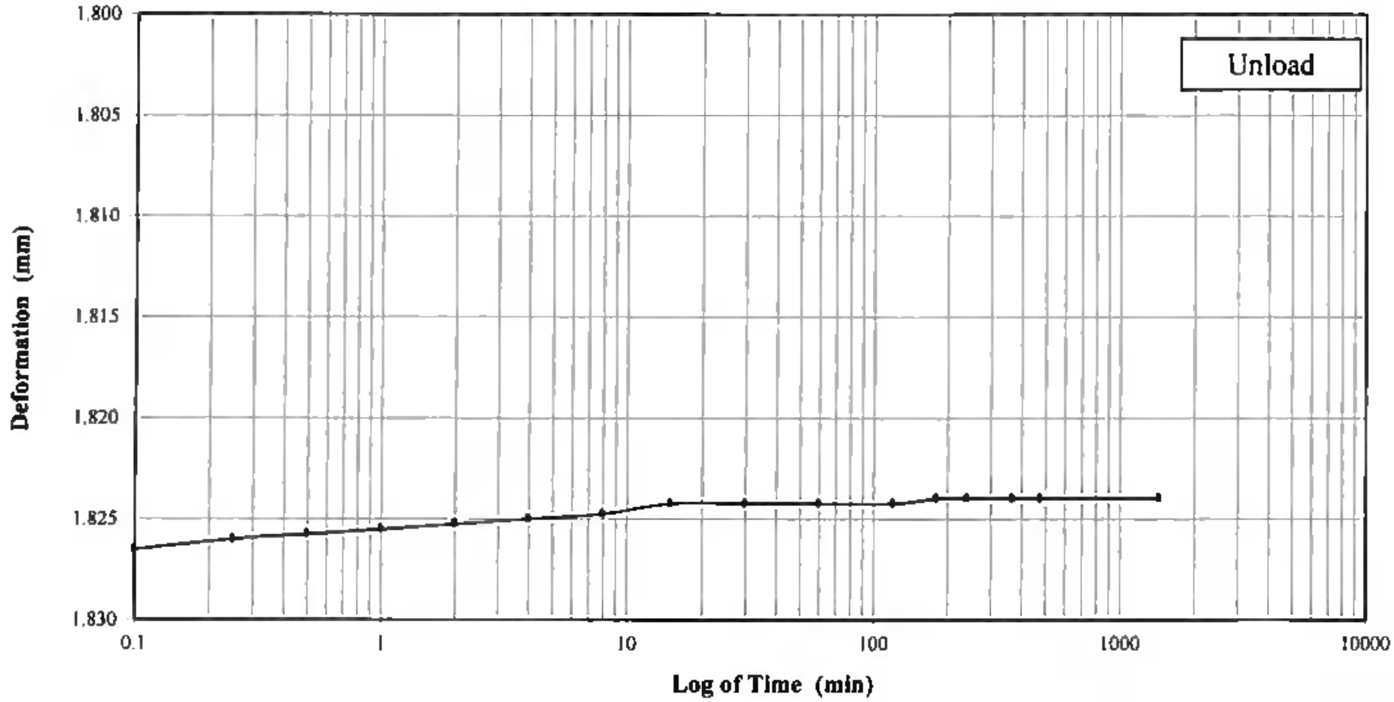
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

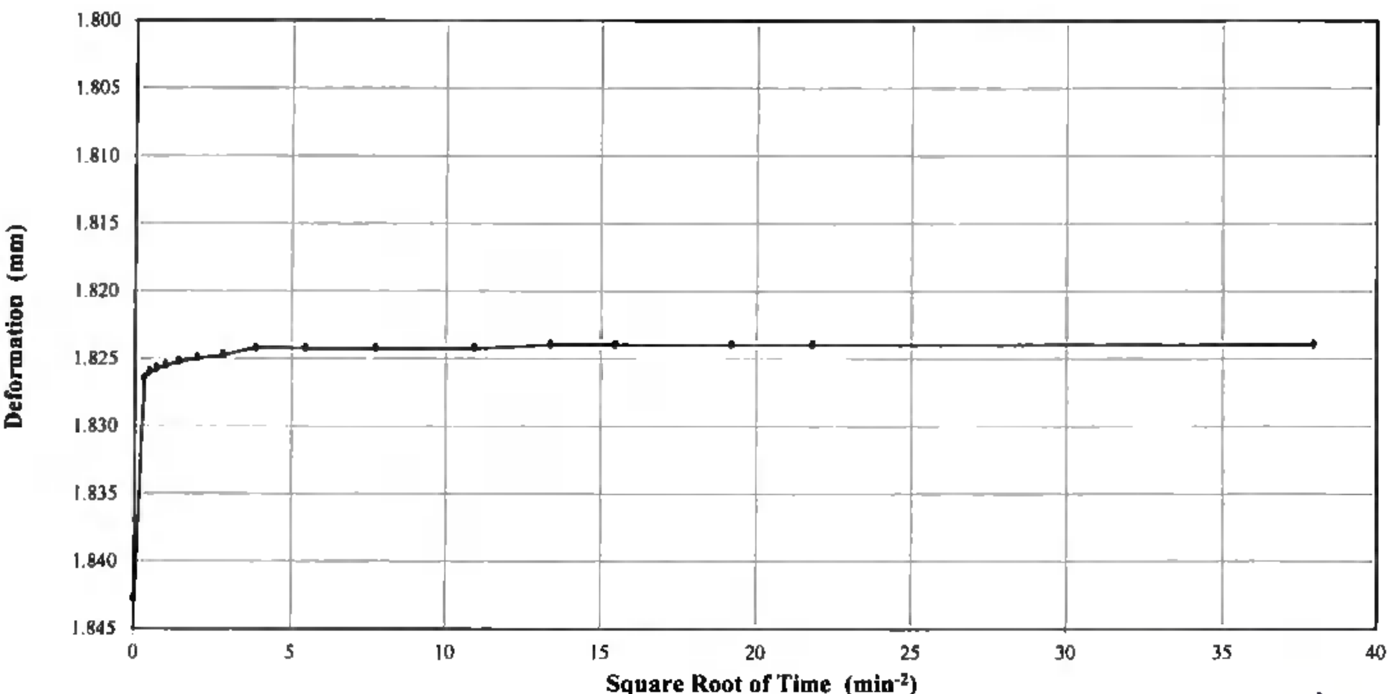
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107A, S-01 (6.0-8.0')
Lab Sample No:	13K149

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 6 - 2000 psf





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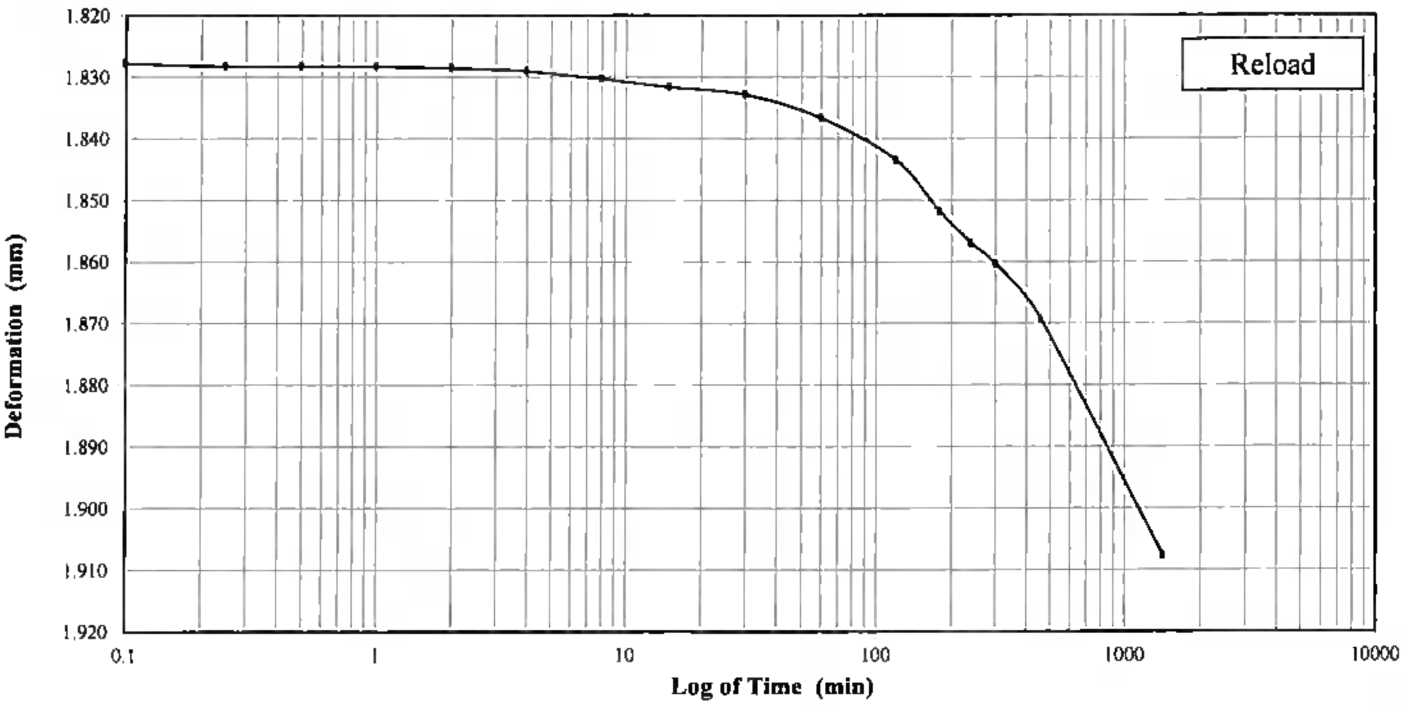
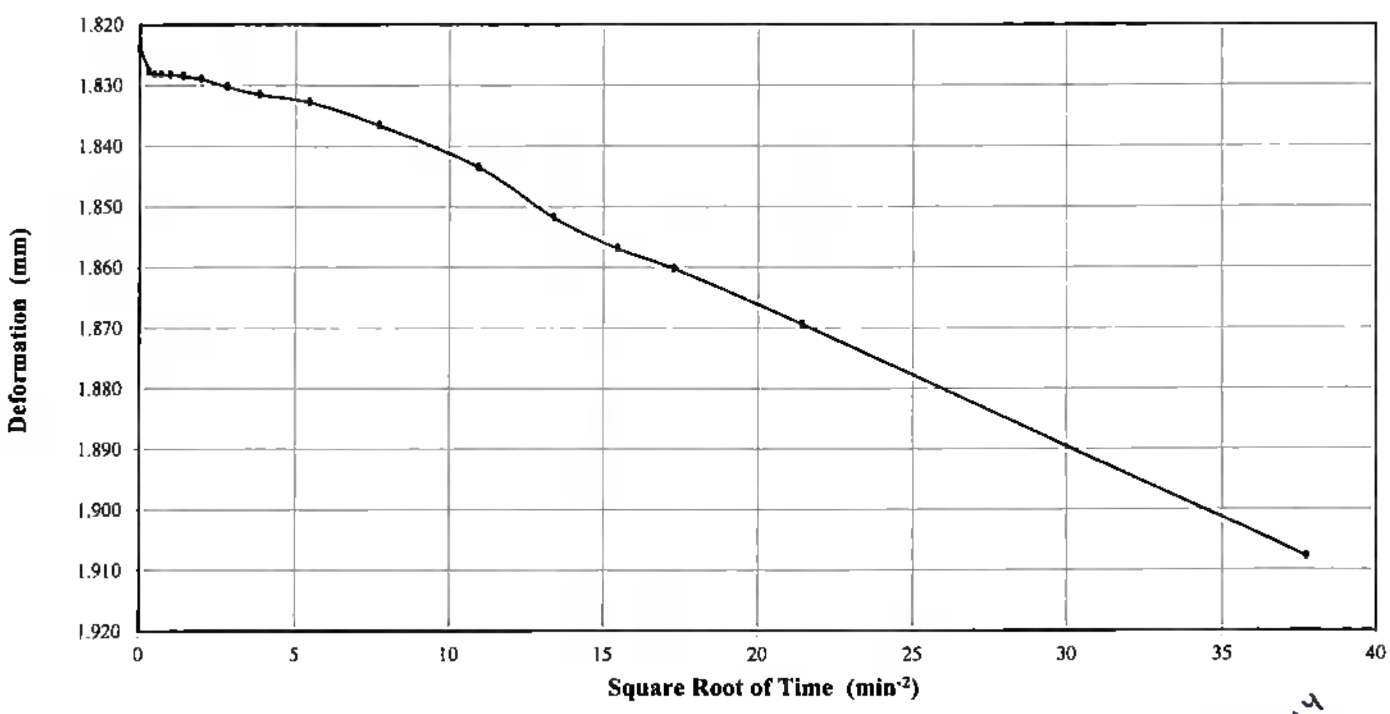
Excel Geotechnical Testing, Inc. "Excellence in Testing" 953 Forrest Street, Roswell, Georgia 30075 Tel: (770) 910 7537 Fax: (770) 910 7538	Project Name: Winyah Generating Station Project No: 618 Client Sample ID: SPT-107A, S-01 (6.0-8.0') Lab Sample No: 13K149

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 7 - 4000 psf





2-3-14
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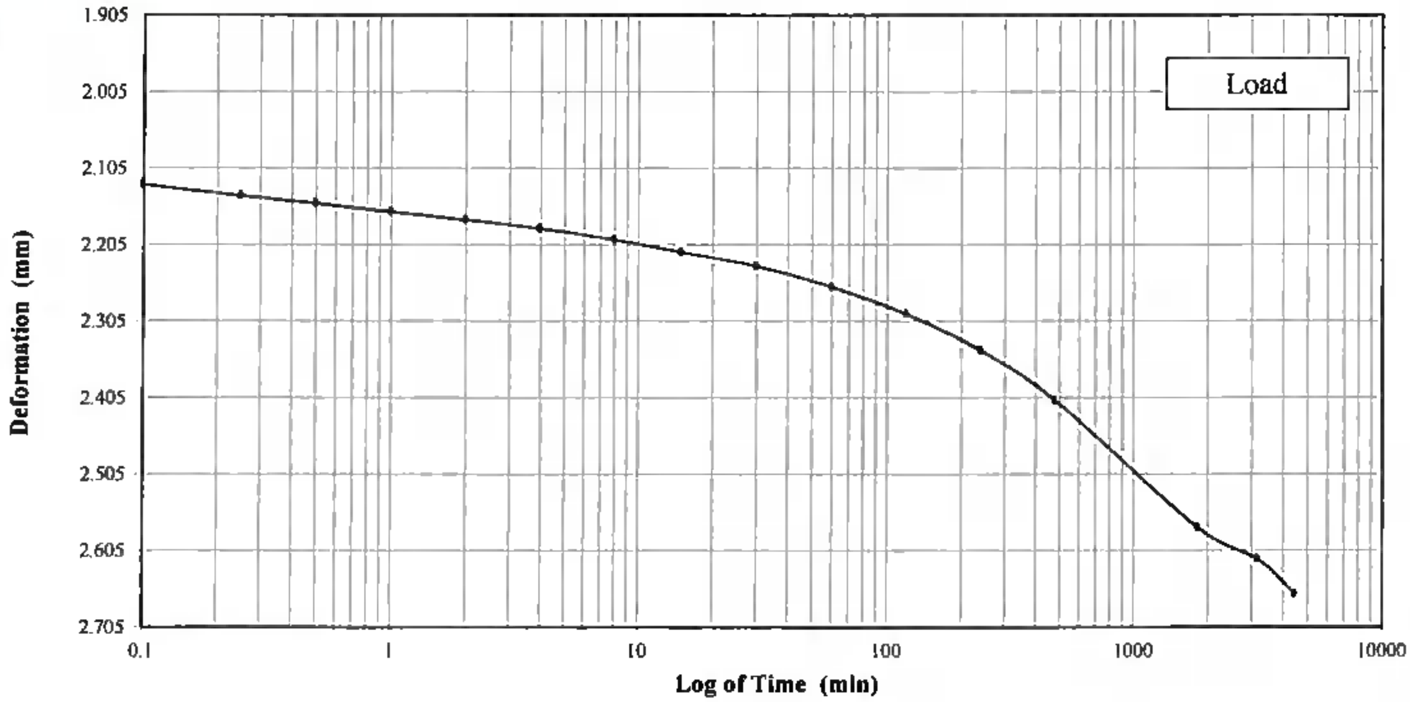
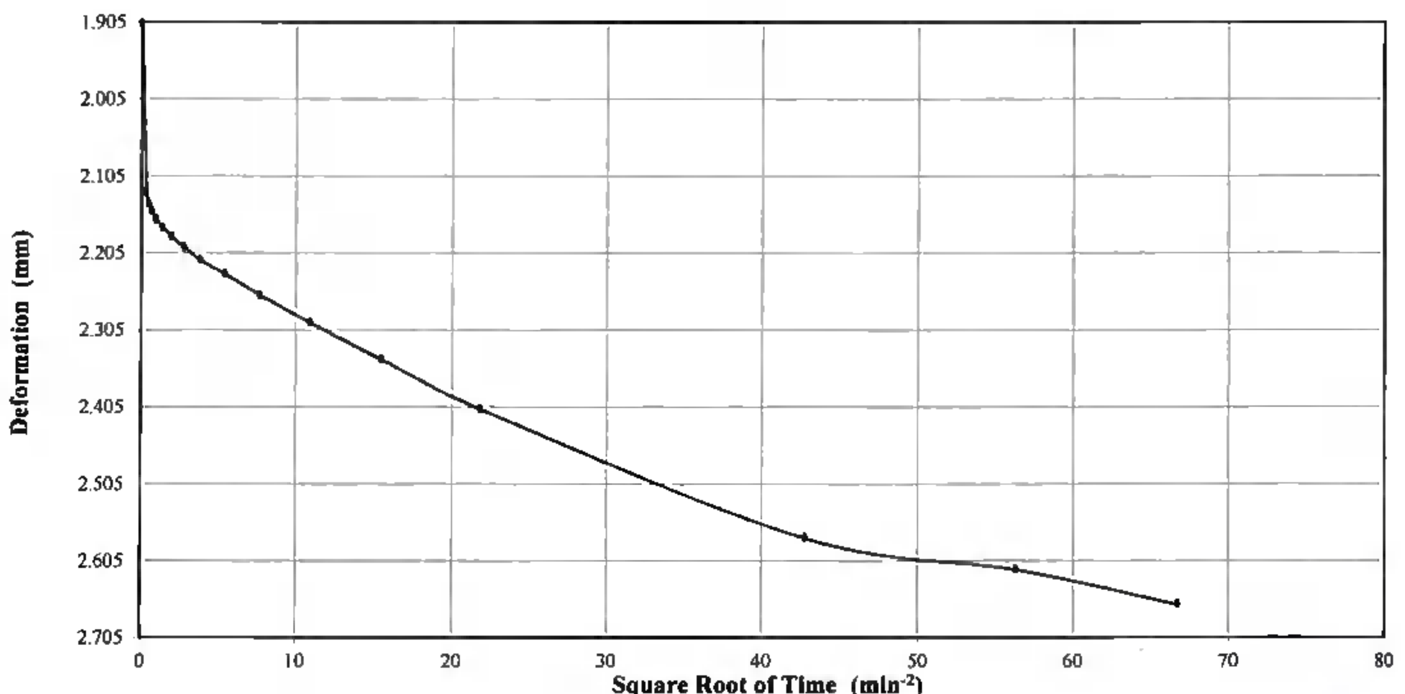
Excel Geotechnical Testing, Inc.	Project Name: Winyah Generating Station
"Excellence in Testing"	Project No: 618
953 Forrest Street, Roswell, Georgia 30075	Client Sample ID: SPT-107A, S-01 (6.0-8.0')
Tel: (770) 910 7537 Fax: (770) 910 7538	Lab Sample No: 13K149

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** **Figure 8 - 8000 psf**





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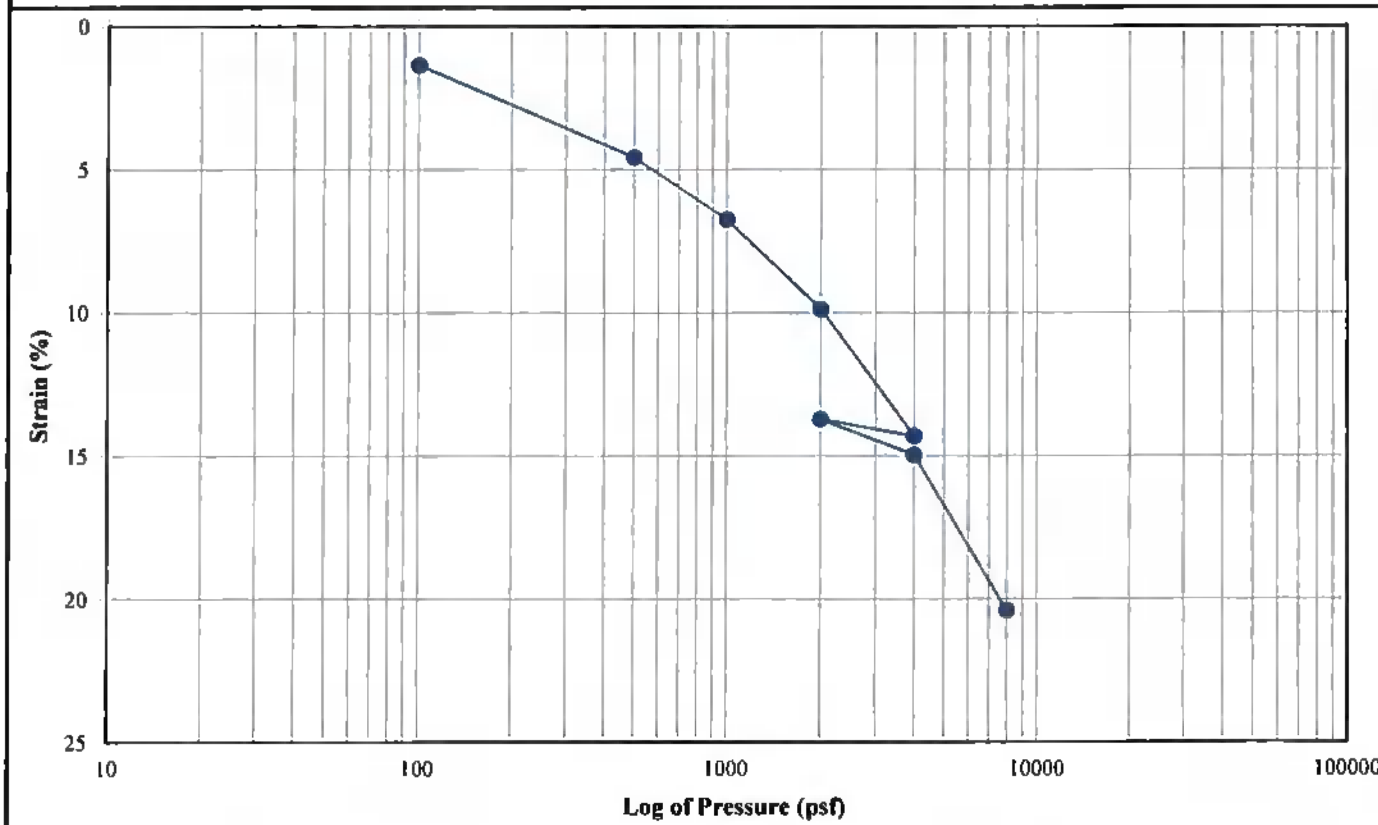
Excel Geotechnical Testing, Inc
"Excellence in Testing"
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

Project Name: Winyah Generating Station
Project No: 618
Client Sample ID: SPT-107B, S-03 (16.0-18.0')
Lab Sample No: 13K155

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST



Client Sample ID	Lab Sample No.	Specimen Quality 1-10 (Bad to Good)	Test Specimen Initial Conditions: Height (cm)	Diameter (cm)	Dry Unit Weight (pcf)	Moisture Content (%)	Consolidation Pressure (psf)	Pressure Increment Duration (min)	Accumu.[1] Vertical Strain (%)	Figure No.	Remarks
SPT-107B, S-03 (16.0-18.0')	13K155	7	2.54	6.35	50.3	83.2	100	1332	1.35	1	
							500	1424	4.58	2	
							1000	1437	6.74	3	
							2000	1430	9.87	4	
							4000	1721	14.31	5	
							2000	1320	13.72	6	
							4000	1253	14.97	7	
							8000	1455	20.40	8	

Notes:

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

2-4-14
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Excel Geotechnical Testing, Inc.
"Excellence in Testing"

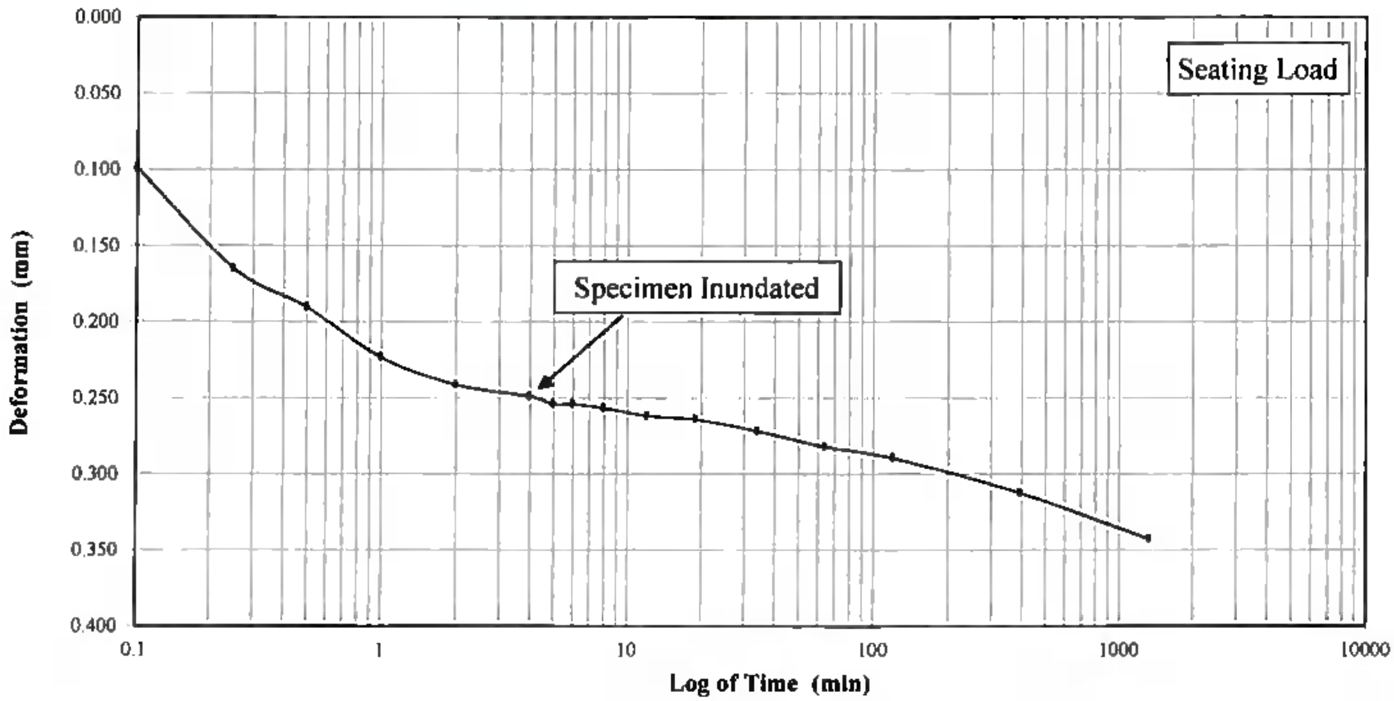
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

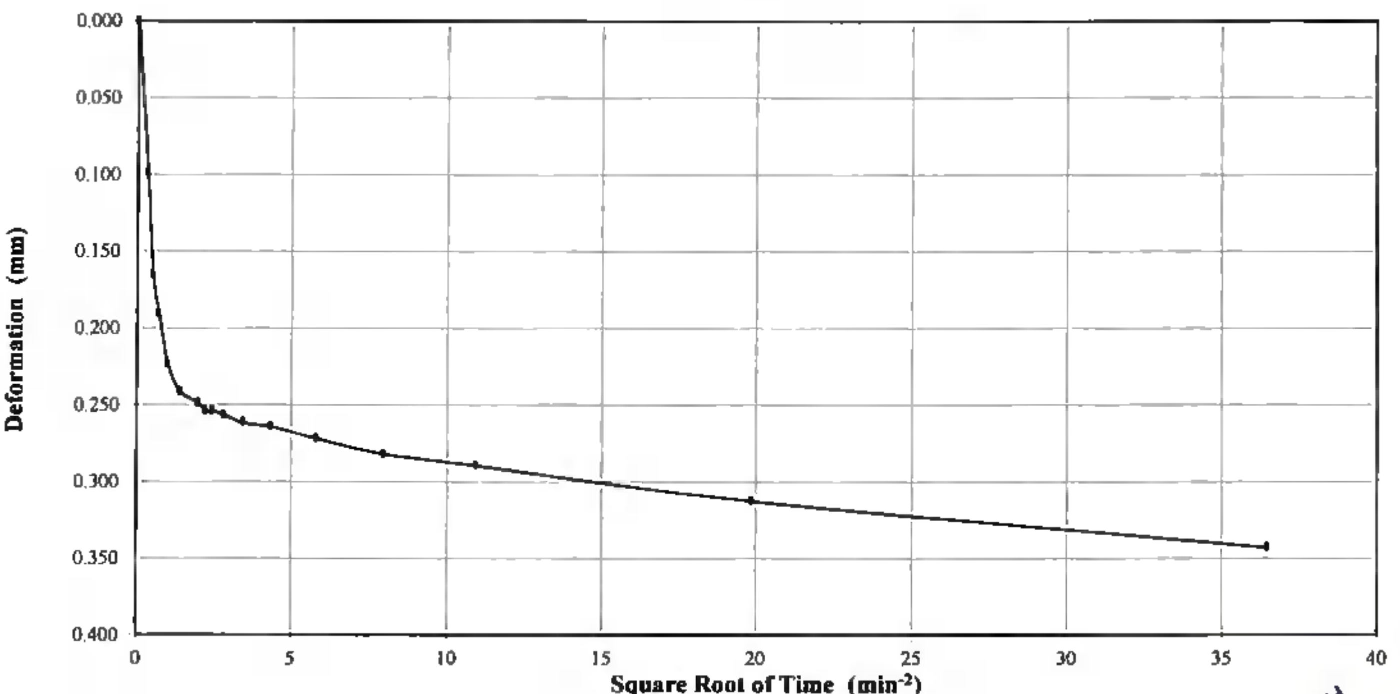
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107B, S-03 (16.0-18.0')
Lab Sample No:	13K155

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 1 - 100 psf





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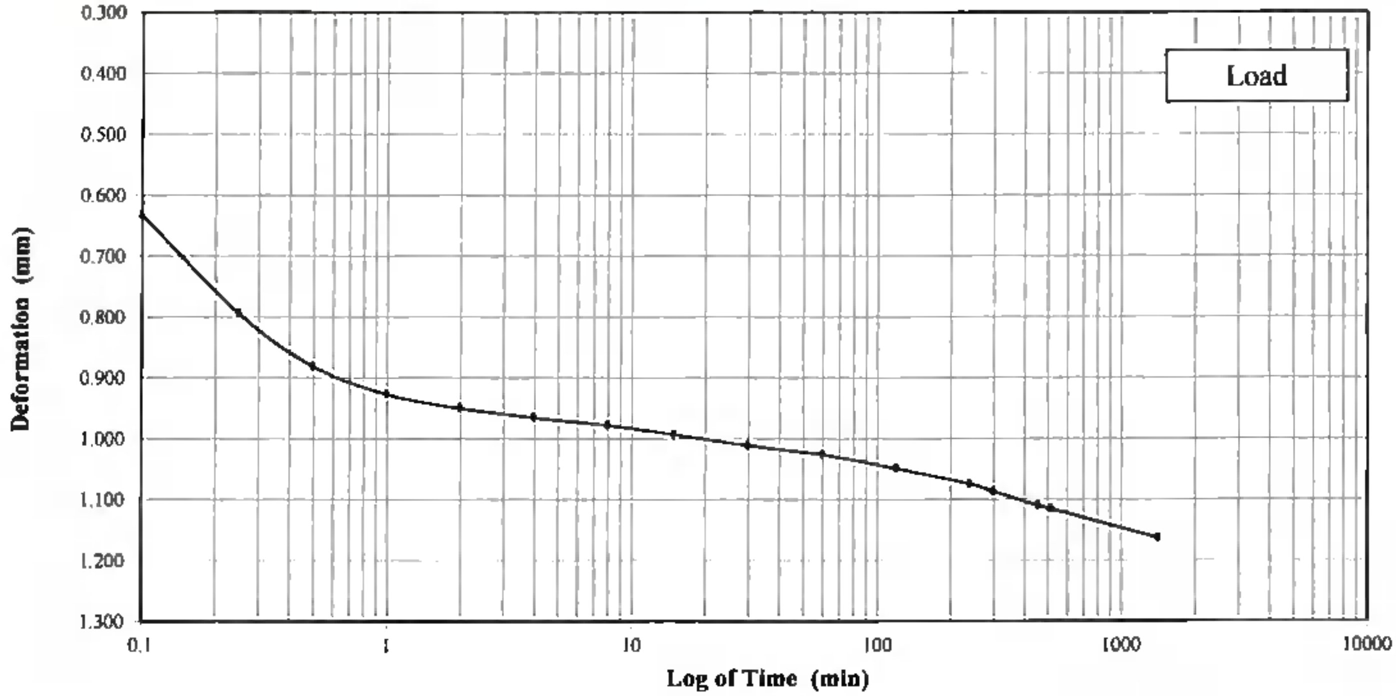
953 Forrest Street, Roswell, Georgia 30075
Tel: (770) 910 7537 Fax: (770) 910 7538

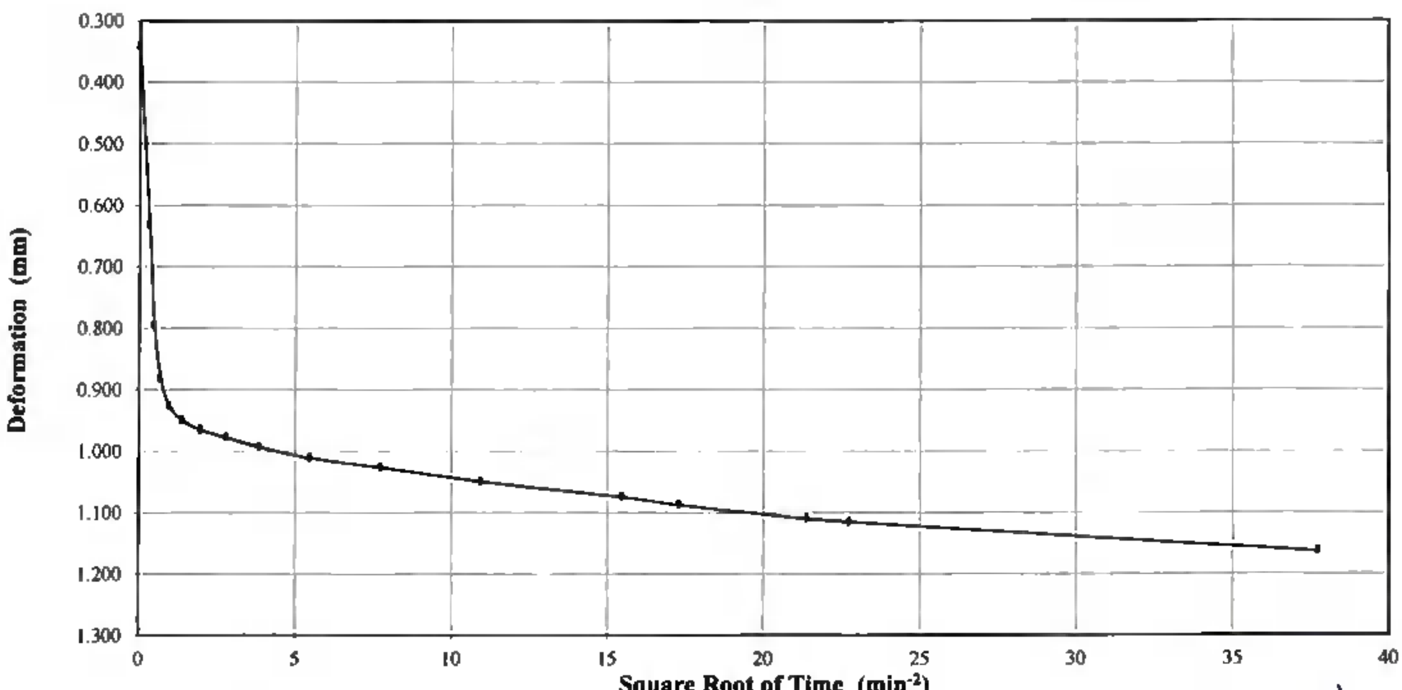
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107B, S-03 (16.0-18.0')
Lab Sample No:	13K155

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 2 - 500 psf





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953 Forrest Street, Roswell, Georgia 30075
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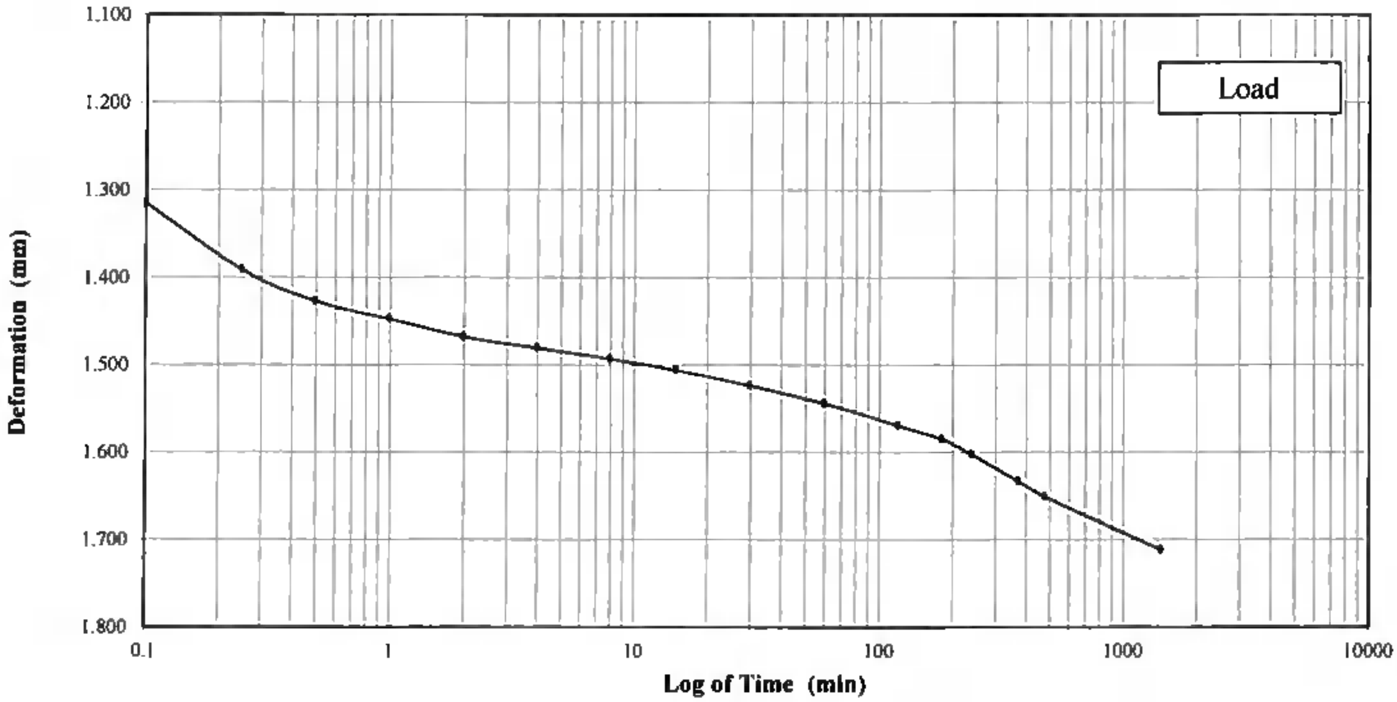
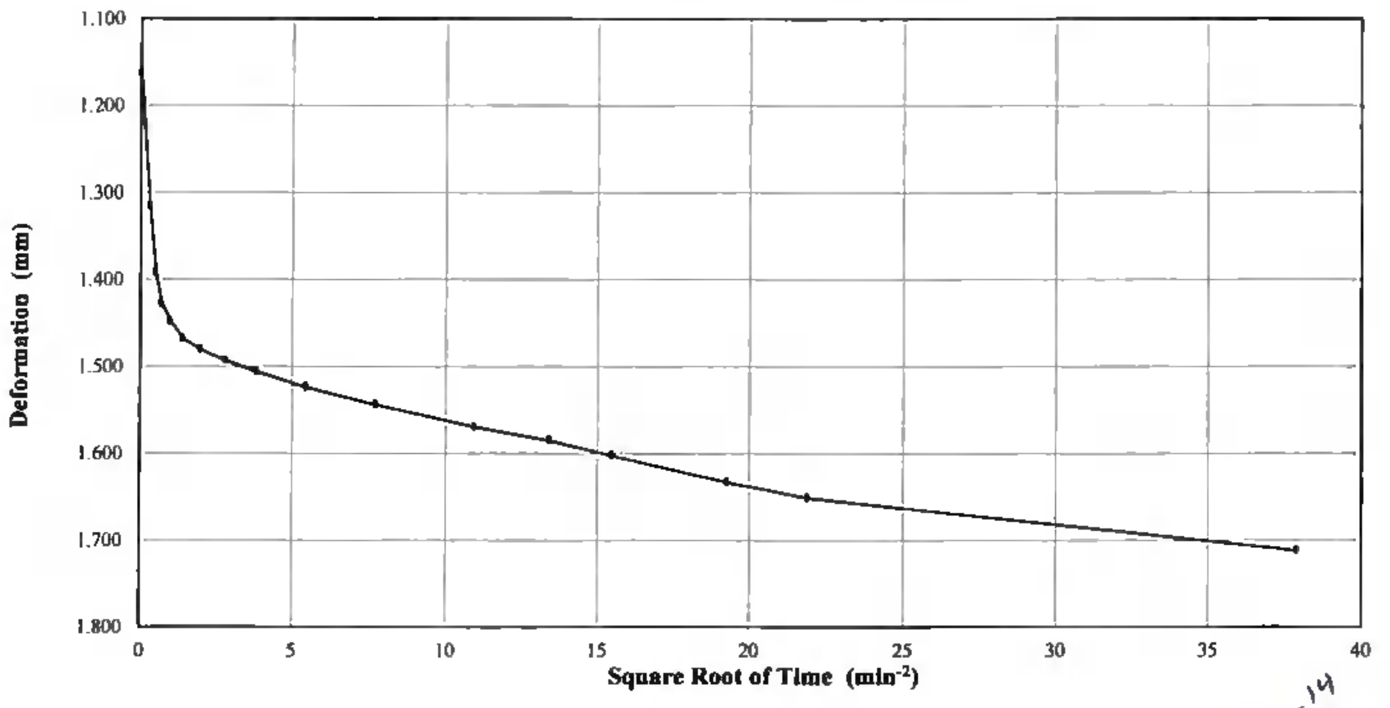
Project Name:	Winyah Generating Station
Project No:	618
Client Sample ID:	SPT-107B, S-03 (16.0-18.0')
Lab Sample No:	13K155

ASTM D 2435

ONE-DIMENSIONAL CONSOLIDATION TEST

Figure 3 - 1000 psf

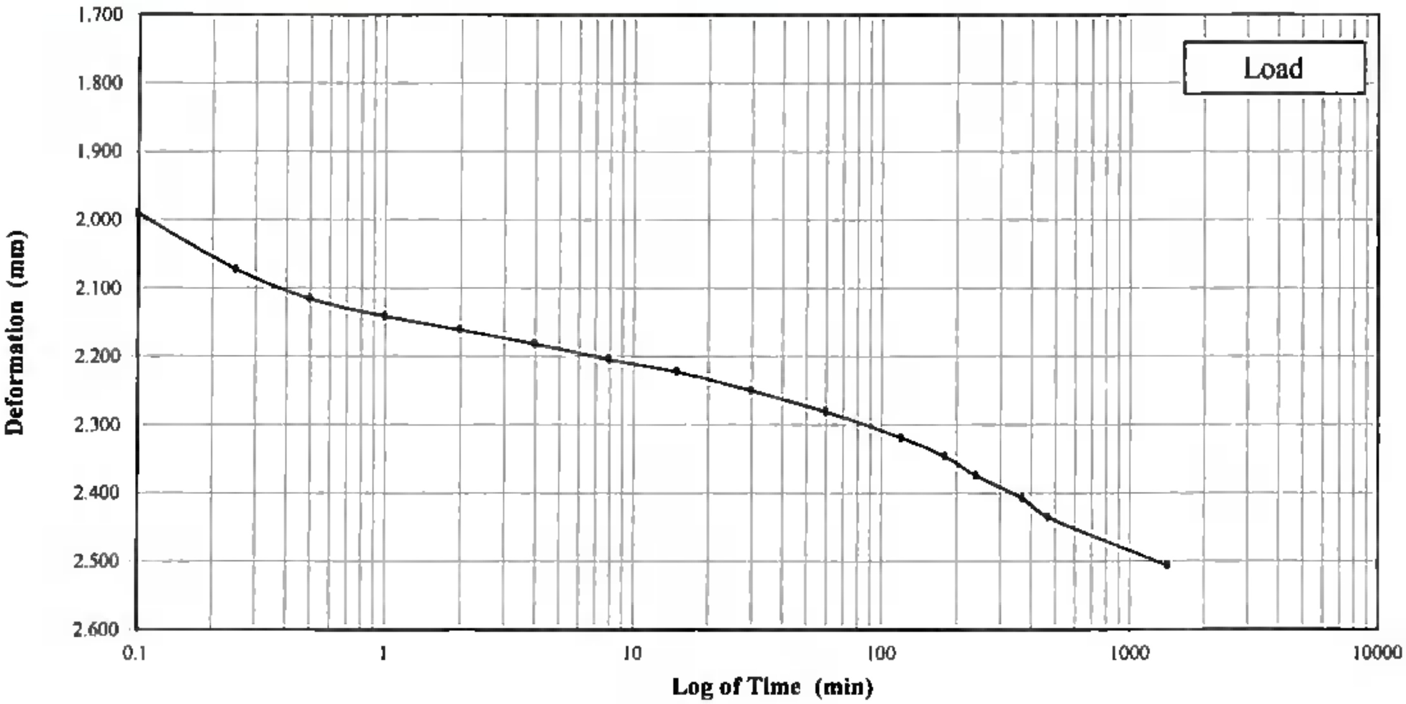


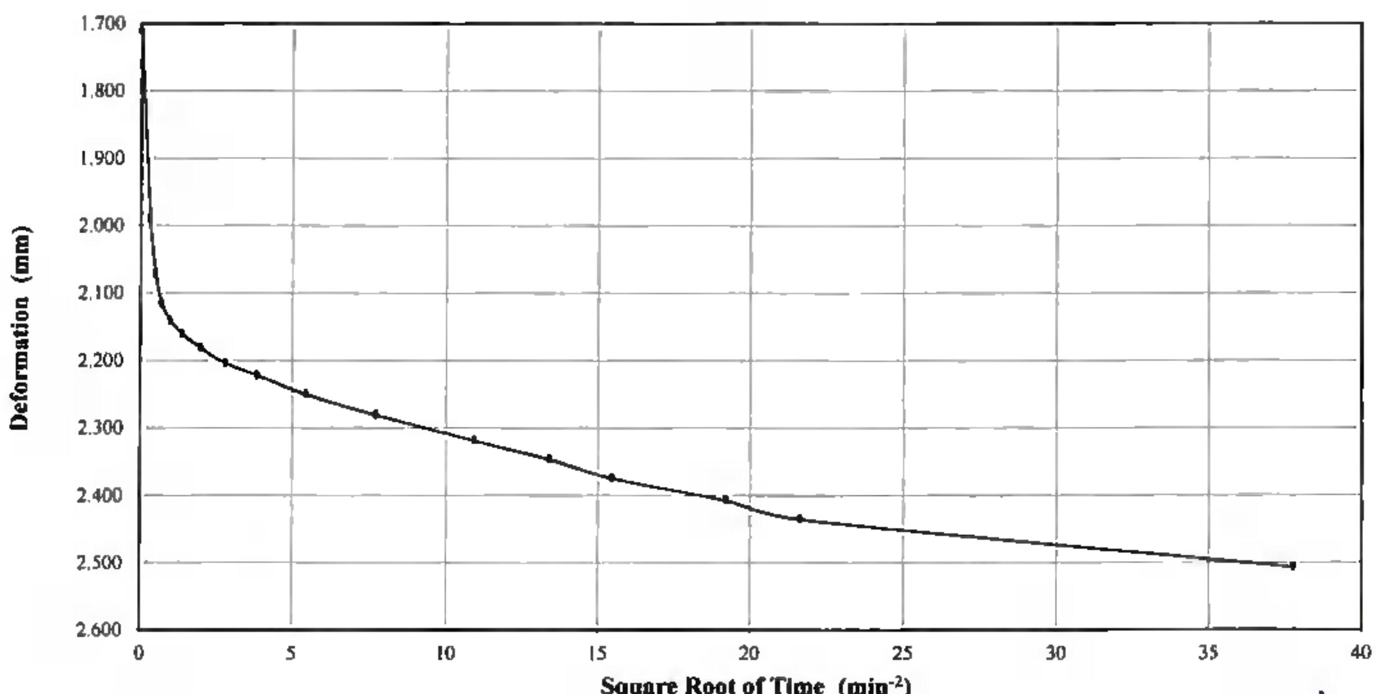


2-4-14
NSR

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-107B, S-03 (16.0-18.0') Lab Sample No: 13K155

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 4 - 2000 psf

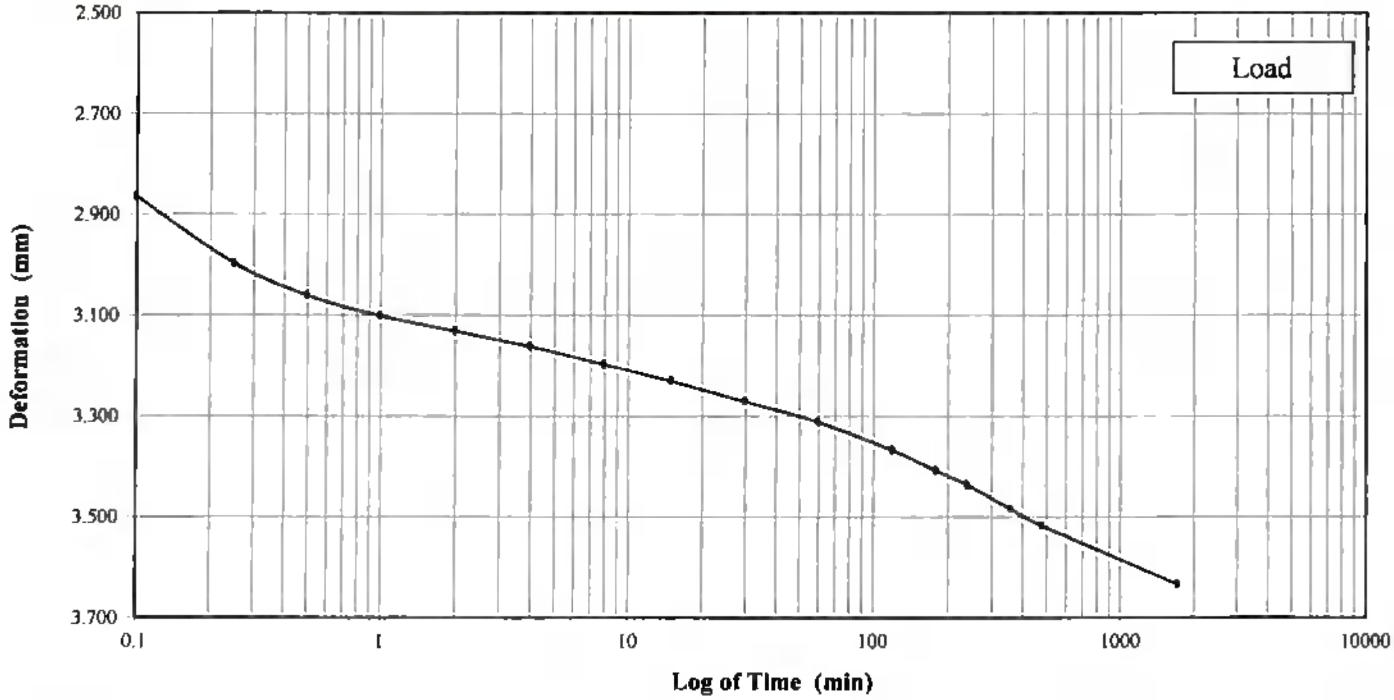


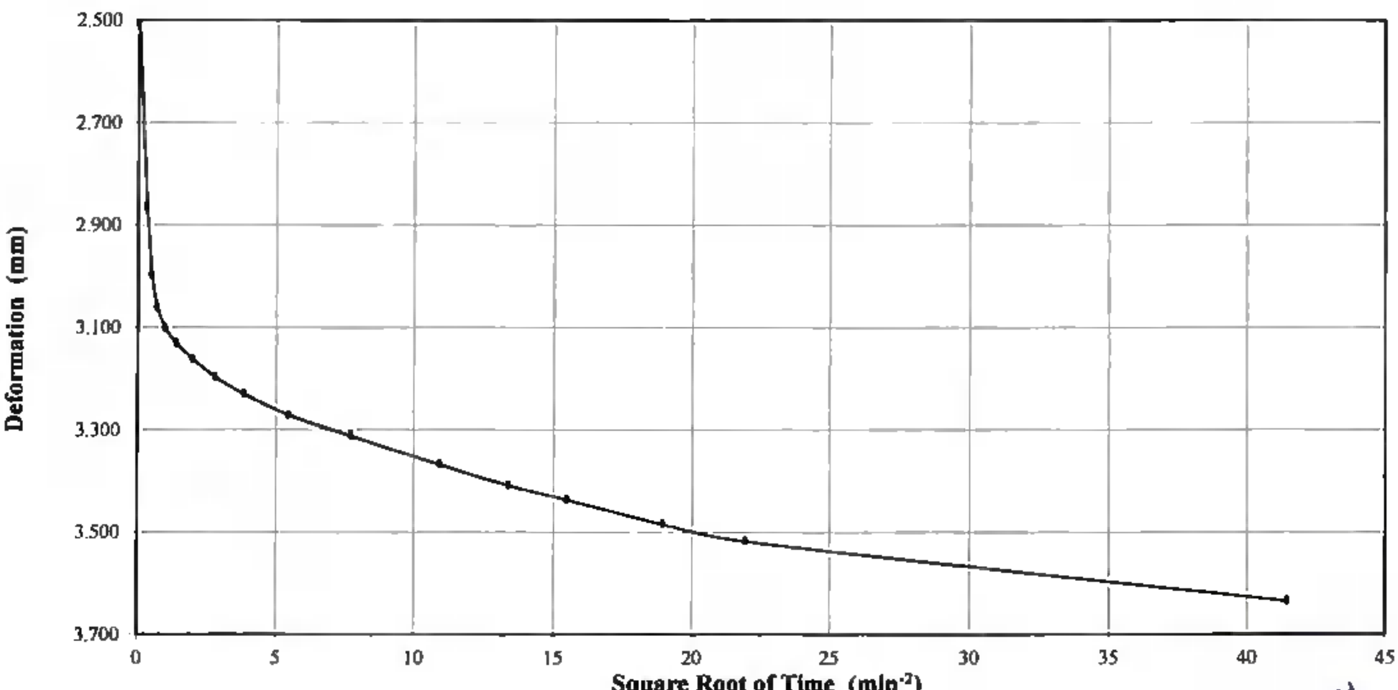


2-4-14
NSR

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-107B, S-03 (16.0-18.0') Lab Sample No: 13K155

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** **Figure 5 - 4000 psf**

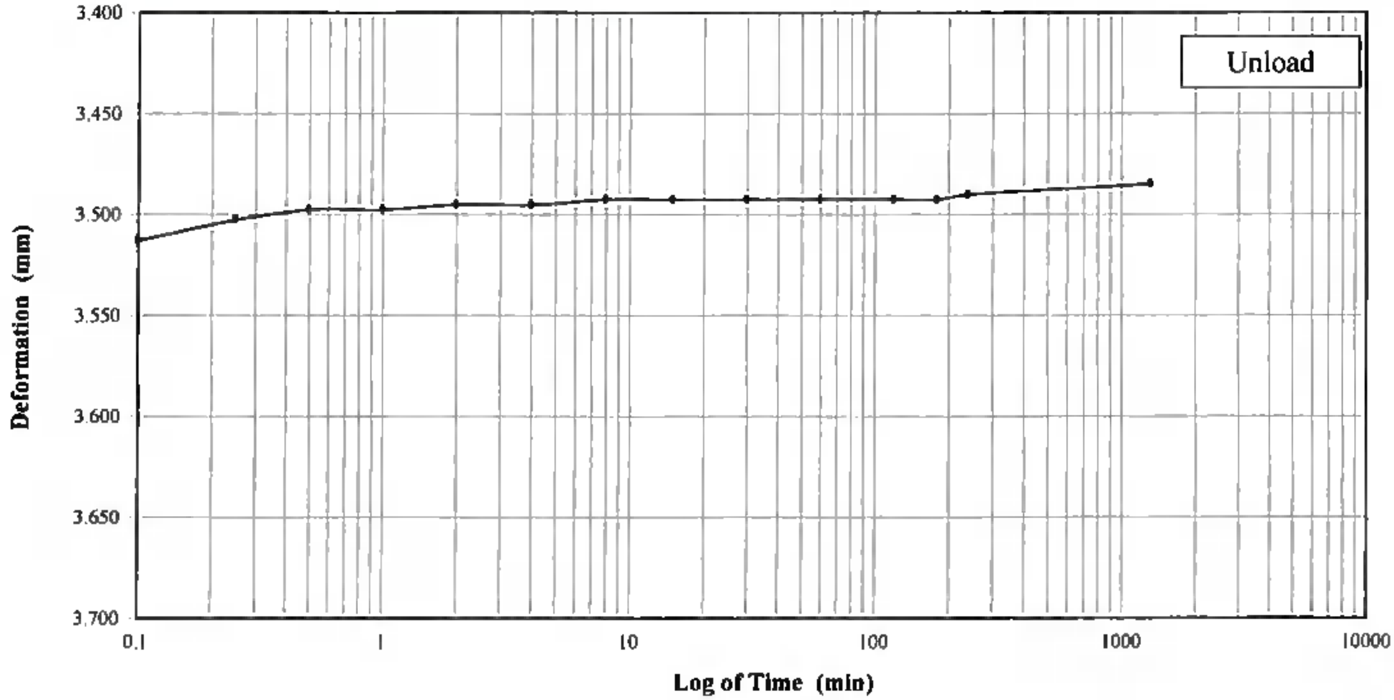


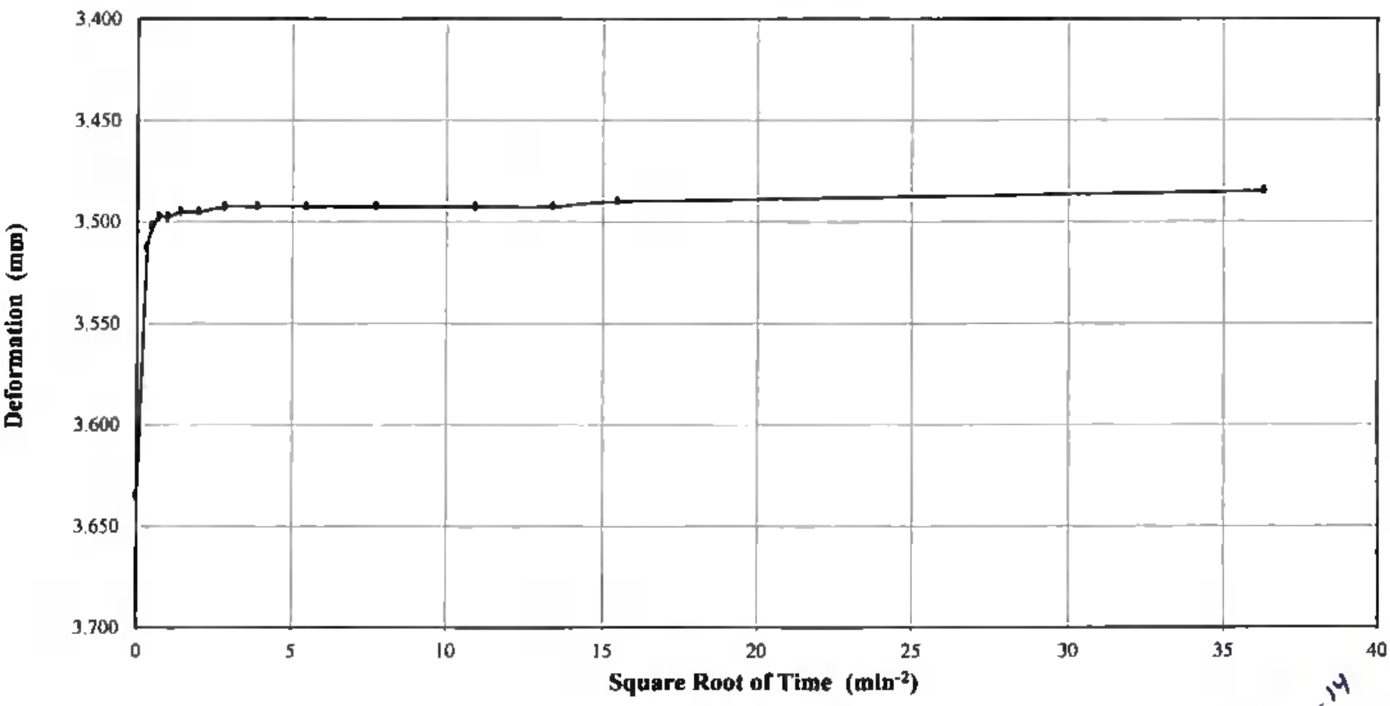


2-11-14 NSR

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-107B, S-03 (16.0-18.0') Lab Sample No: 13K155

ASTM D 2435 — **ONE-DIMENSIONAL CONSOLIDATION TEST** — Figure 6 - 2000 psf



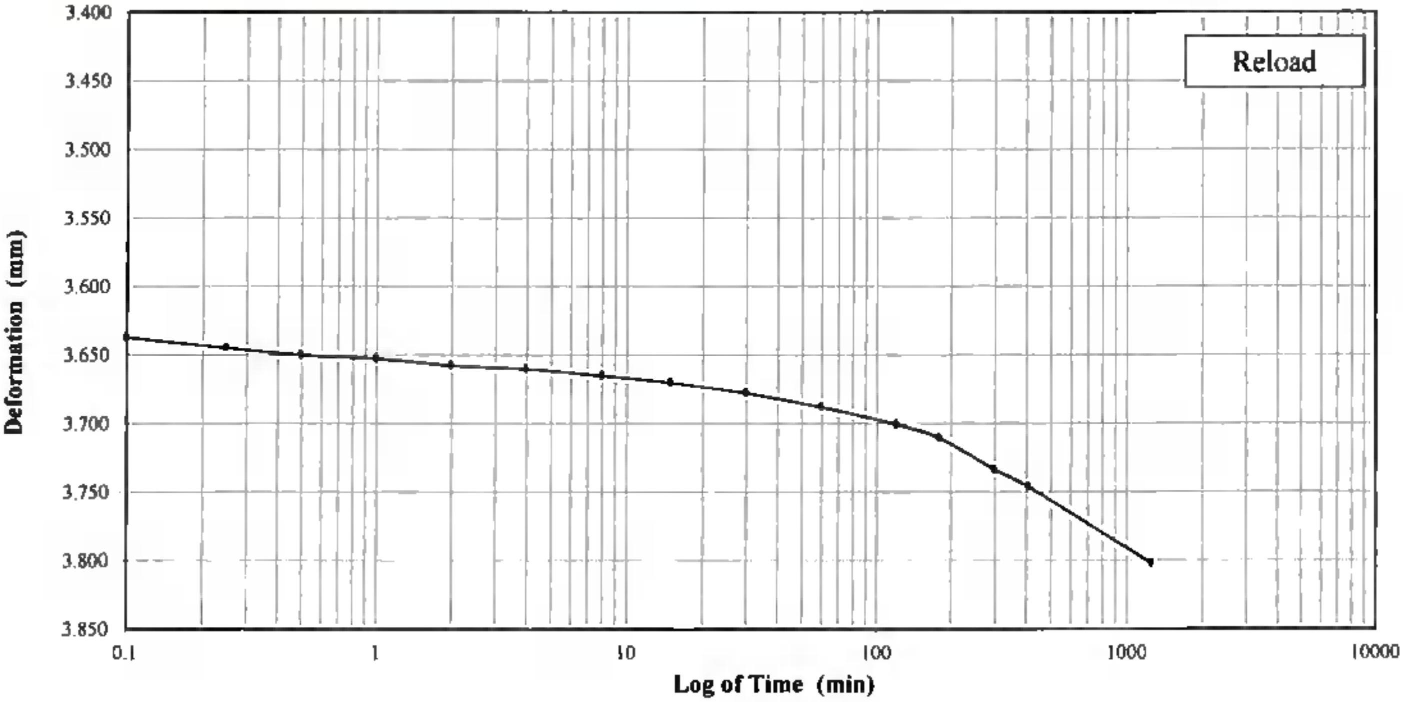


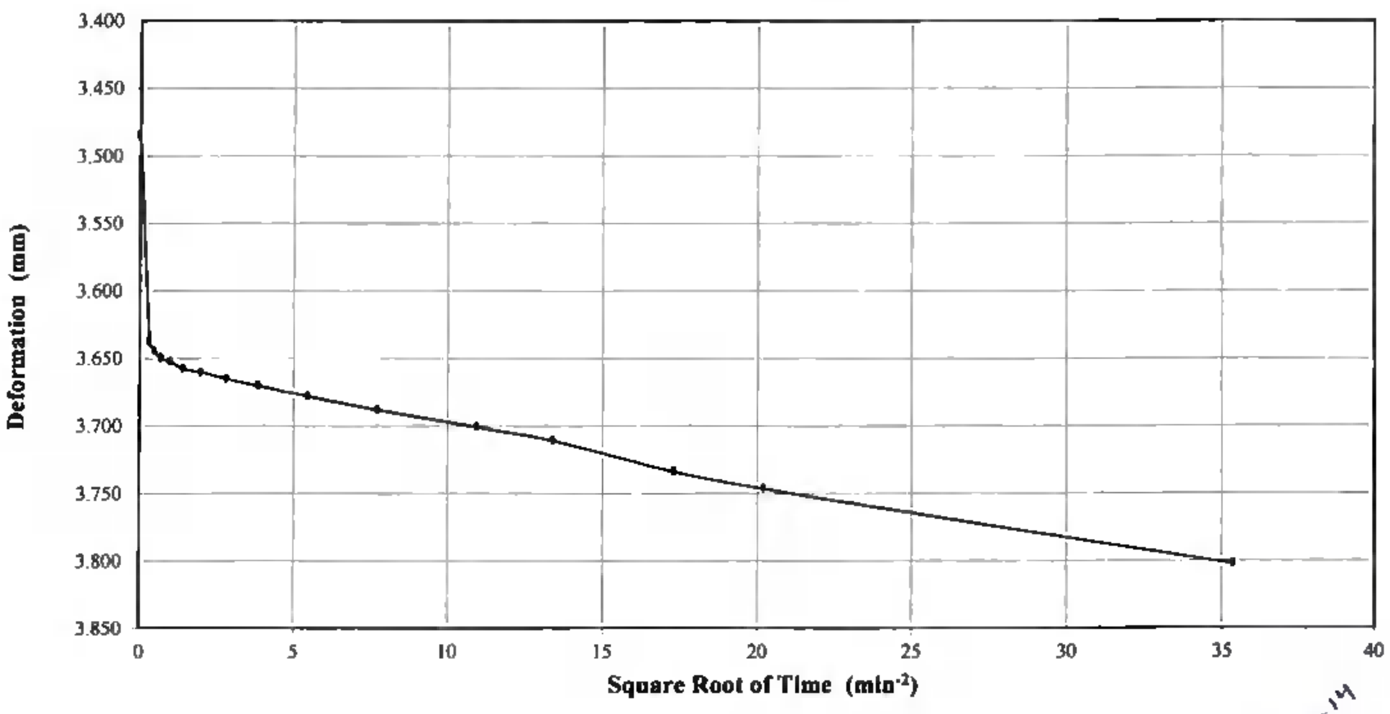
2-4-14
NSR

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-107B, S-03 (16.0-18.0')
Lab Sample No: 13K155

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 7 - 4000 psf

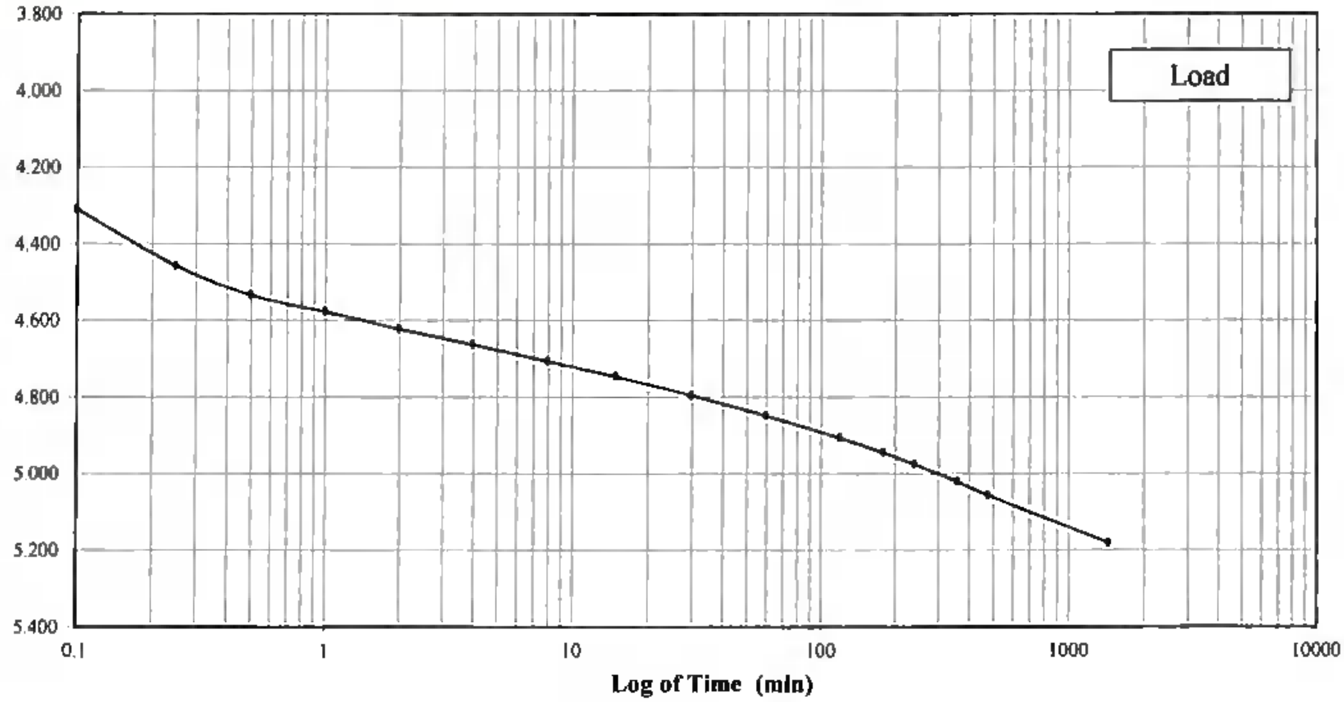


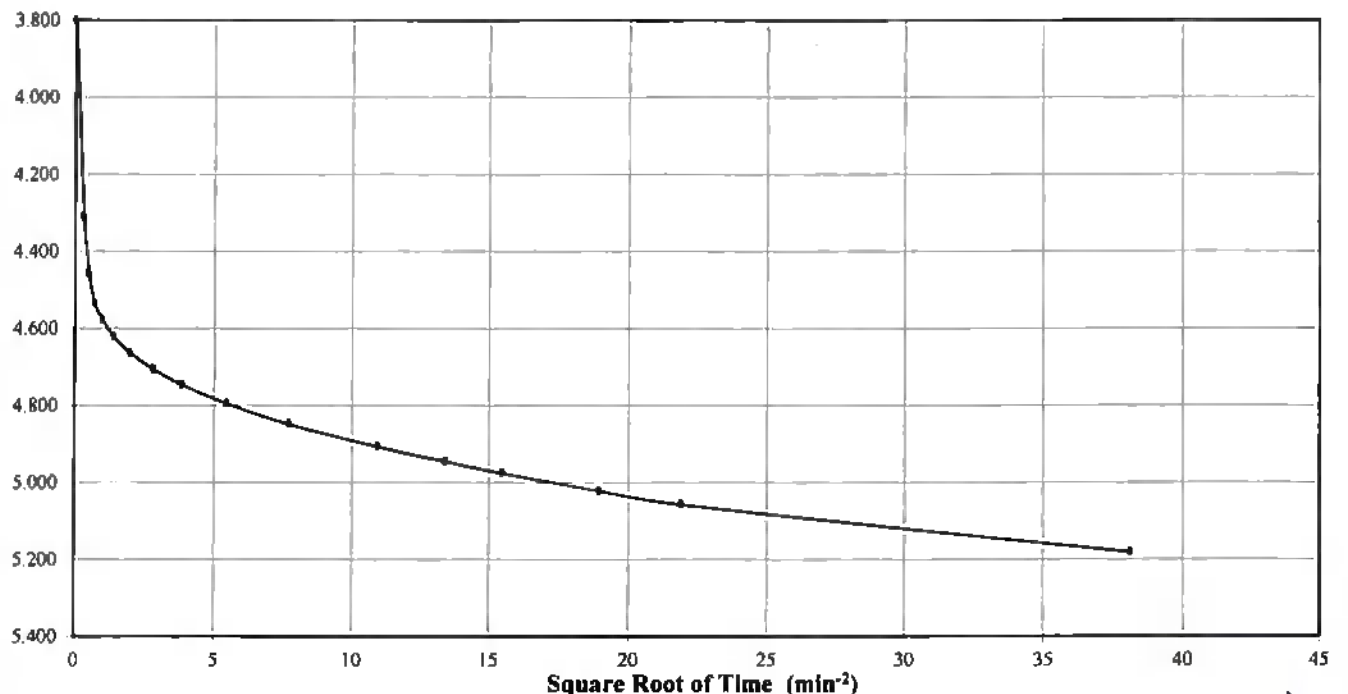


2-11-14
NSR

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-107B, S-03 (16.0-18.0') Lab Sample No: 13K155

ASTM D 2435 **ONE-DIMENSIONAL CONSOLIDATION TEST** Figure 8 - 8000 psf





2-4-14
NJR

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

FLEXIBLE WALL PERMEABILITY TEST [1]

ASTM D 5084 *

Project Name:	Winyah Generating Station
Project Number:	618
Client Name:	Geosyntec Consultants
Site Sample ID:	SPT-106, ST-01 (60.0-62.0')
Lab Sample Number:	13J353
Material Type:	Soil
Specified Value (cm/sec):	NA
Date Test Started:	12/5/2013

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	5.66	7.34	75.7	40.5	98.0	70.0	28.0	DTW	6	4.5E-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
NJK

ATTACHMENT 4-B

S&ME Laboratory Testing Results
(S&ME, 1978)

SOIL DATA SUMMARY

SBME JOB NO. SS7735

BORING NUMBER	SAMPLE DEPTH	CLASSIFICATION	STANDARD PENETRATION RESISTANCE	NATURAL MOISTURE (%)	% FINER # 200	UNIT WEIGHT P.C.F. W	UNIT WEIGHT P.C.F. D	PROCTOR DATA MAX	PROCTOR DATA OMC	SPECIFIC GRAVITY	VOID RATIO E.	UNCOFINED COMP. MAX.	ATTERBERG LIMIT LL	ATTERBERG LIMIT PI	TRIAXIAL SHEAR C	TRIAXIAL SHEAR ϕ	CONSOLIDATION C_C	OTHER
SC30	2-3'	Gray Clayey SAND		27.5	40.1								30	13				
SC31	2-4'	Gray Clayey SAND		19.7	25.2								27	9				
SC34	1.5-3'	Tan Clayey SAND		16.7 / 24	33.6	113.2	97.0	110.5	15.6		.738		31	11	1750 / 150	21° / 36.9°		
SC34	7-9'	Gray Silty SAND			11.1			106.0	15.0					NP				
SC36	2-3'	Gray Sandy CLAY		32.7	62.5								39	17				
SC36	3-4'	Gray Clayey SAND		17.6 / 22.3	37.9	123.8	105.3	113.1	14.5		.838		30	12	1150 / 170	23.5° / 40°		
SC36	6-7'	Gray SAND			1.2									NP				
SC15	9.5-11.5 / 17-19														1200	16°	(UU)	
SC38	2-3'	Gray Sandy CLAY		33.8	52.4								43	22				
SC39	1.5-3'	Gray Sandy CLAY		28.8	75.2								60	38				
SC39	6-7'	Gray SILT			57.9								33	10				
SC49	1-2'	Gray Sandy CLAY		28.0	44.7								42	18				
SC50	1-2'	Gray Sandy CLAY		26.9	49.5								36	17				

* A detailed description is presented on attached Test Pit Logs.

SOIL & MATERIAL ENGINEERS, INC.

SOIL DATA SUMMARY

SBME JOB NO. SS7735

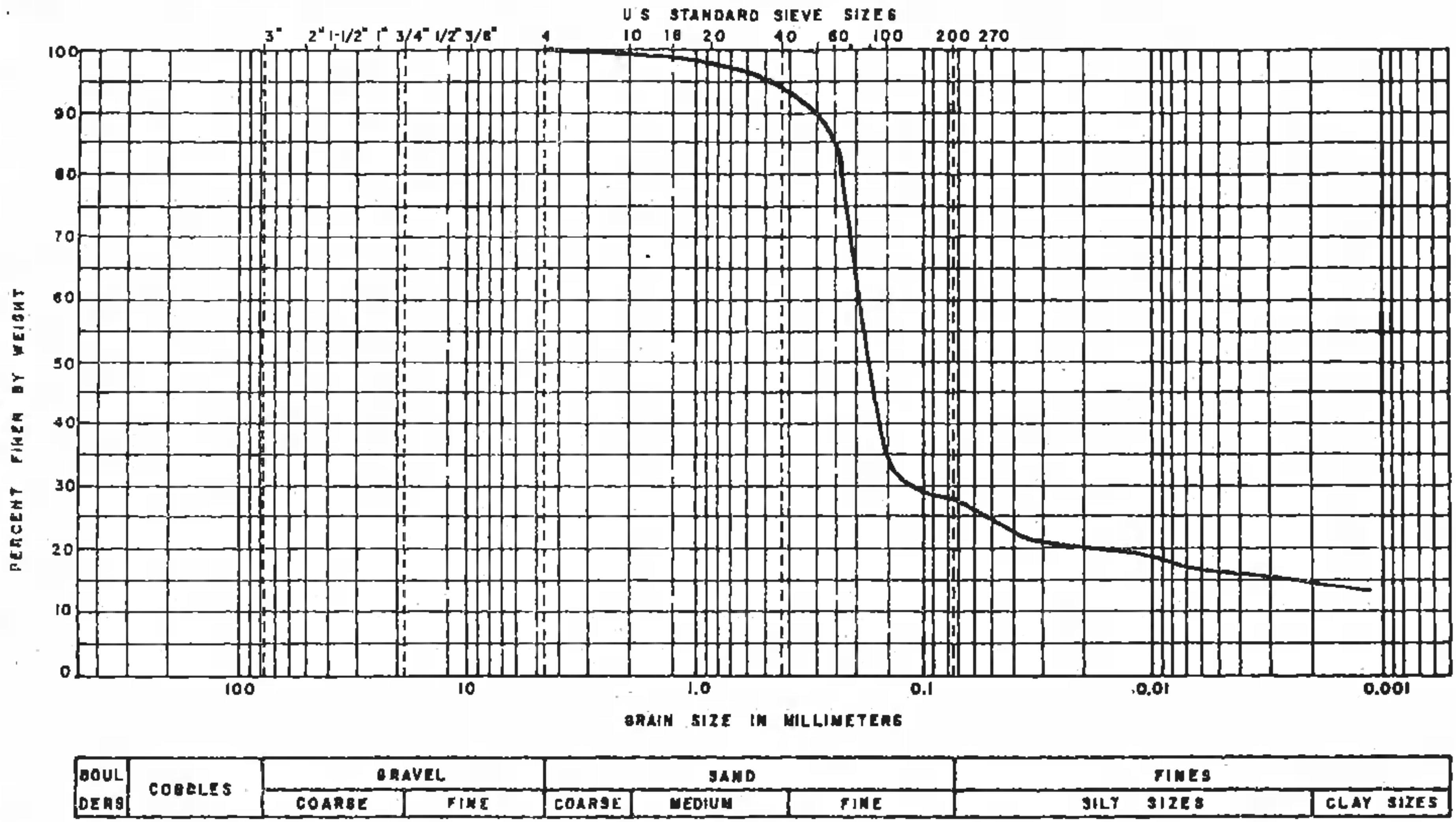
BORING NUMBER	SAMPLE DEPTH	CLASSIFICATION	STANDARD PENETRATION RESISTANCE	NATURAL MOISTURE (%)	% FINER # 200	UNIT WEIGHT P.C.F. W	UNIT WEIGHT P.C.F. D	PROCTOR DATA MAX	PROCTOR DATA OMC	SPECIFIC GRAVITY	VOID RATIO E_o	UNCOFINED COMP. MAX.	ATTERBERG LIMIT LL	ATTERBERG LIMIT PI	TRIAXIAL SHEAR C	TRIAXIAL SHEAR φ	CONSOLIDATION C_c	OTHER
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.

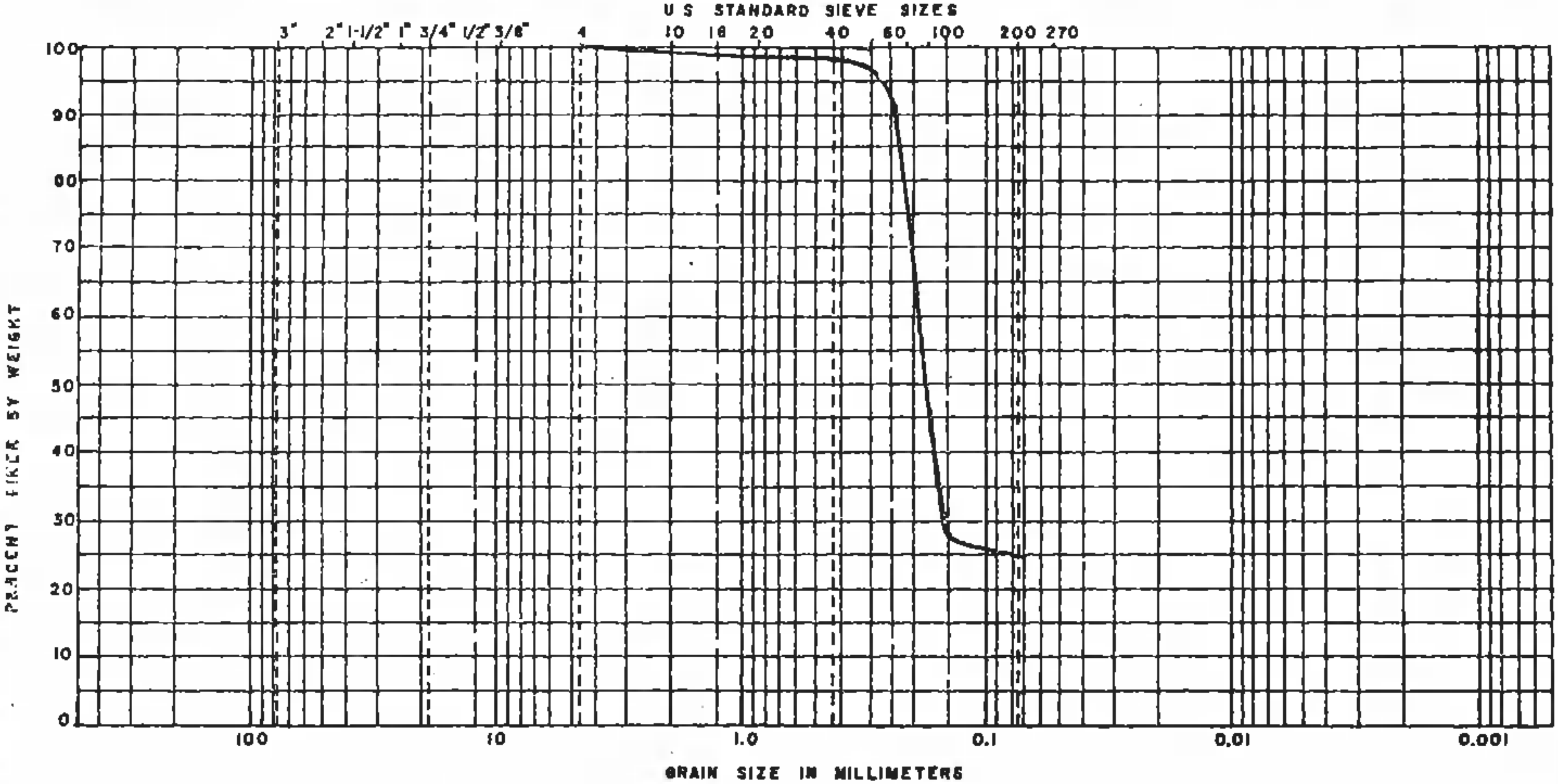


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-25	3.0-4.0	7.6	34.0	17.2	16.8	Red Gray Tan Silty Clayey Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-25	6.0-8.0		Non-Plastic			White Tan Silty Fine to Medium SAND

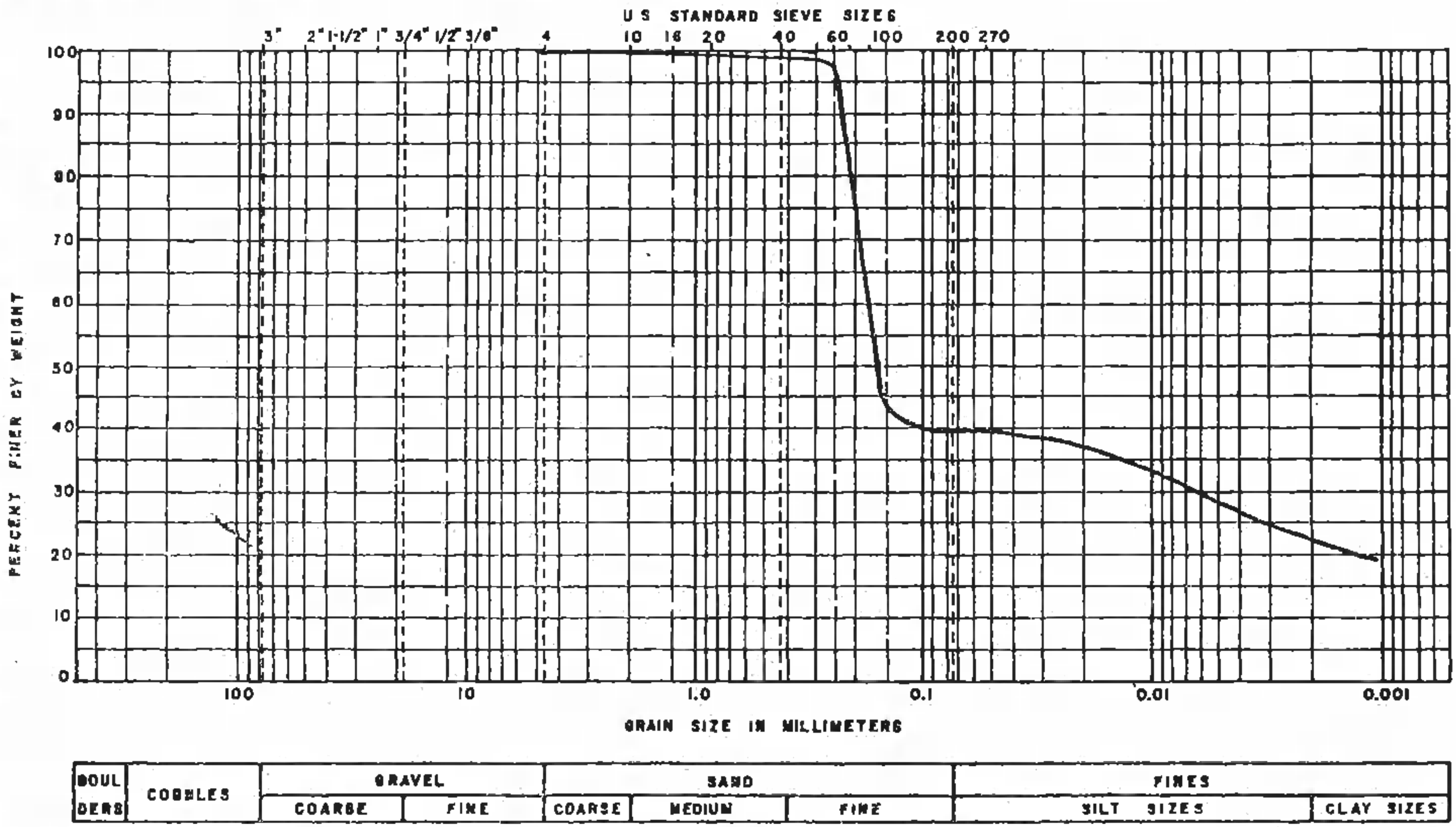
GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

SM-19

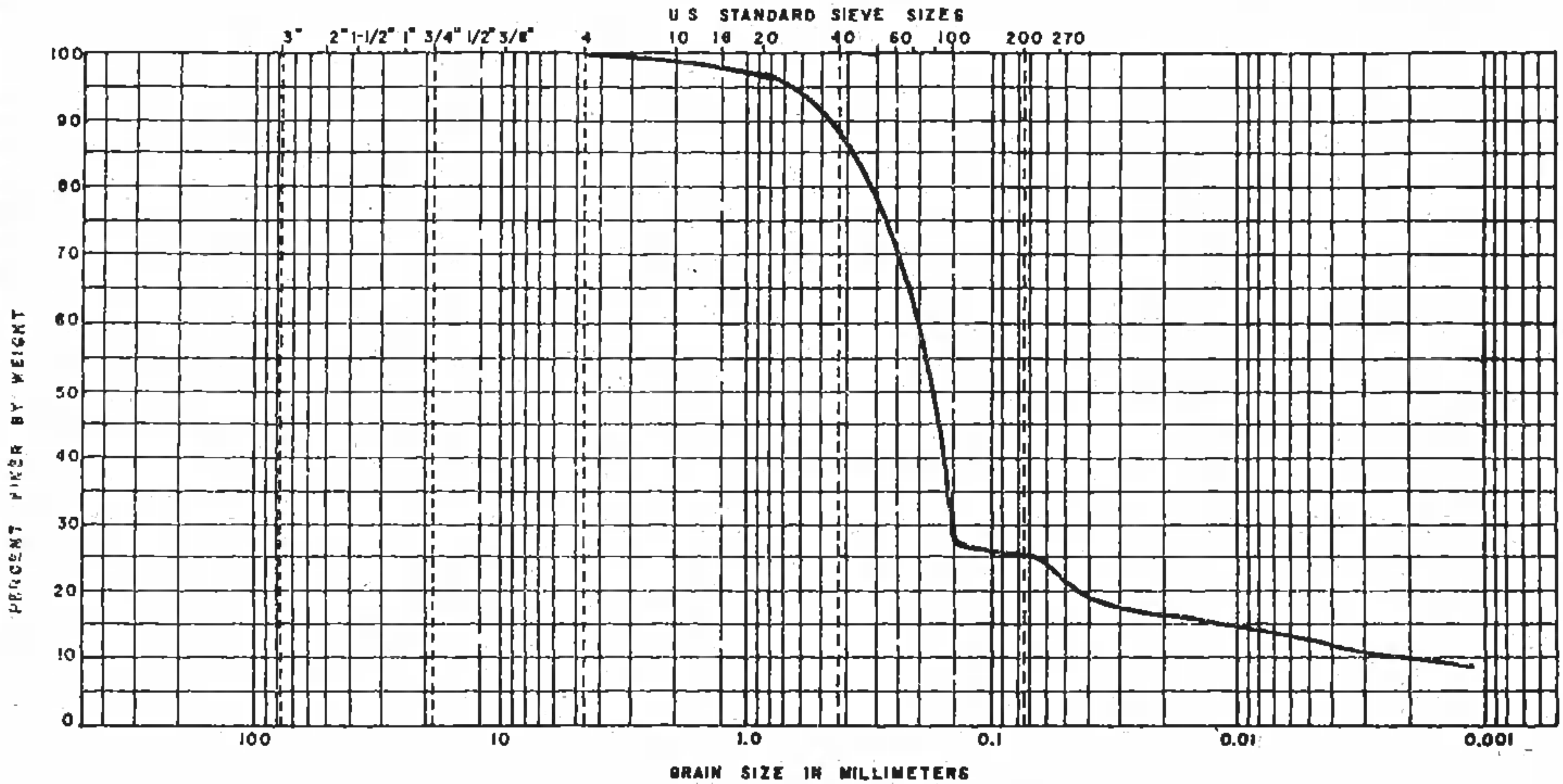
GRAIN SIZE DISTRIBUTION



BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-30	2.0-3.0	27.5	29.9	17.2	12.7	Red Orange Gray Silty Clayey Fine to Medium SAND

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.



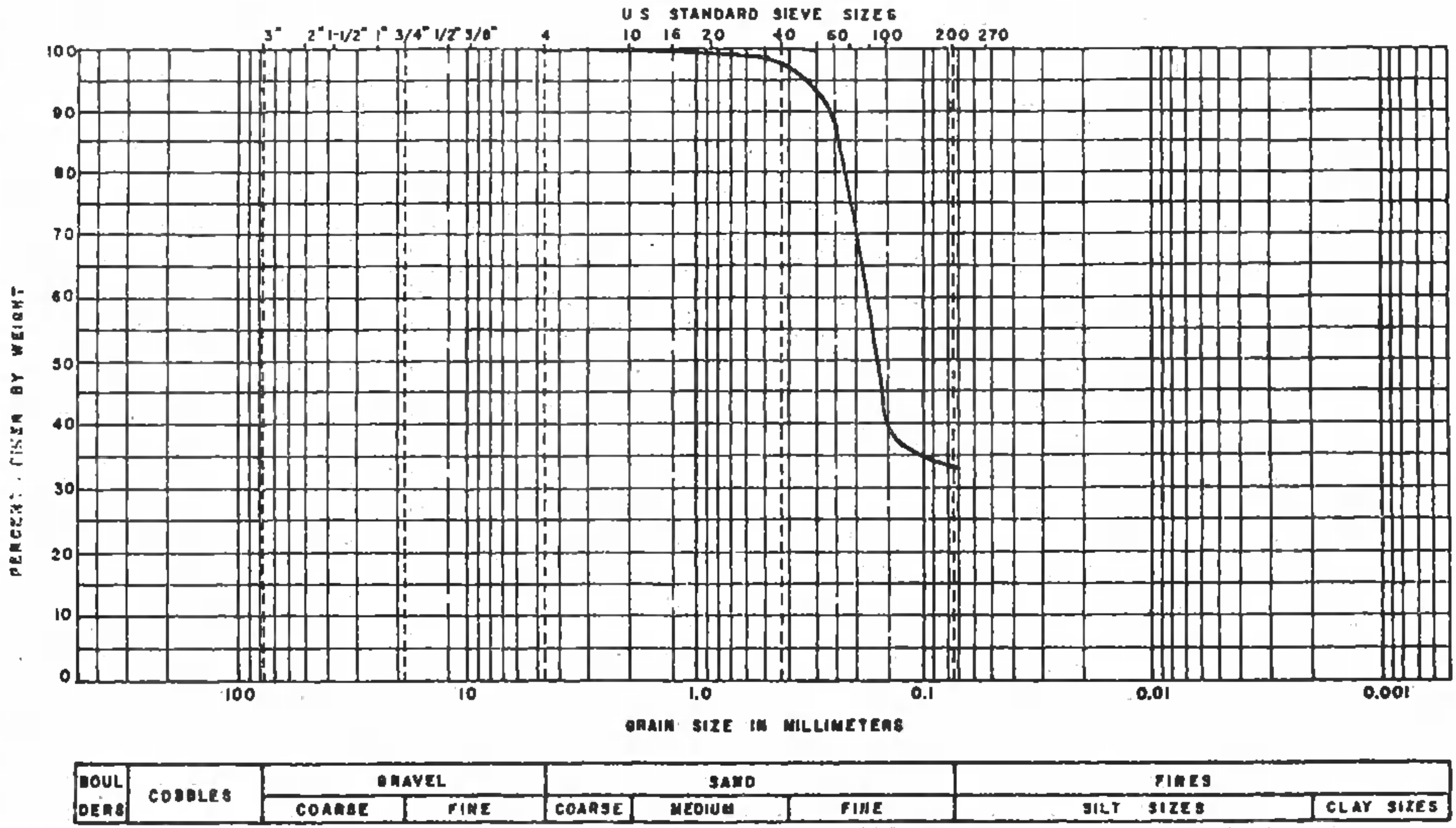
BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-31	2.0-4.0	19.7	27.4	18.4	9.0	Orange Red Gray Slightly Silty Clayey Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.



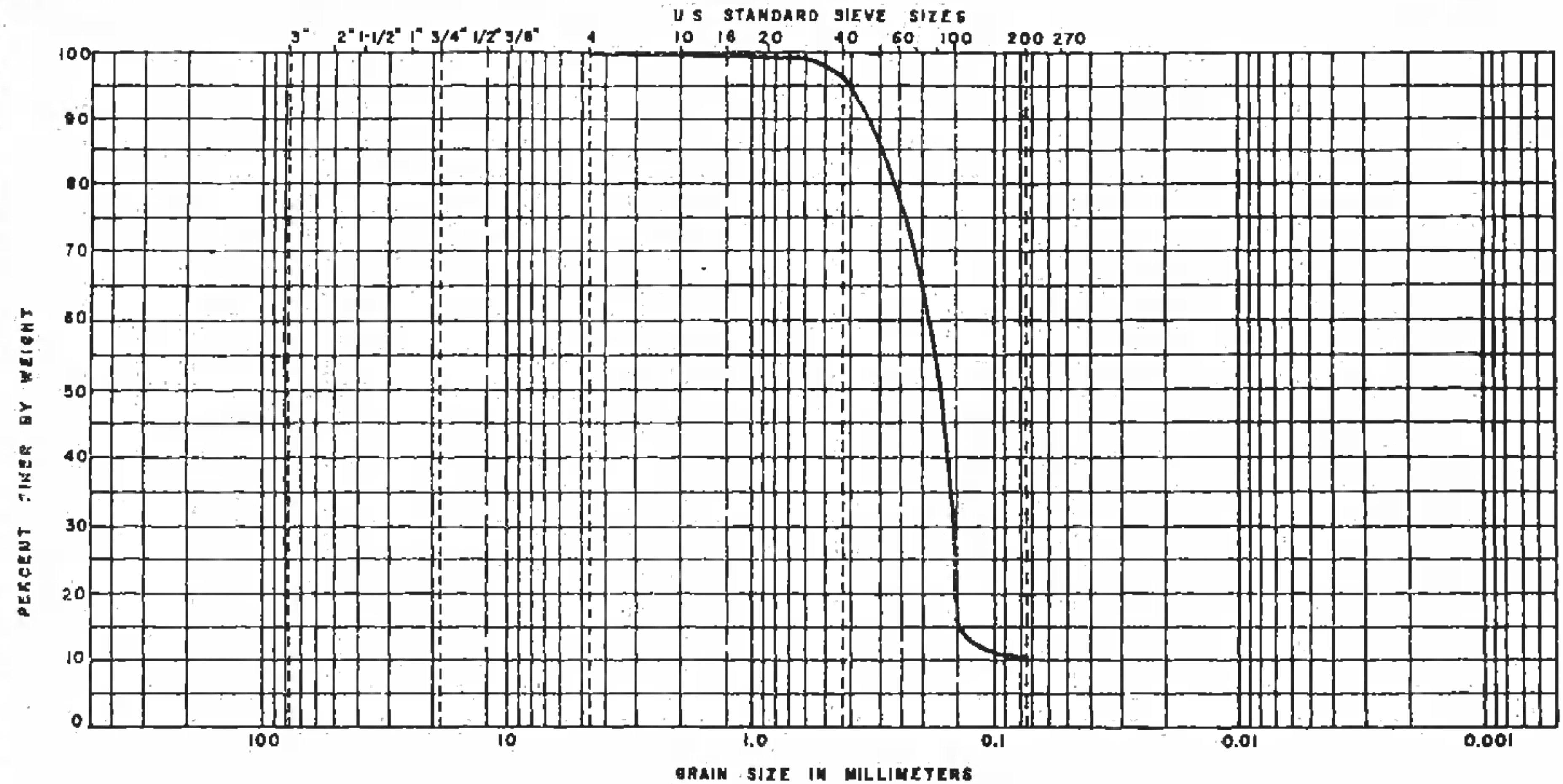
BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-34	1.5-3.0	24.0	31.3	20.4	10.4	Tan Clayey Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

SM-19



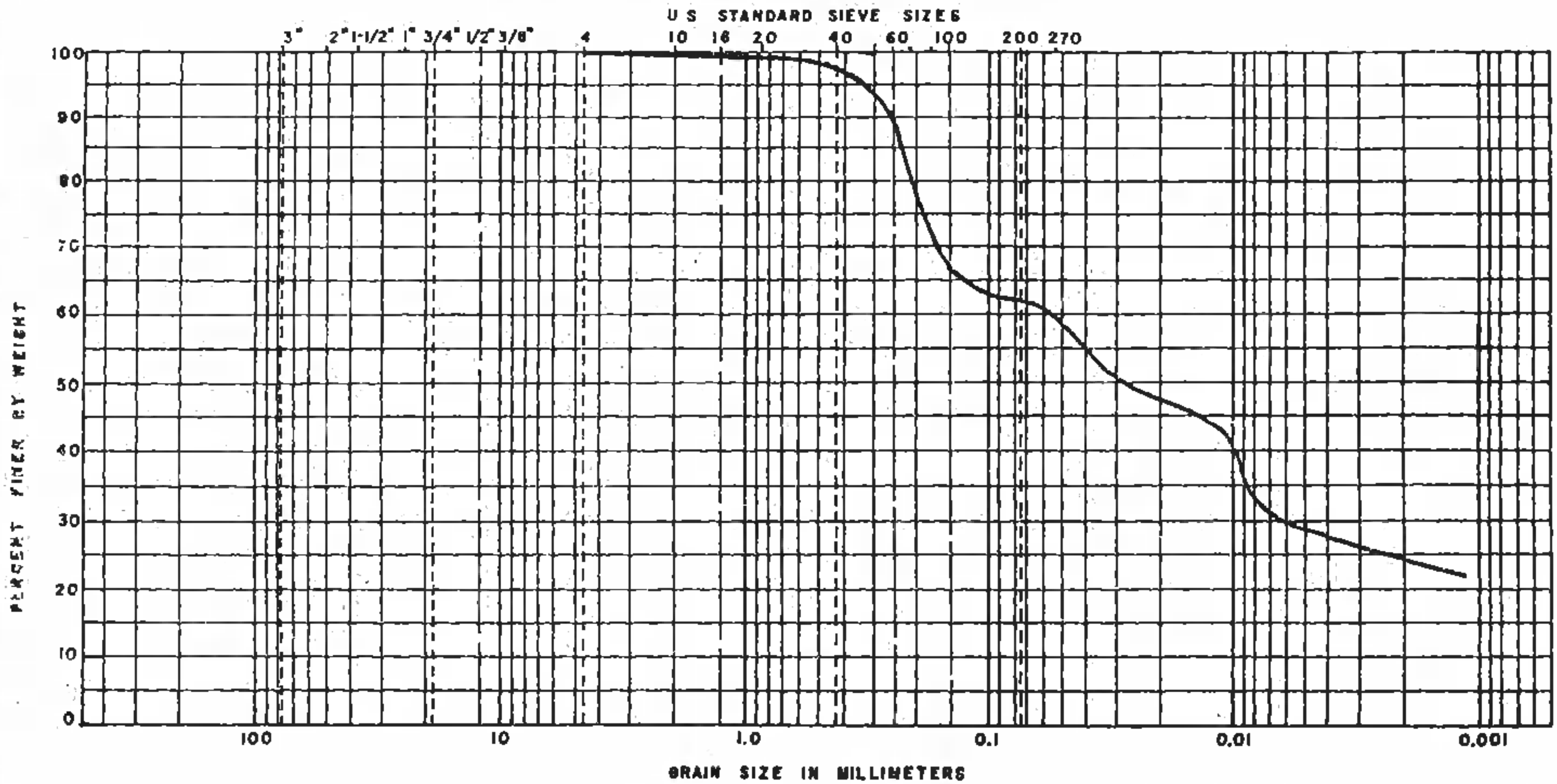
BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-34	7.0-9.0	22.0	Non-Plastic			Gray Tan Silty Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.



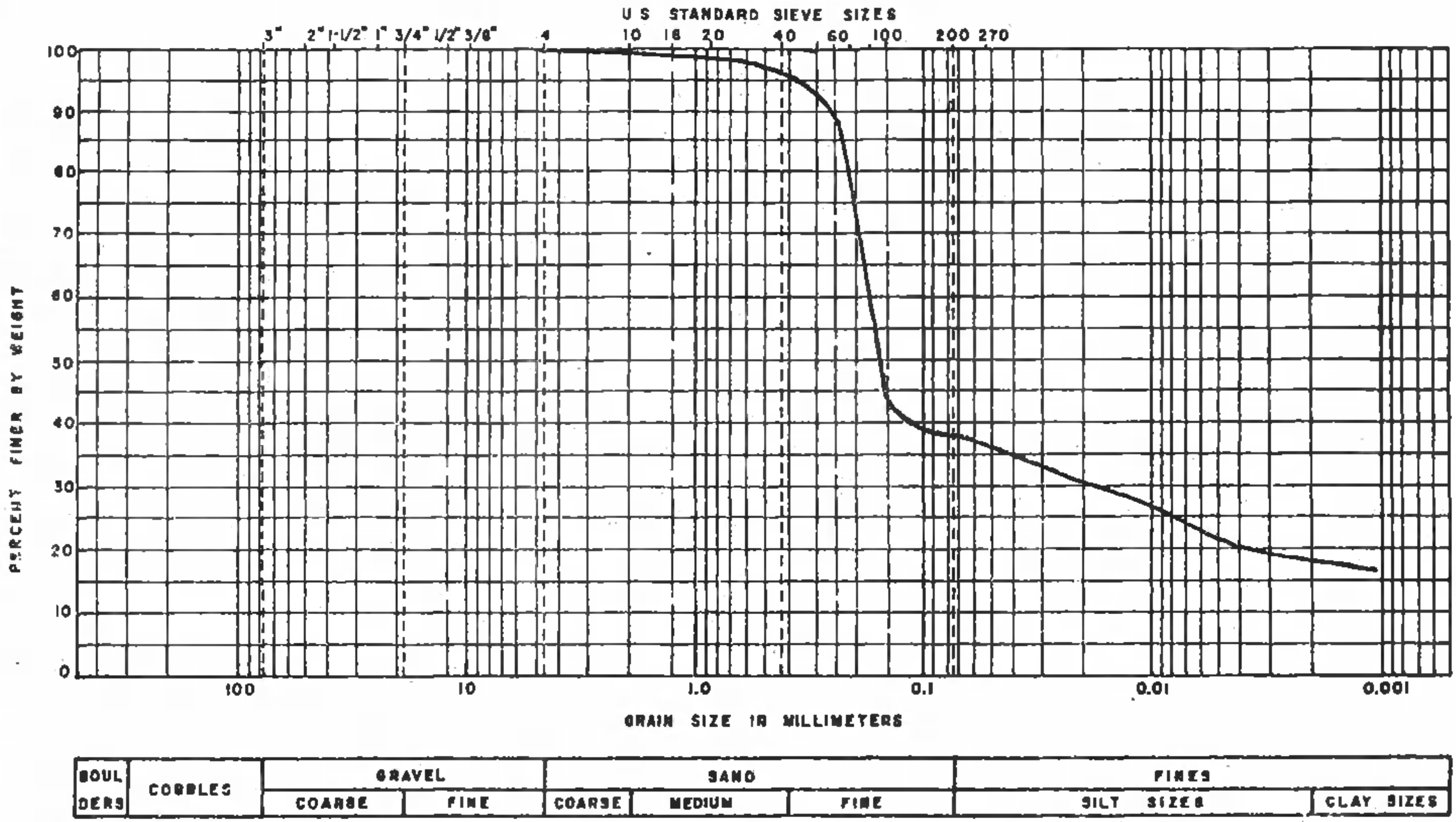
BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC 36	2.0-3.0	32.7	39.4	22.5	16.9	Tan Red Gray Slightly Sandy Silty CLAY

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

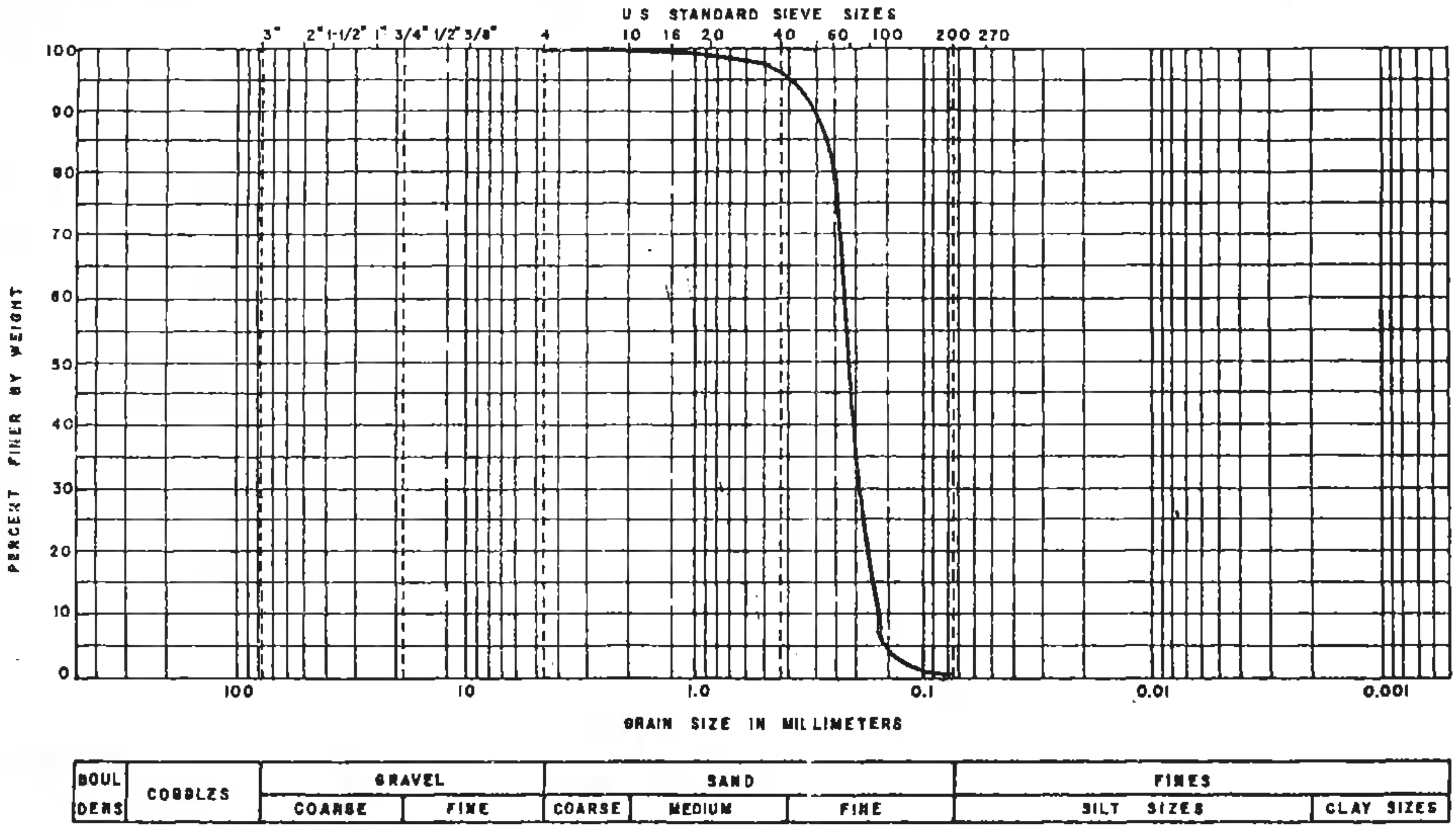


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-36	3.0-4.0	22.3	30.2	17.9	12.3	Tan Red Gray Clayey Silty Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC

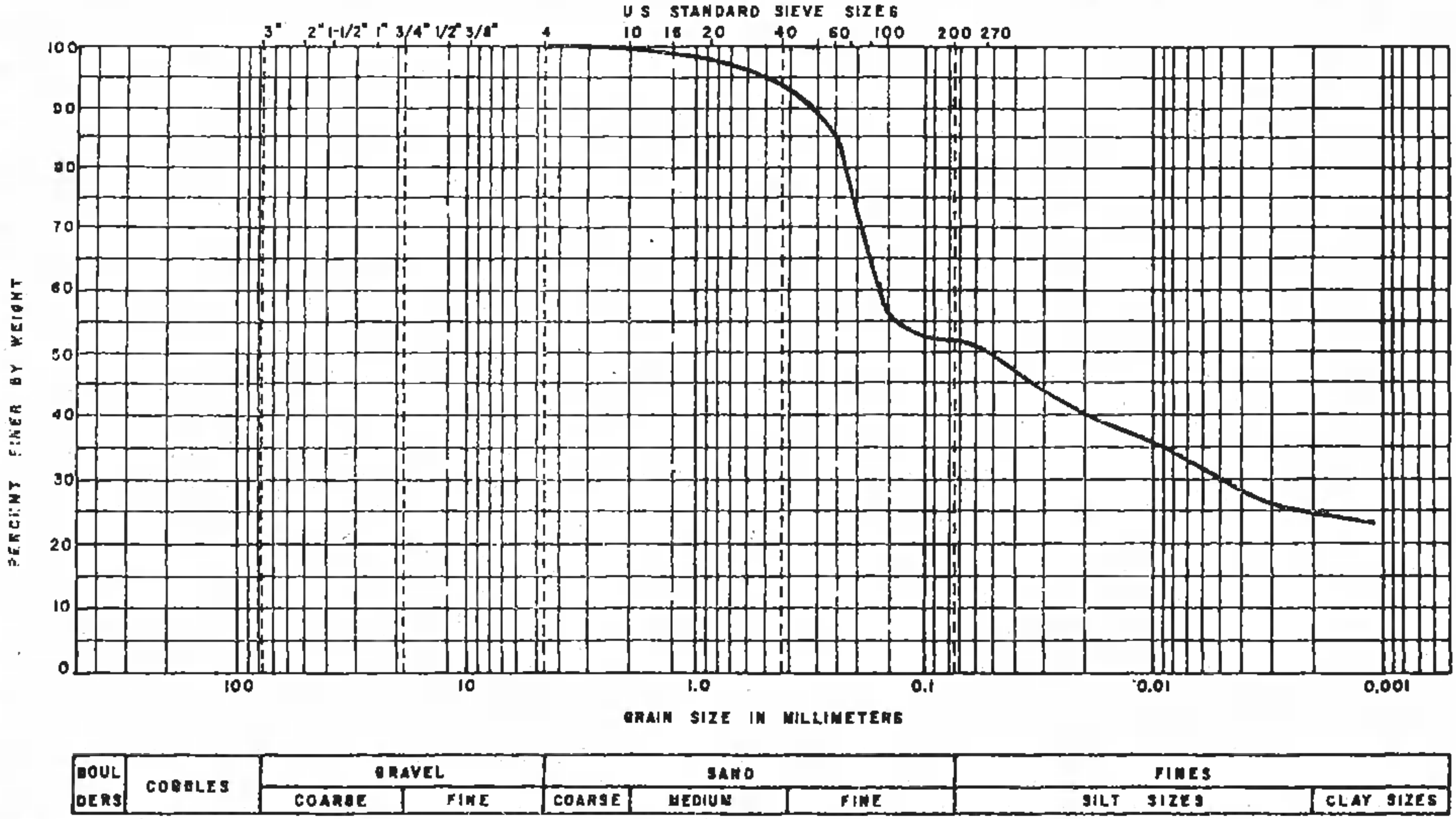


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-36	6.0-7.0	23.5	Non-Plastic			Tan Gray Slightly Silty Fine to Medium SAND

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

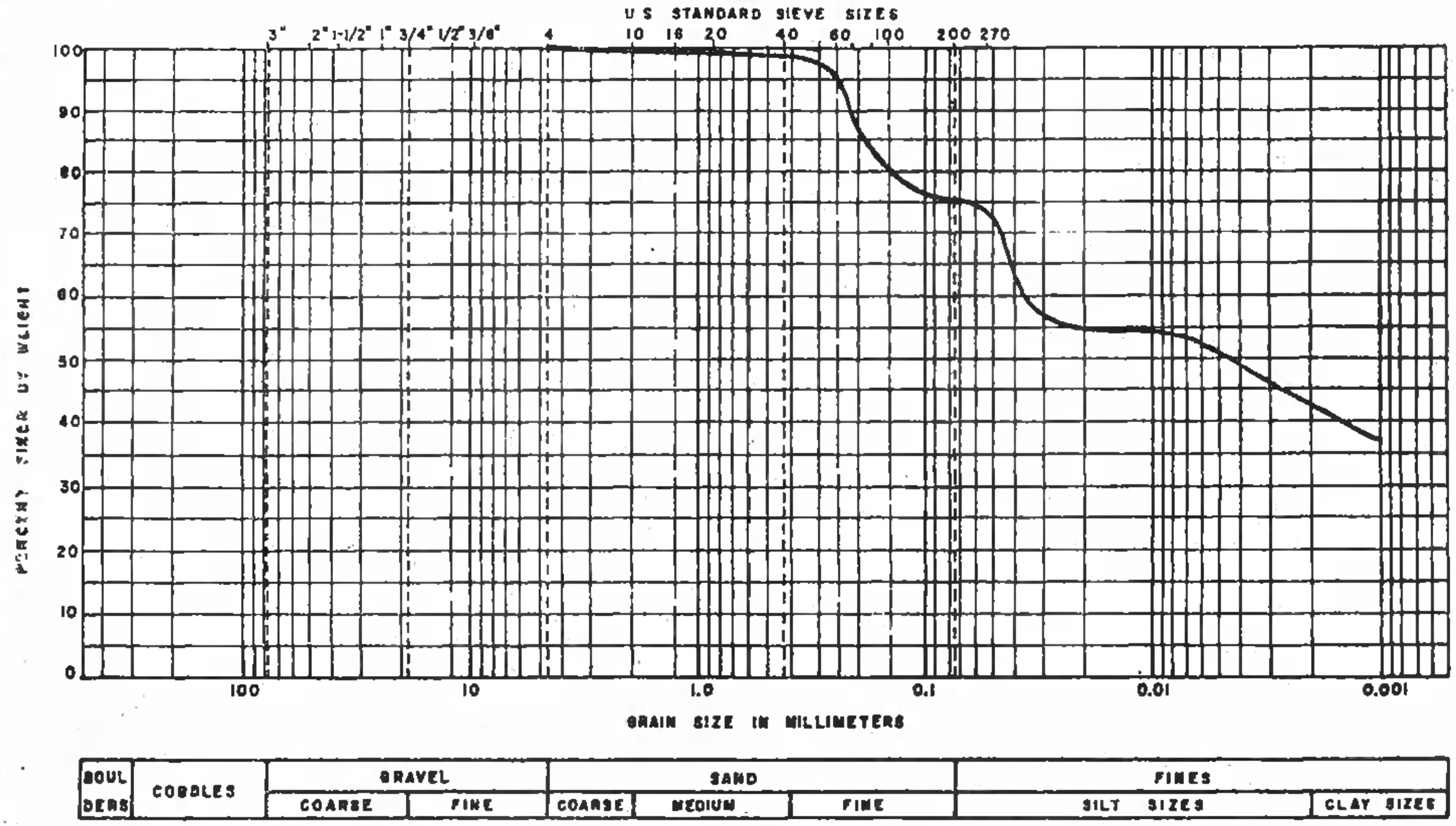


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-38	2.0-3.0	33.8	43.4	21.9	21.5	Orange Gray Slightly Sandy Silty CLAY

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

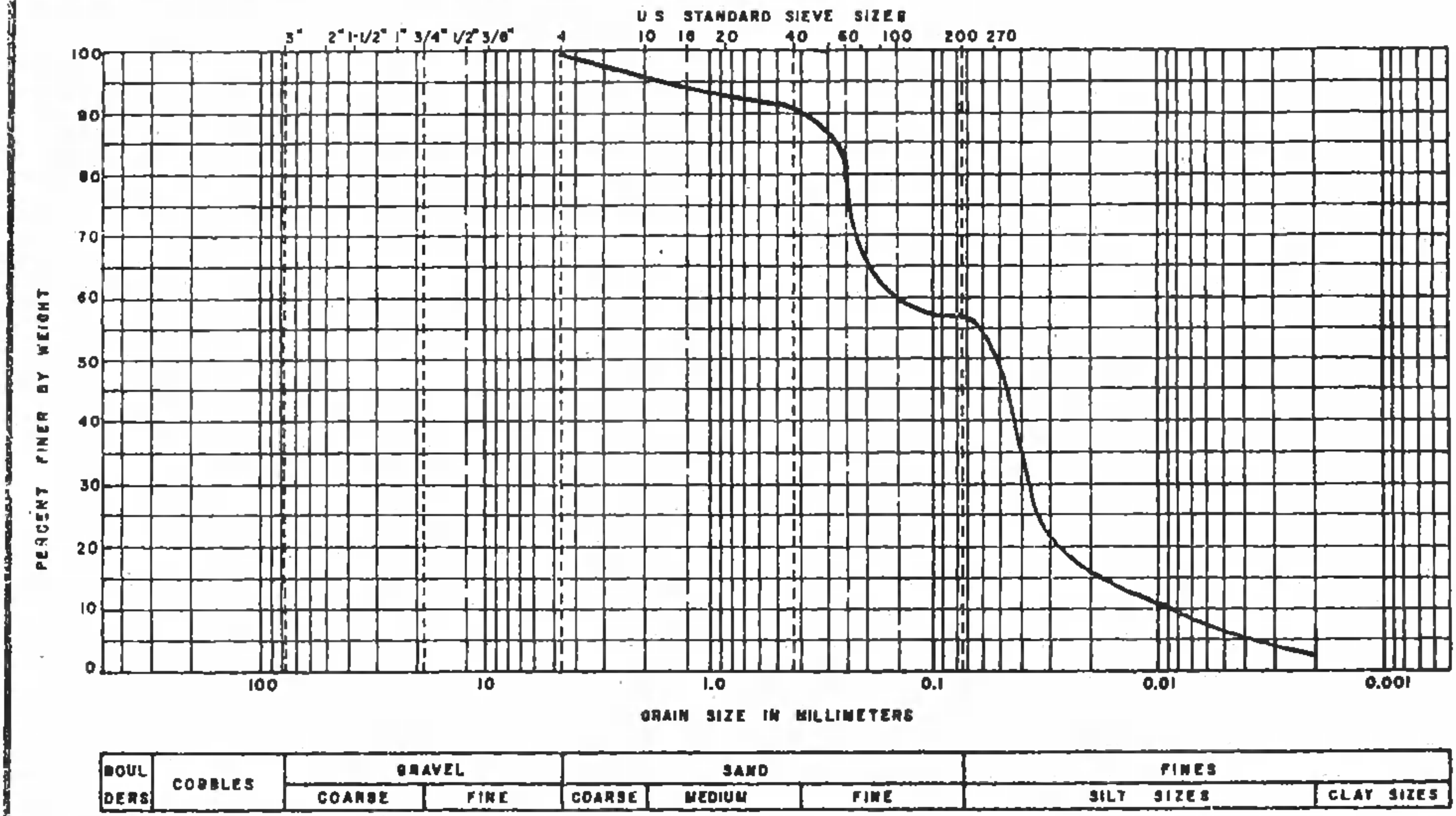


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-39	1.5-2.0	28.8	60.1	21.8	38.3	Orange Gray Slightly Sandy Silty CLAY

GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.

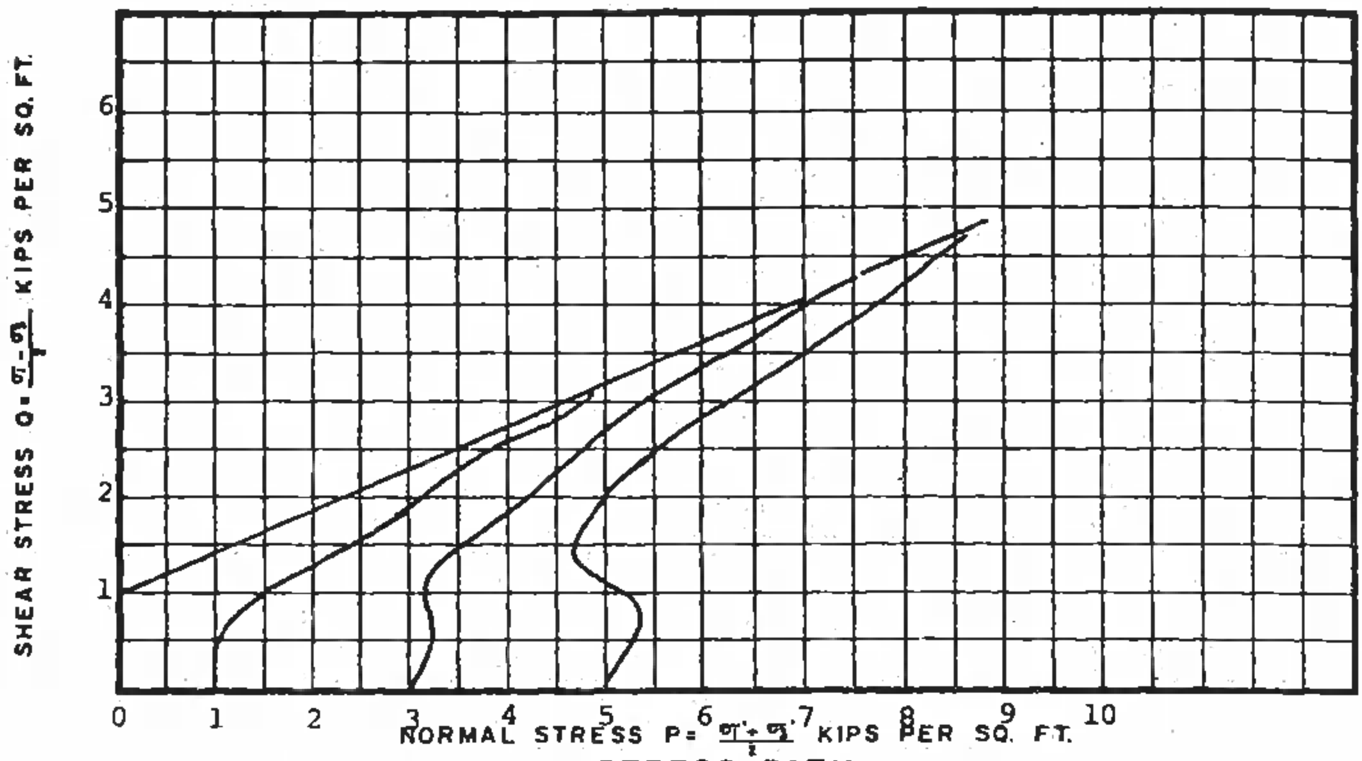


BORING NO	ELEV. OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
SC-39	6.0-7.0	79.2	33.4	23.9	9.5	Gray Green Silty Clayey Medium SAND

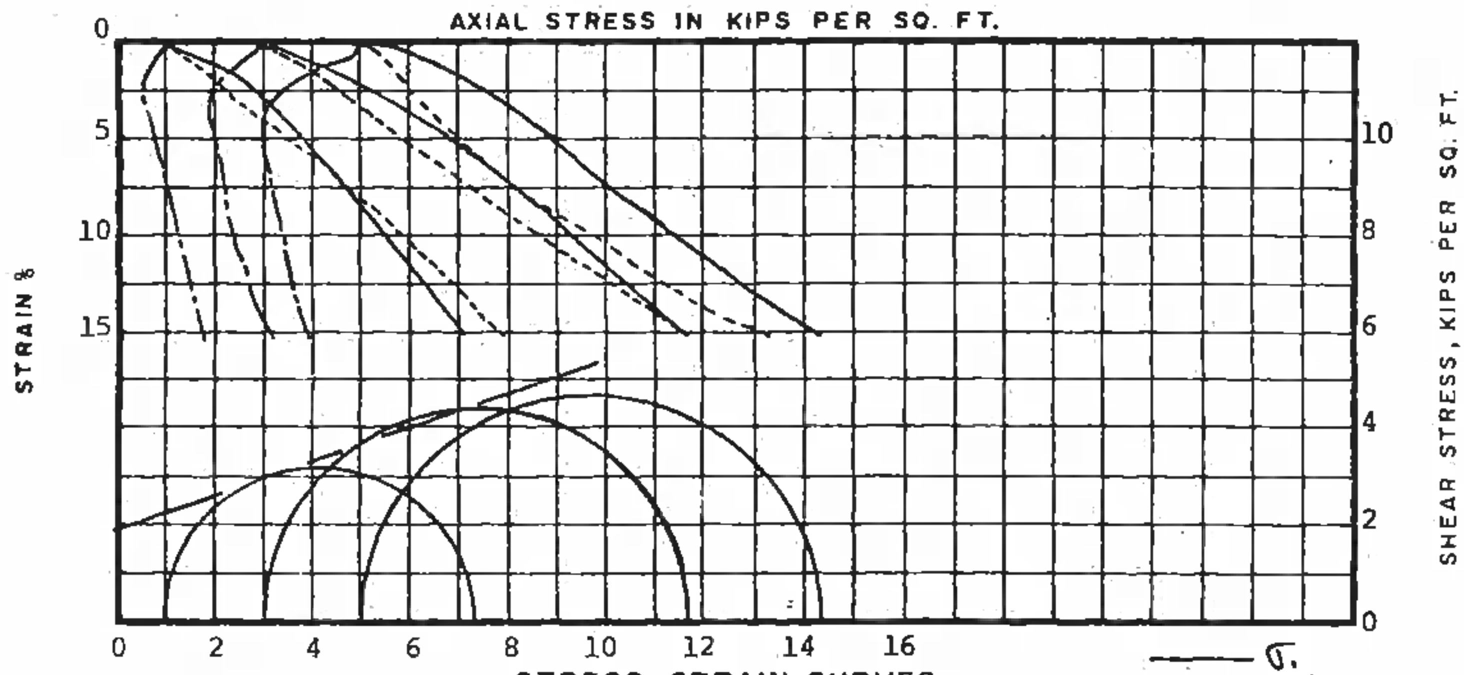
GRAIN SIZE DISTRIBUTION

JOB NO. SS7735

SOIL & MATERIAL ENGINEERS, INC.



STRESS PATH



STRESS–STRAIN CURVES

MOHR DIAGRAMS

"COHESION", c 2000 psf c' 1110 psf

ANGLE OF SHEAR RESISTANCE ϕ =19°, ϕ' =25.8°

UNIT WEIGHT, γ, 105.3, 105.4, 105.6 pcf

WATER CONTENT w 17.8, 17.6, 17.4 %

VOID RATIO, e .601, .559, .596

SATURATED
CONSOLIDATED UNDRAINED

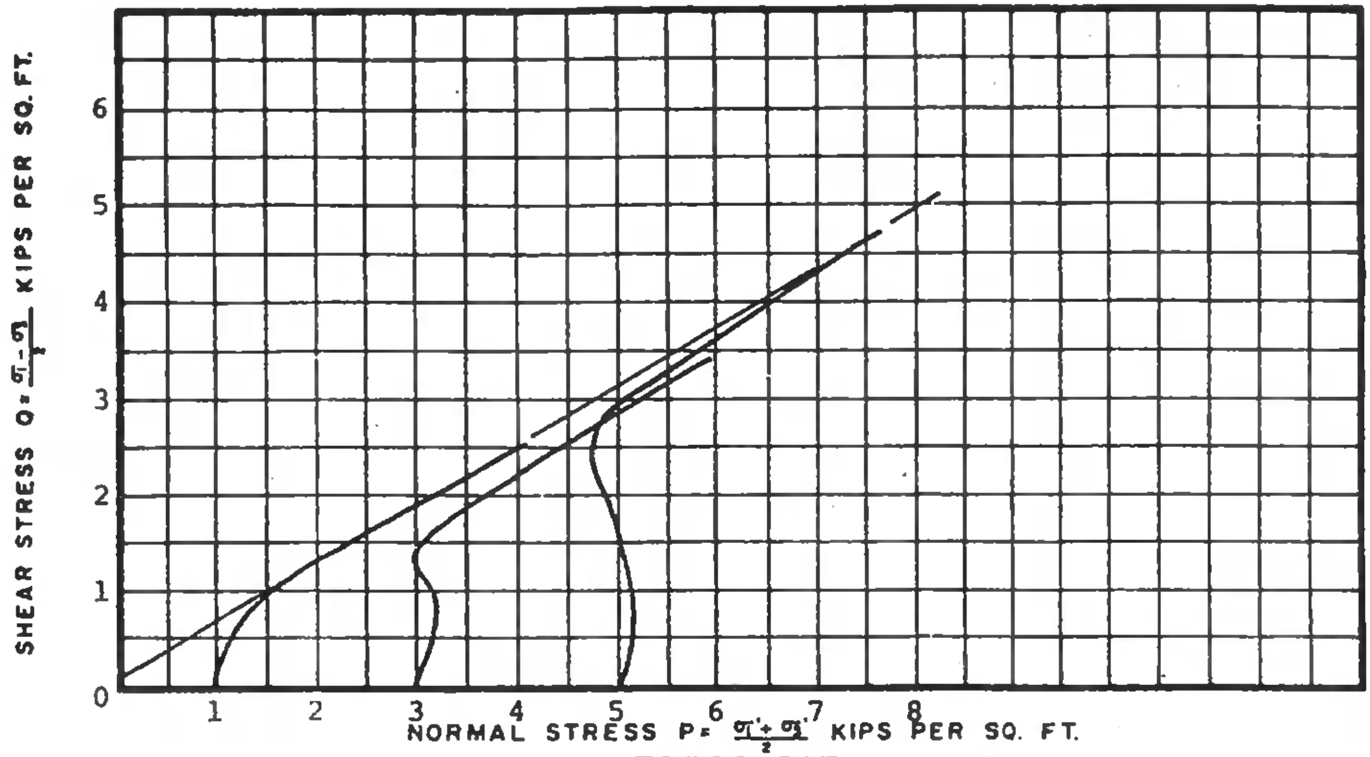
TRIAXIAL SHEAR TEST

PORE PRESSURE MEASUREMENTS

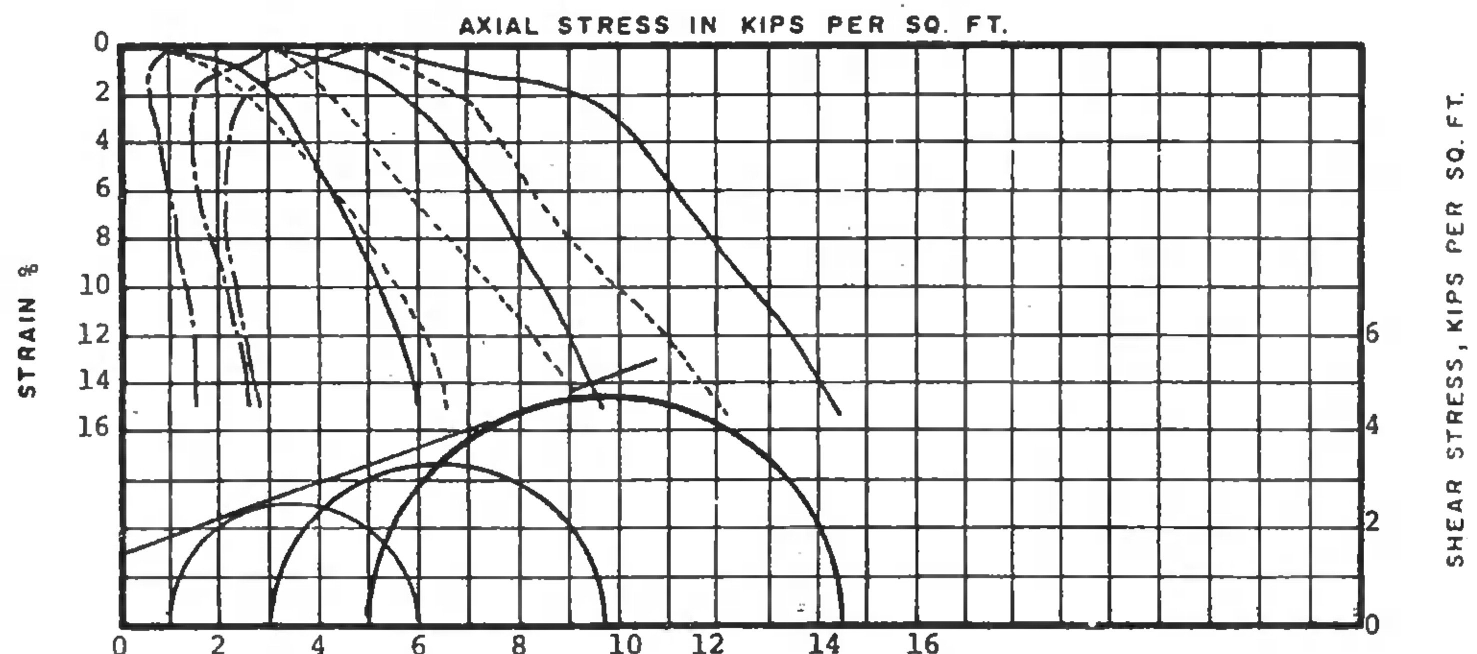
BORING NO. SC-25 SAMPLE NO. BAG

ELEV. OR DEPTH 3-4' JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.



STRESS PATH



STRESS-STRAIN CURVES

MOHR DIAGRAMS

"COHESION", c 1750 psf c' 150 psf

ANGLE OF SHEAR RESISTANCE. ϕ =21°, ϕ'= 36.9°

UNIT WEIGHT, γ_d 95.2, 100.6, 95.2 pcf

WATER CONTENT w 16.4, 17.2, 16.4 %

VOID RATIO. e .7695, .6743, .7695

SATURATED
CONSOLIDATED UNDRAINED

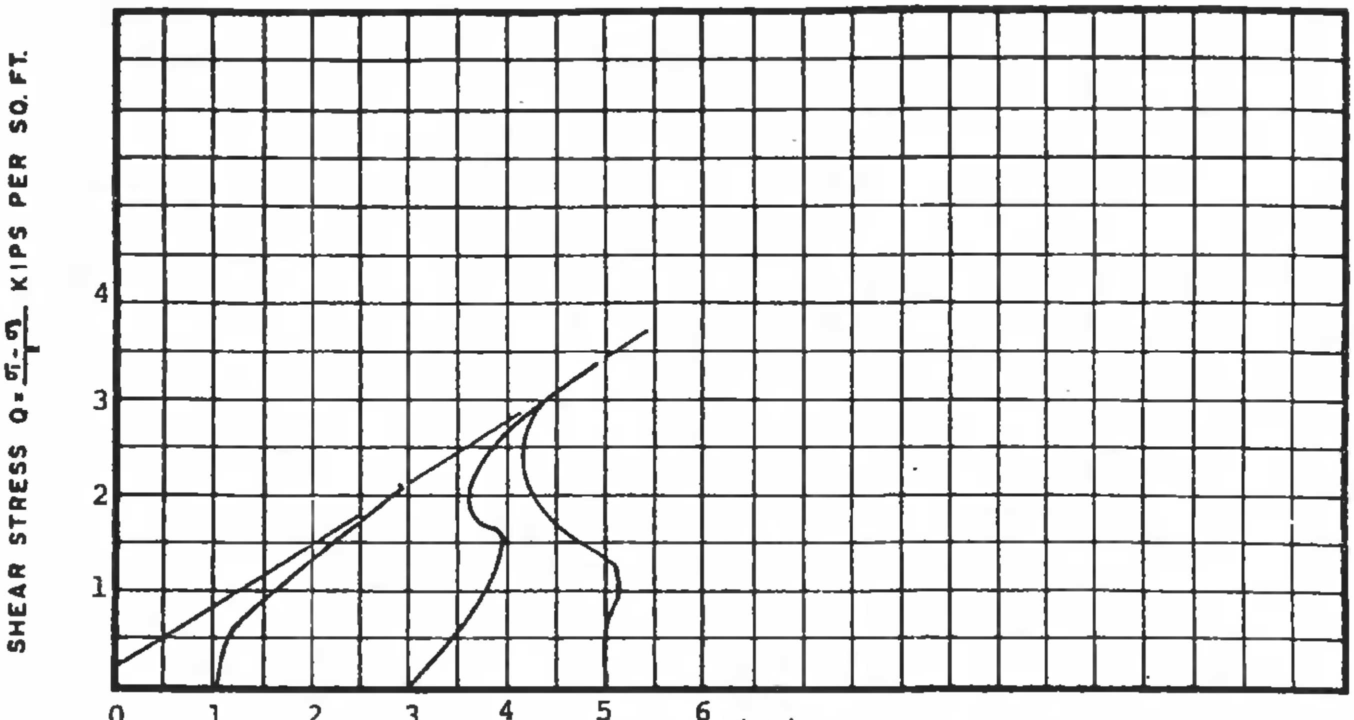
TRIAXIAL SHEAR TEST

PORE PRESSURE MEASUREMENTS

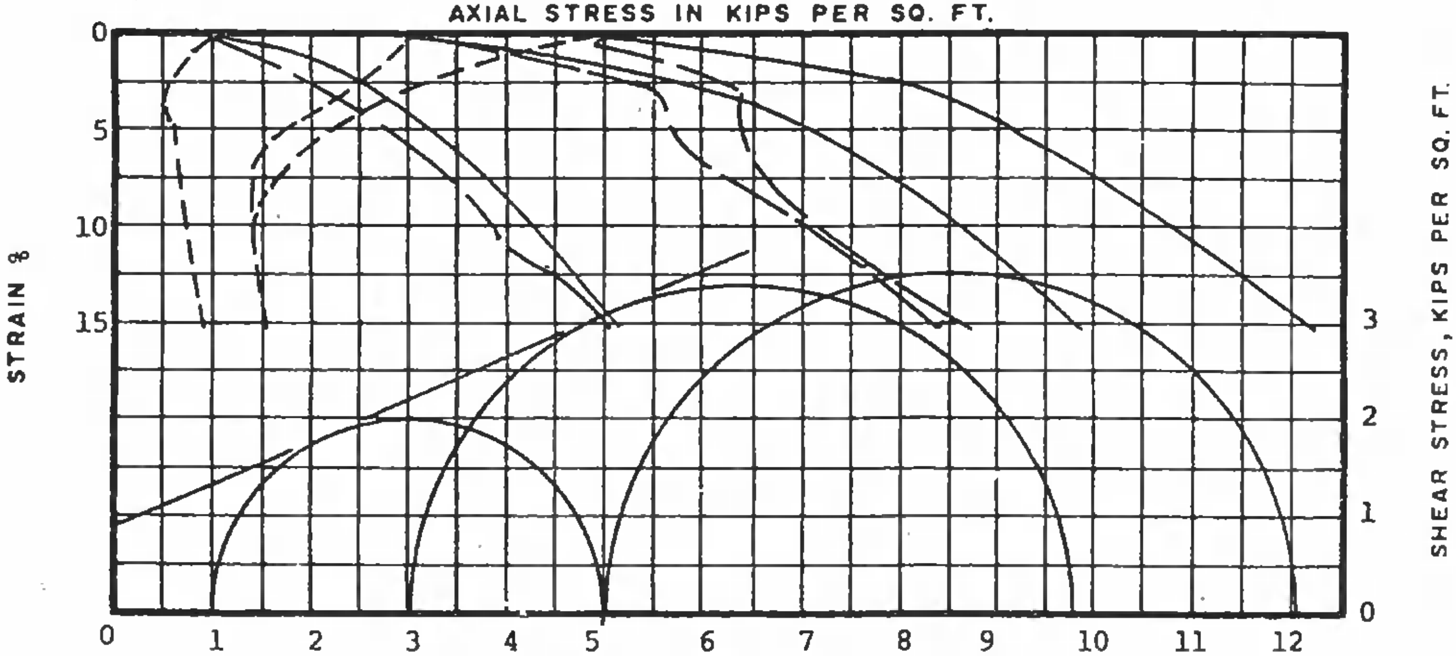
BORING NO. SC-34 SAMPLE NO. Baq

ELEV. OR DEPTH 1.5-3.0 JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.



STRESS PATH



STRESS-STRAIN CURVES

MOHR DIAGRAMS

———— σ_1

- ———- σ_1'

— — — —- σ_3'

"COHESION", c 1150 pcf c' 170 psf

ANGLE OF SHEAR RESISTANCE. ϕ =23.5° ϕ' =40°

UNIT WEIGHT, γ_t 105.3, 105.3, 105.3 pcf

WATER CONTENT. w 17.8, 17.6, 17.4 %

VOID RATIO. e .841, .837, .836

SATURATED
CONSOLIDATED UNDRAINED

TRIAXIAL SHEAR TEST

PORE PRESSURE MEASUREMENTS

BORING NO. SC-36 SAMPLE NO. BAG

ELEV. OR DEPTH 3-4' JOB NO. SS7735

SOIL & MATERIAL ENGINEERS INC.

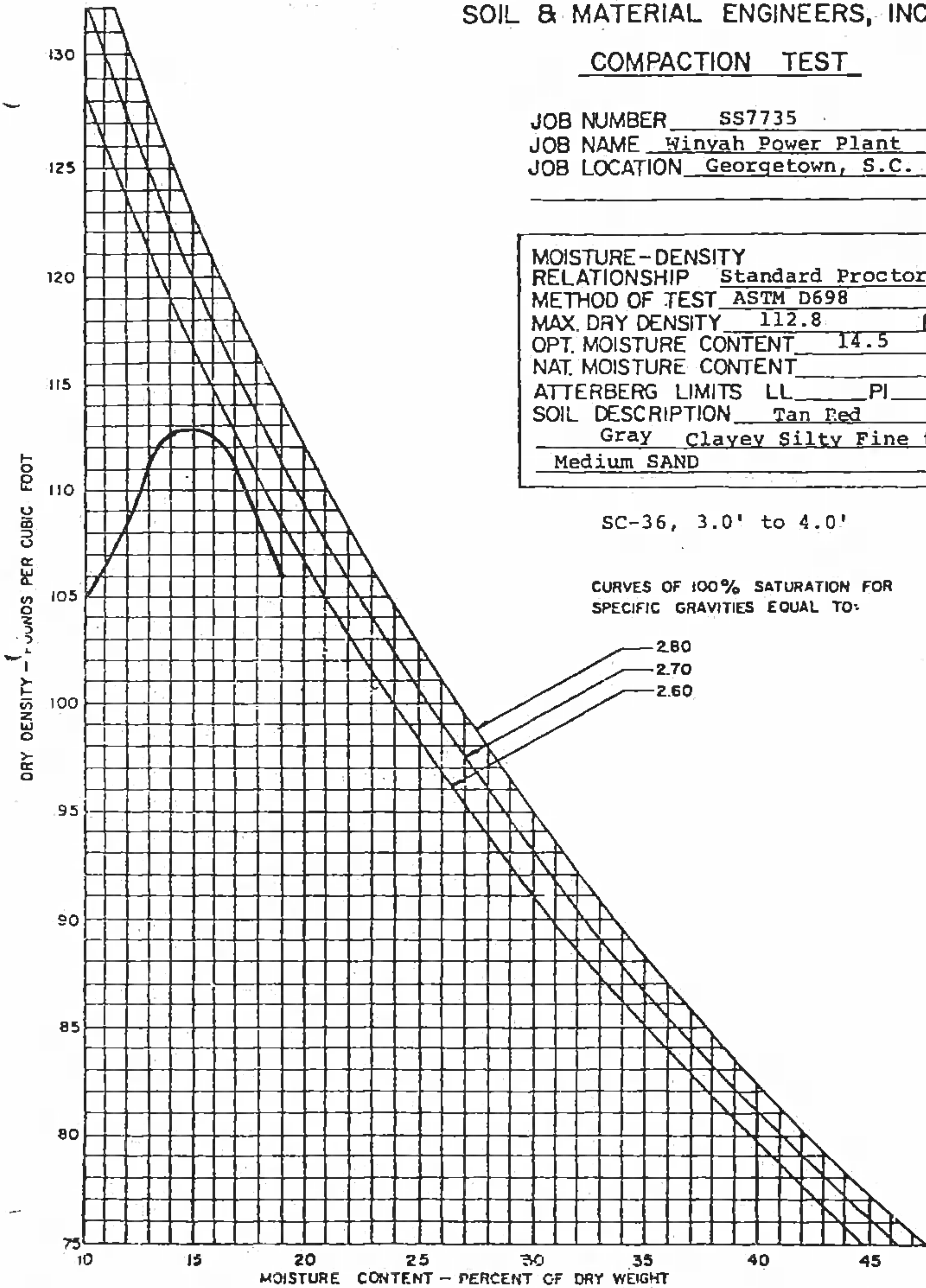
SOIL & MATERIAL ENGINEERS, INC.

COMPACTION TEST

JOB NUMBER SS7735
JOB NAME Winyah Power Plant
JOB LOCATION Georgetown, S.C.

MOISTURE-DENSITY RELATIONSHIP Standard Proctor
METHOD OF TEST ASTM D698
MAX. DRY DENSITY 112.8 PCF
OPT. MOISTURE CONTENT 14.5 %
NAT. MOISTURE CONTENT %
ATTERBERG LIMITS LL PI
SOIL DESCRIPTION Tan Red Gray Clayey Silty Fine to Medium SAND

SC-36, 3.0' to 4.0'



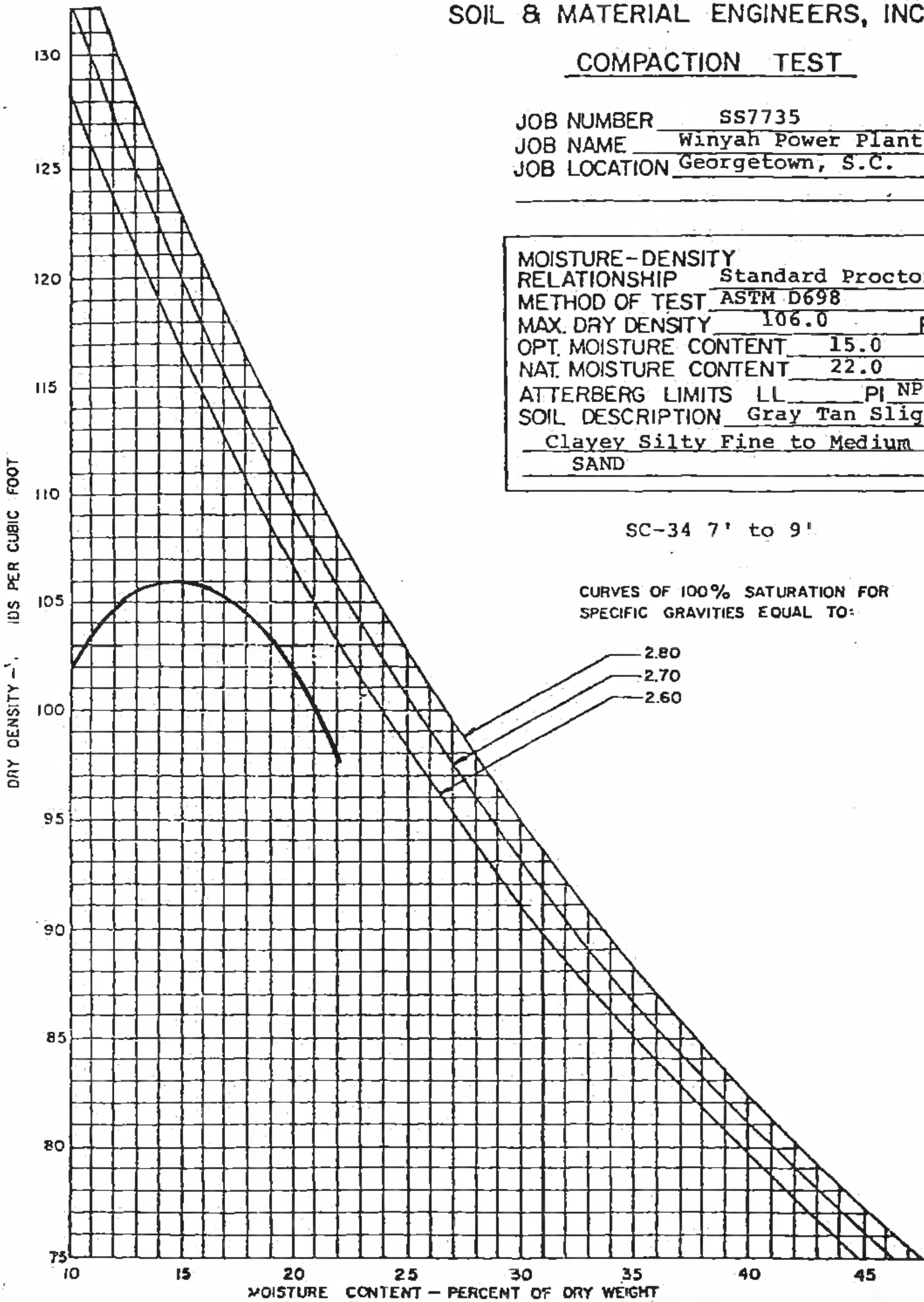
SOIL & MATERIAL ENGINEERS, INC.

COMPACTION TEST

JOB NUMBER SS7735
JOB NAME Winyah Power Plant
JOB LOCATION Georgetown, S.C.

MOISTURE-DENSITY RELATIONSHIP	Standard Proctor
METHOD OF TEST	ASTM D698
MAX. DRY DENSITY	106.0 PCF
OPT. MOISTURE CONTENT	15.0 %
NAT. MOISTURE CONTENT	22.0 %
ATTERBERG LIMITS	LL ___ PI NP
SOIL DESCRIPTION	Gray Tan Slightly Clayey Silty Fine to Medium SAND

SC-34 7' to 9'



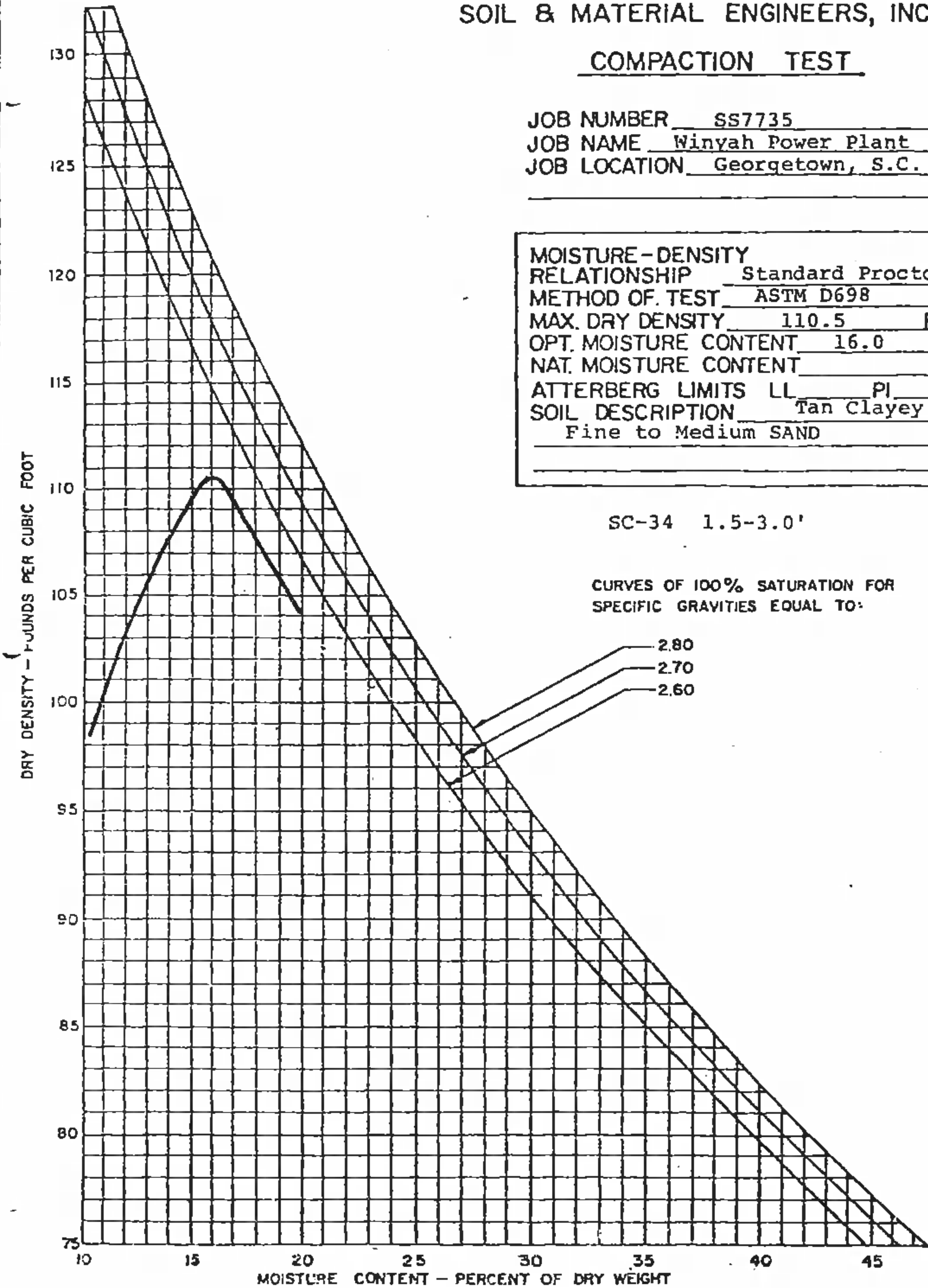
SOIL & MATERIAL ENGINEERS, INC.

COMPACTION TEST

JOB NUMBER SS7735
JOB NAME Winyah Power Plant
JOB LOCATION Georgetown, S.C.

MOISTURE-DENSITY RELATIONSHIP Standard Proctor
METHOD OF TEST ASTM D698
MAX. DRY DENSITY 110.5 PCF
OPT. MOISTURE CONTENT 16.0 %
NAT. MOISTURE CONTENT %
ATTERBERG LIMITS LL PI
SOIL DESCRIPTION Tan Clayey Fine to Medium SAND

SC-34 1.5-3.0'



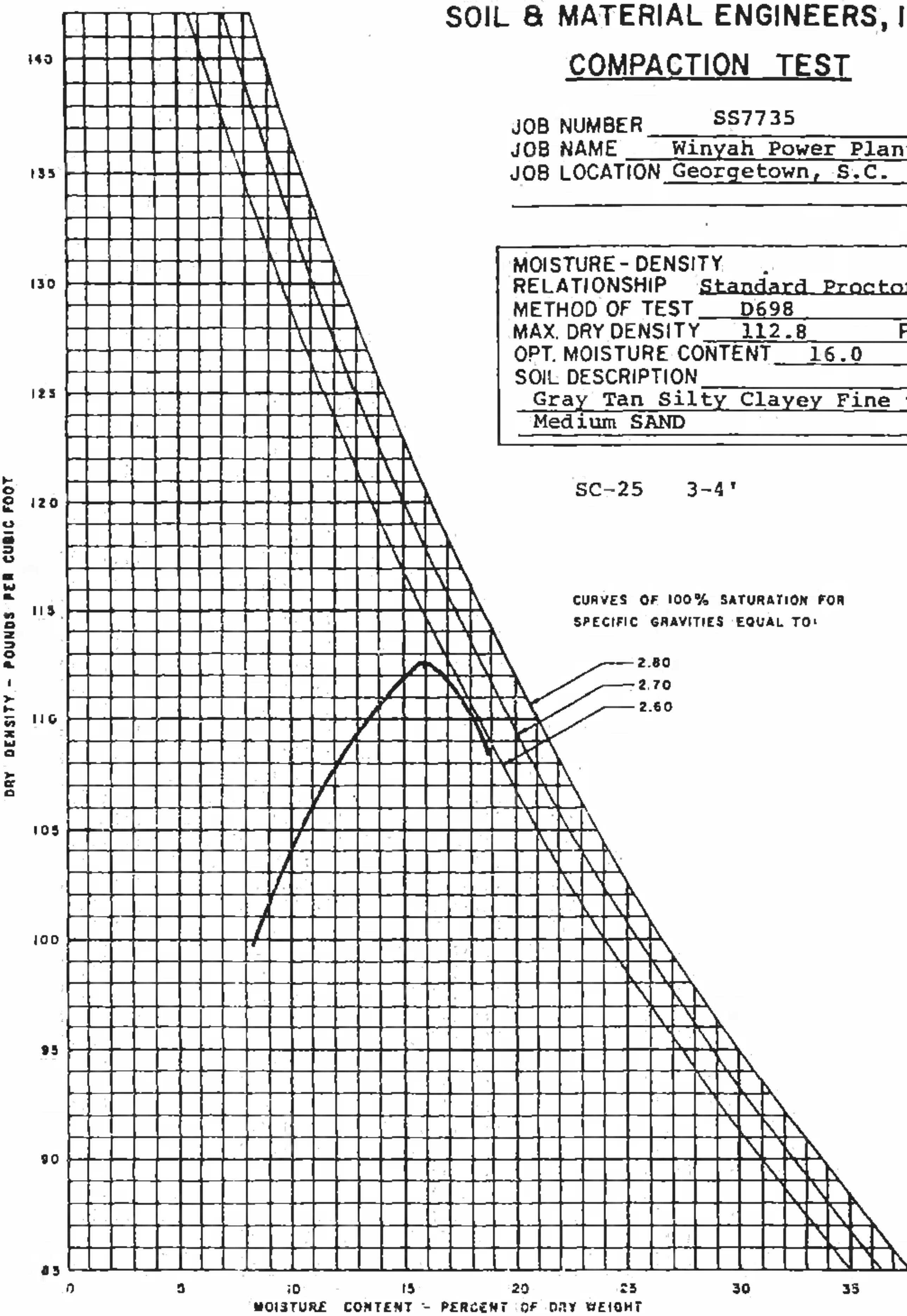
SOIL & MATERIAL ENGINEERS, INC.

COMPACTION TEST

JOB NUMBER SS7735
JOB NAME Winyah Power Plant
JOB LOCATION Georgetown, S.C.

MOISTURE - DENSITY RELATIONSHIP	Standard Proctor
METHOD OF TEST	D698
MAX. DRY DENSITY	112.8 PCF
OPT. MOISTURE CONTENT	16.0 %
SOIL DESCRIPTION	Gray Tan Silty Clayey Fine to Medium SAND

SC-25 3-4'



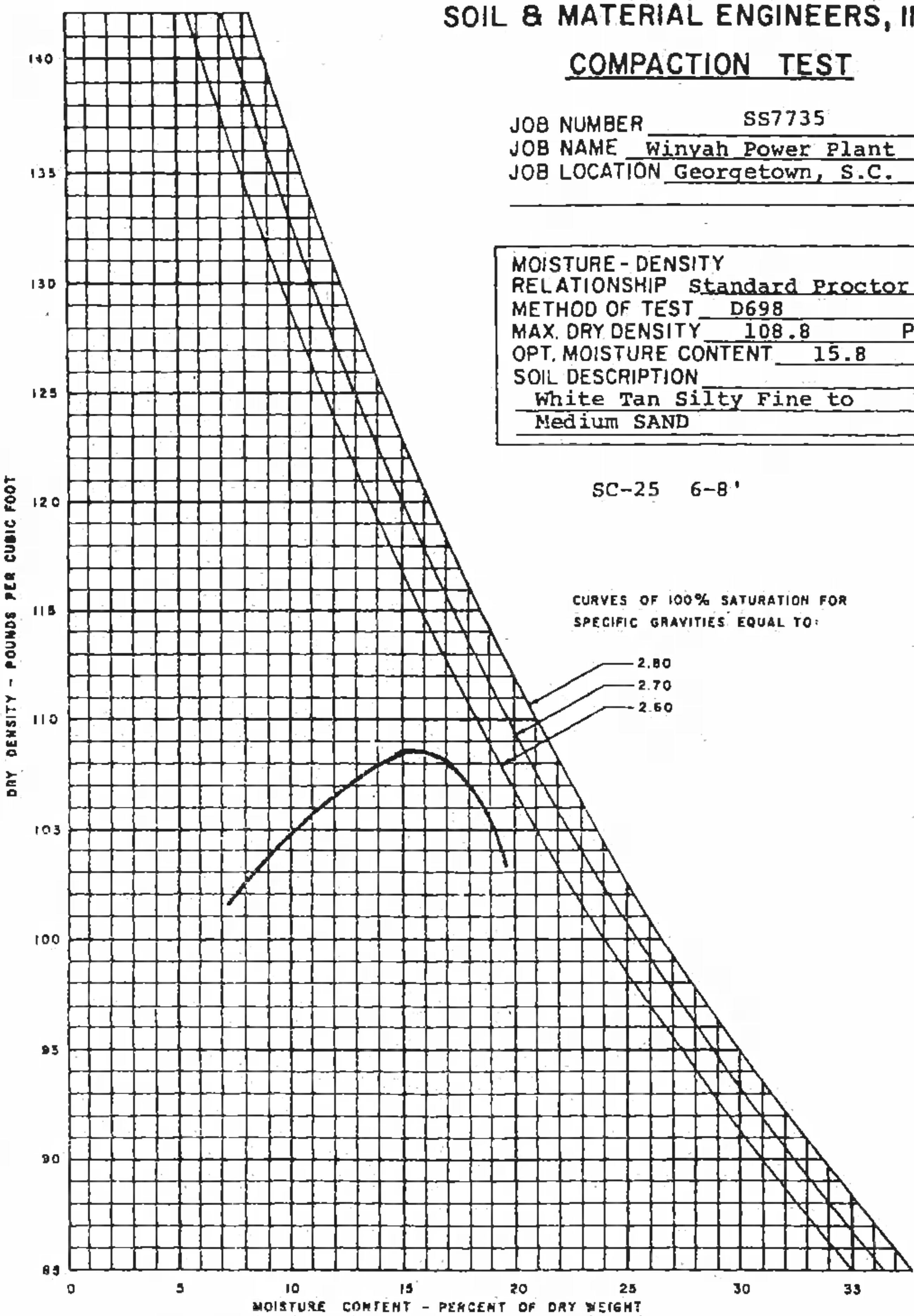
SOIL & MATERIAL ENGINEERS, INC.

COMPACTION TEST

JOB NUMBER SS7735
JOB NAME Winyah Power Plant
JOB LOCATION Georgetown, S.C.

MOISTURE - DENSITY RELATIONSHIP Standard Proctor
METHOD OF TEST D698
MAX. DRY DENSITY 108.8 PCF
OPT. MOISTURE CONTENT 15.8 %
SOIL DESCRIPTION
White Tan Silty Fine to Medium SAND

SC-25 6-8'



ATTACHMENT 4-C

PCRA Laboratory Testing Results
(PCRA, 1999)

TABLE 4
SUMMARY OF SOILS TESTING
WINYAH GENERATING STATION
SANTEE COOPER, SOUTH CAROLINA

Boring No.	Sample No.	Depth (ft)	Sample Type	General Field Soil Description	Testing Performed					USCS Soil Group Name
					Moisture Content (ASTM D2216)	Sieve Analysis w/o Hydrom.(ASTM D422)	Atterberg Limits (ASTM D4318)	Flex Wall Permeability Undist. (ASTM D5084)	Flex Wall Permeability Undist. (ASTM D5084)	
B-101	S-17	32.0-34.0	SPT	Gray Sand		sp-sm				Poorly Graded Sand w/ Silt
B-101	S-19	36.0-38.0	SPT	Gray Silty Sand		SM	NP			Silty Sand
B-102	S-13	24.0-26.0	SPT	Gray Silty Clay	61.9%	CH	LL = 67 PI = 38			Fat Clay w/ Sand
B-113	ST-1	55.0-57.0	Shelby Tube	Black Sandy Clay	42.9% 45.4%	CH	LL = 69 PI = 39			Fat Clay w/ Sand
B-113, ST-1 First Permability Test (Flexible Wall)								9.5×10^{-6} cm/s		
B-113, ST-1 Second Permability Test (Rigid Wall)									1.2×10^{-7} cm/s	Note: Modified ASTM D 5084
B-113, ST-1 Third Permability Test (Flexible Wall)								1.3×10^{-7} cm/s		
B-113	S-4	6.0-8.0	SPT	Dark Brown Sand		sp-sm				Poorly Graded Sand w/ Silt
B-116	S-10	43.0-45.0	SPT	Lt. Gray Sand w/ Shells		sp-sm				Poorly Graded Sand w/ Silt
B-201	S-5	8.0-10.0	SPT	Mottled Orange and Gray Clayey Sand		SM	NP			Silty Sand
B-201	S-7	12.0-14.0	SPT	Mottled Orange and Gray Sandy Clay	18.1%	SC	LL = 45 PI = 25			Clayey Sand
B-201	S-21	40.0-42.0	SPT	Gray Clayey Sand w/ shells		SC	LL = 47 PI = 27			Clayey Sand
B-205	S-6	10.0-12.0	SPT	Dark Brown Sand		sm				Silty Sand
B-206	S-18	34.0-36.0	SPT	Gray Sandy Clay	31.8%	SM	NP			Silty Sand
B-212	S-10	18.0-20.0	SPT	Gray/Brn Sandy Clay	19.1%	SC	LL = 37 PI = 20			Clayey Sand
B-214	S-20	38.0-40.0	SPT	Gray Silty Clay	95.5%	cl	Not Enough Material			Lean Clay
B-214	S-25	60.0-62.0	SPT	Gray/Blk Sandy Clay	44.7%	CH	LL = 69 PI = 38			Fat Clay
PZ-104	S-17	42.0-44.0	SPT	Brown Sand w/ Shells		sp-sm				Poorly Graded Sand w/ Silt

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LABORATORY TEST REPORT

June 22, 1999

Project No. 99139-01

Mr. Robbie Warner
PC Rizzo
Expo Mart Suite 270 E
105 Mall Blvd.
Monroeville, PA 15146

RE: Soils Testing – Santee Cooper / Winyah 99-1988.02

Transmitted herein are the results of the soils testing performed for PC Rizzo verified on the Project Verification Form, submitted June 4,1999. The testing was performed in accordance with the ASTM methods listed on the enclosed data sheets. The remaining sample materials for this project will be retained for a minimum of 90 days as directed by the Geotechnics' Quality Program.

Disclaimer

The test results are believed to be representative of the samples submitted but are indicative only of the specimens which were evaluated. Geotechnics has no direct knowledge of the origin of the samples, implies no position with regard to the disposition of the test results, i.e., pass/fail, and makes no claims as to the suitability of the material for its intended use.

The test data and all associated project information provided shall be held in strict confidence and disclosed to other parties only with authorization of the Client and Geotechnics. The test data submitted herein is considered integral with this report and is not to be reproduced except in whole and only with the authorization of the Client and Geotechnics.

We are pleased to provide these testing services. Should you have any questions or if we may be of further assistance, please do not hesitate to contact our office.

Respectively submitted,

David R. Backstrom
Laboratory Director

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MOISTURE CONTENT

ASTM D 2216 (SOP-S1)

Client PC RIZZO
Client Reference S.C. WINYAH 99-1988.02
Project No. 99139-01

Lab ID	.003	.006
Boring No.	B-102	B-201
Depth (ft)	24-26	12-14
Sample No.	S-13	S-7
Tare Number	24	90
Wt. of Tare & WS (gm)	154.07	173.79
Wt. of Tare & DS (gm)	130.04	162.64
Wt. of Tare (gm)	91.25	100.94
Wt. of Water (gm)	24.03	11.15
Wt. of DS (gm)	38.79	61.7
Water Content (%)	**61.9**	**18.1**

Notes : *NA*

Tested By *BS* *Date* *6/8/99* *Checked By* iB *Date* 6-11-99

DCN: CT-S1 DATE 6-30-98 REVISION: 2

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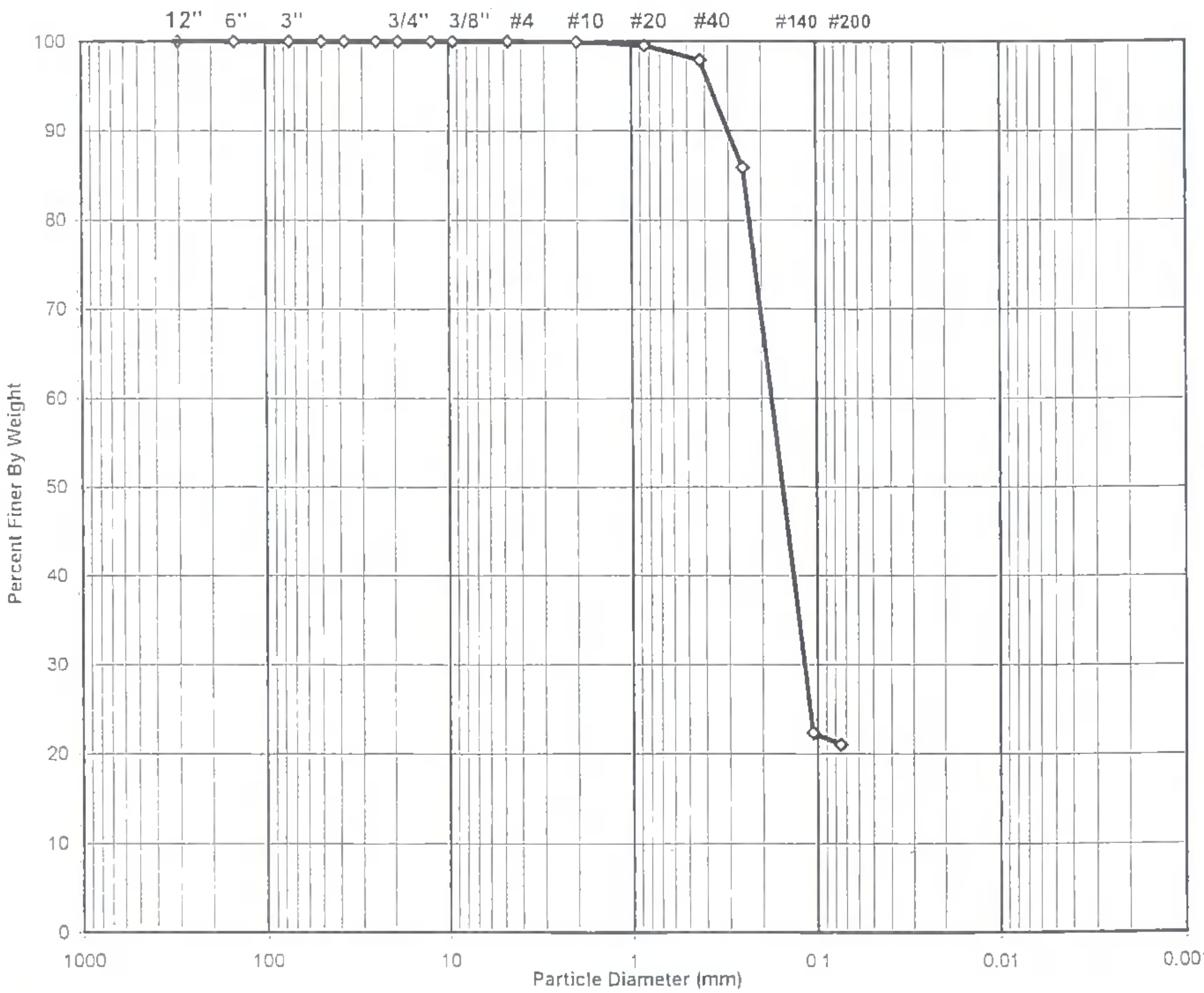
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	8-10
Project No.	99139-01	Sample No.	S-5
Lab ID	99139-01.005	Soil Color	BROWN

	SIEVE ANALYSIS		*HYDROMETER*
USCS	gravel	sand	silt and clay



USCS Symbol *SM, TESTED*

USCS Classification *SILTY SAND (NON-PLASTIC FINES)*

Tested By BS Date 6/8/99 Checked By LB Date 6-11-99

 DCN: CT-S3A DATE 6-25-99 REVISION: 2 *C:\MY DOCUMENTS\PrintQ\[C270.xls]Sheet1*

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	8-10
Project No.	99139-01	Sample No.	S-5
Lab ID	99139-01.005	Soil Color	BROWN

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	528	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	391.30	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	337.48	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	99.31	Weight of Tare (gm)	NA
Weight of Water (gm)	53.82	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	238.17	Weight of Dry Soil (gm)	NA
Moisture Content (%)	**22.6**	**Moisture Content (%)**	NA

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	238.17
Dry Weight - 3/4" Sample (gm)	188.1	Weight of minus #200 material (gm)	50.07
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	188.10
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	0.00	0.00	0.00	100.00	**100.00**
#4	4.75	0.00	0.00	0.00	100.00	**100.00**
#10	2.00	0.23	0.10	0.10	99.90	**99.90**
#20	0.850	0.82	0.34	0.44	99.56	**99.56**
#40	0.425	4.00	1.68	2.12	97.88	**97.88**
#60	0.250	28.73	12.06	14.18	85.82	**85.82**
#140	0.106	151.21	63.49	77.67	22.33	**22.33**
#200	0.075	3.11	1.31	78.98	21.02	**21.02**
Pan	-	50.07	21.02	100.00	-	-

Tested By BS Date 6/8/99 Checked By LB Date 6-11-99

 DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MY DOCUMENTS\PnnIQ\C270.xls]Sheet1

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ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	8-10
Project No.	99139-01	Sample No.	S-5
Lab ID	99139-01.005	Visual Description	**BROWN SILT**
			(Minus No. 40 sieve material, Airdried)

NON - PLASTIC MATERIAL

Tested By DA *Date* 6/9/99 *Checked By* LB *Date* 6-10-99

 DCN: CT-S4C DATE: 7-11-97 REVISION : 2 *C:\MY DOCUMENTS\PrintO\[C248.xls]Sheet1*

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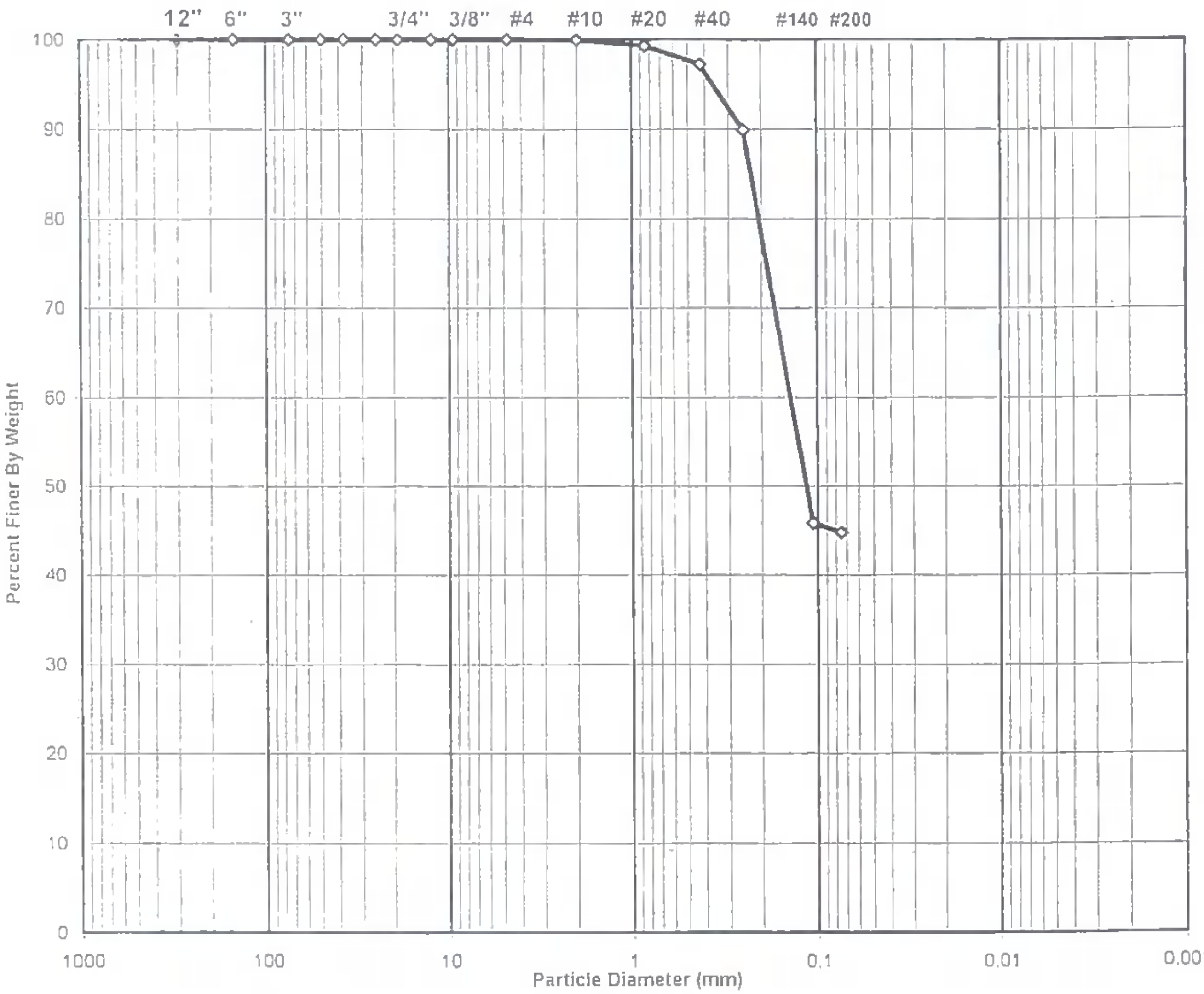
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	12-14
Project No.	99139-01	Sample No.	S-7
Lab ID	99139-01.006	Soil Color	BROWN

	SIEVE ANALYSIS		*HYDROMETER*
USCS	gravel	sand	silt and clay



USCS Symbol *SC, TESTED*

USCS Classification *CLAYEY SAND*

Tested By BS Date 6/8/99 Checked By LB Date 6-11-99

 DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MY DOCUMENTS\Print:O\[C271.xls]Sheet1

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	12-14
Project No.	99139-01	Sample No.	S-7
Lab ID	99139-01.006	Soil Color	BROWN

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	680	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	261.05	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	261.05	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	101.49	Weight of Tare (gm)	NA
Weight of Water (gm)	0.00	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	159.56	Weight of Dry Soil (gm)	NA
Moisture Content (%)	0.0	**Moisture Content (%)**	NA

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	159.56
Dry Weight - 3/4" Sample (gm)	88.2	Weight of minus #200 material (gm)	71.36
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	88.20
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	0.00	0.00	0.00	100.00	**100.00**
#4	4.75	0.00	0.00	0.00	100.00	**100.00**
#10	2.00	0.12	0.08	0.08	99.92	**99.92**
#20	0.850	1.09	0.68	0.76	99.24	**99.24**
#40	0.425	3.25	2.04	2.80	97.20	**97.20**
#60	0.250	11.67	7.31	10.11	89.89	**89.89**
#140	0.106	70.37	44.10	54.21	45.79	**45.79**
#200	0.075	1.70	1.07	55.28	44.72	**44.72**
Pan	-	71.36	44.72	100.00	-	-

Tested By BS Date 6/8/99 Checked By LB Date 6-11-99

DCN: CT-S3A DATE 6-25-98 REVISION: 2

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ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	12-14
Project No.	99139-01	Sample No.	S-7
Lab ID	99139-01.006	Soil Description	BROWN LEAN CLAY

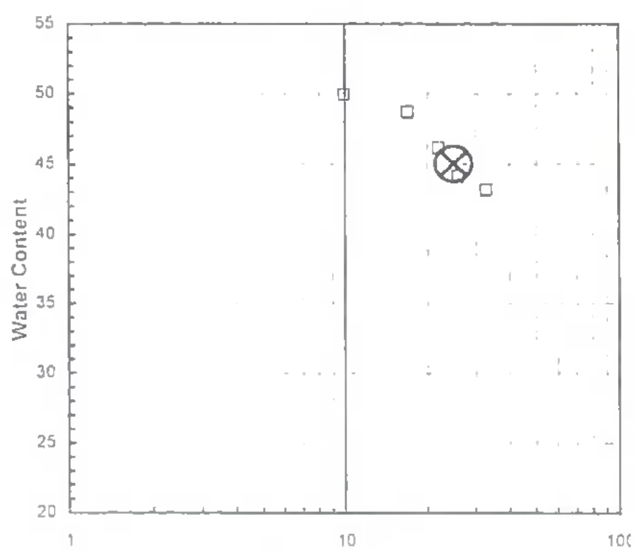
Note: The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description. (Minus No. 40 sieve material, Airdried)

Liquid Limit Test	1	2	3	4	5	MULTIPOINT
Tare Number	161	2288	83A	135	1837	
Wt. of Tare & WS (gm)	32.92	35.53	28.41	34.15	33.43	
Wt. of Tare & DS (gm)	28.42	30.58	23.66	29.34	28.38	
Wt. of Tare (gm)	17.99	19.36	13.36	19.46	18.26	
Wt. of Water (gm)	4.5	4.95	4.75	4.81	5.05	
Wt. of DS (gm)	10.43	11.22	10.3	9.88	10.12	
Moisture Content (%)	**43.1**	**44.1**	**46.1**	**48.7**	**49.9**	
Number of Blows	**33**	**26**	**22**	**17**	**10**	

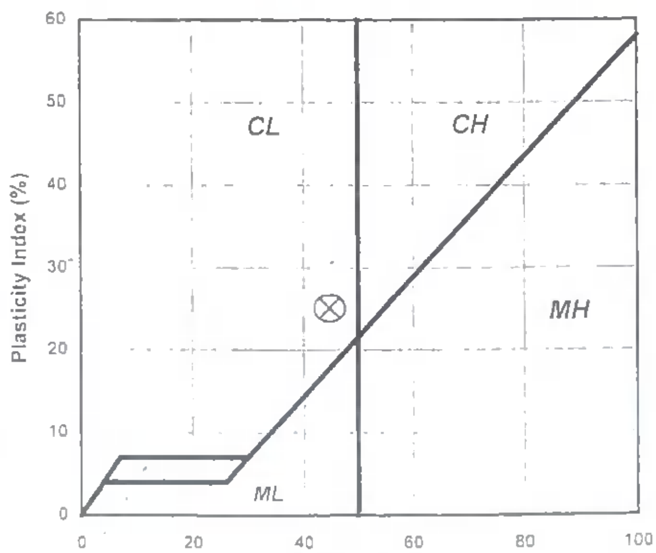
Plastic Limit Test	1	2	3
Tare Number	2023	67	26
Wt. of Tare & WS (gm)	23.54	24.18	22.46
Wt. of Tare & DS (gm)	22.49	23.20	21.47
Wt. of Tare (gm)	17.20	18.18	16.38
Wt. of Water (gm)	1.05	0.98	0.99
Wt. of DS (gm)	5.29	5.02	5.09
Moisture Content (%)	**19.8**	**19.5**	**19.4**

Test Results	
Liquid Limit (%)	45
Plastic Limit (%)	20
Plasticity Index (%)	25
USCS Symbol	CL

Flow Curve



Plasticity Chart



CL- ML

Tested By LB *Date* 6/9/99 *Checked By* *Date* 6/10/99

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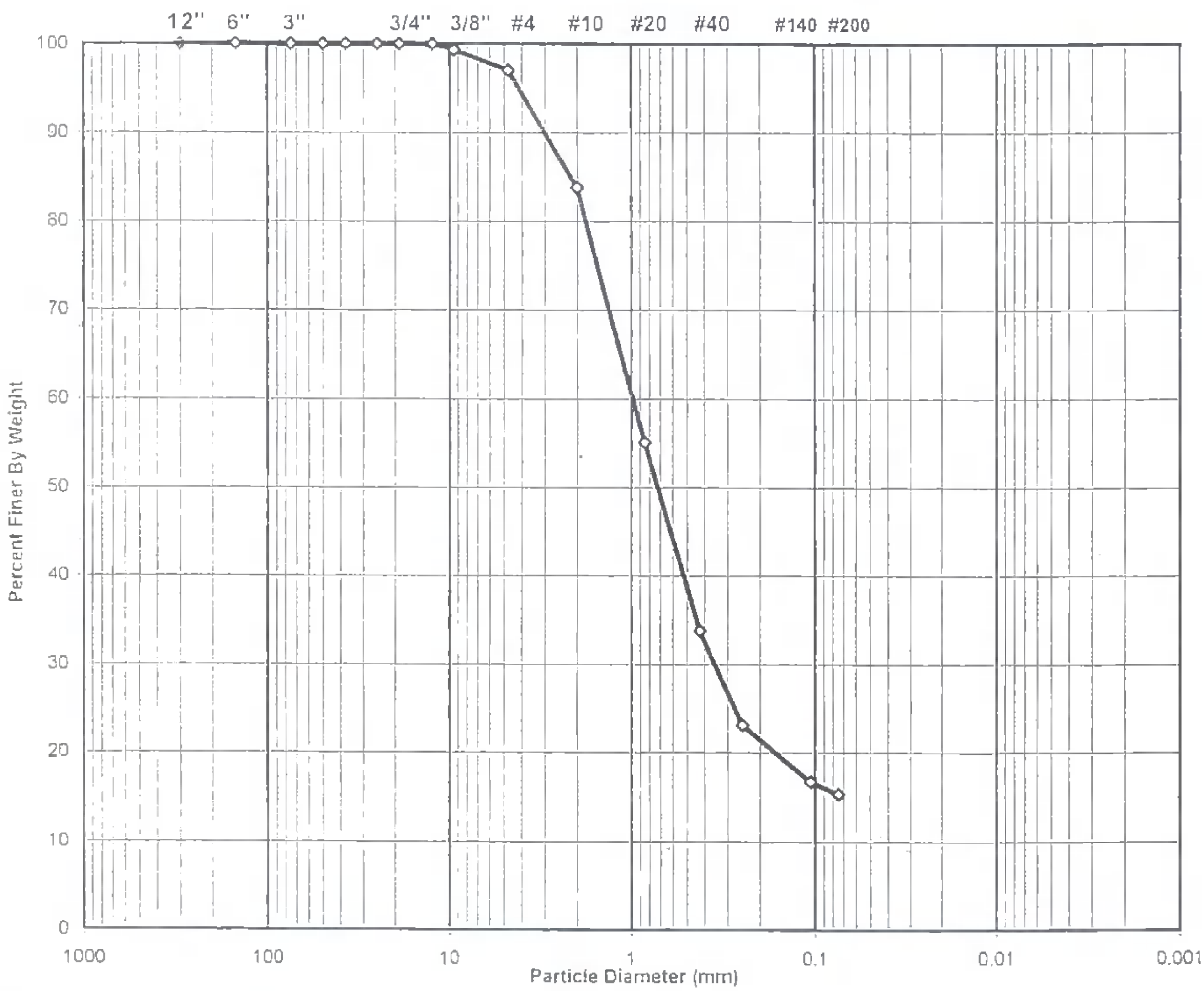
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	40-42
Project No.	99139-01	Sample No.	S-21
Lab ID	99139-01.007	Soil Color	GRAY

USCS	*SIEVE ANALYSIS*		*HYDROMETER*
	gravel	sand	silt and clay



USCS Symbol *SC, TESTED*

USCS Classification *CLAYEY SAND*

Tested By	BS	Date	6/8/99	Checked By	LB	Date	6-11-99

 DCN: GT-S3A DATE 6-25-98 REVISION: 2 C:\MY DOCUMENTS\PrintQ\[C272.xls]Sheet1

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	40-42
Project No.	99139-01	Sample No.	S-21
Lab ID	99139-01.007	Soil Color	GRAY

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	520	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	339.64	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	339.64	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	104.32	Weight of Tare (gm)	NA
Weight of Water (gm)	0.00	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	235.32	Weight of Dry Soil (gm)	NA
Moisture Content (%)	0.0	**Moisture Content (%)**	NA

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	235.32
Dry Weight - 3/4" Sample (gm)	199.3	Weight of minus #200 material (gm)	36.05
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	199.27
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	1.59	0.68	0.68	99.32	**99.32**
#4	4.75	5.38	2.29	2.96	97.04	**97.04**
#10	2.00	31.20	13.26	16.22	83.78	**83.78**
#20	0.850	67.56	28.71	44.93	55.07	**55.07**
#40	0.425	50.10	21.29	66.22	33.78	**33.78**
#60	0.250	25.17	10.70	76.92	23.08	**23.08**
#140	0.106	14.98	6.37	83.28	16.72	**16.72**
#200	0.075	3.29	1.40	84.68	15.32	**15.32**
Pan	-	36.05	15.32	100.00	-	-

Tested By BS Date 6/8/99 Checked By UB Date 6-11-99

DCN: CT-S3A DATE 6-25-98 REVISION: 2

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ONE POINT ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO	Boring No.	B-201
Client Reference	S.C. WINYAH 99-1988.02	Depth (ft)	40-42
Project No.	99139-01	Sample No.	S-21
Lab ID	99139-01.007	Soil Description	GRAY LEAN CLAY (Minus No. 40 sieve material, Airdried)

Liquid Limit Test	1	2
Tare Number	119	165
Wt. of Tare & WS (gm)	41.41	35.68
Wt. of Tare & DS (gm)	33.85	28.67
Wt. of Tare (gm)	17.73	13.62
Wt. of Water (gm)	7.56	7.01
Wt. of DS (gm)	16.12	15.05
Moisture Content (%)	46.9	46.6
Number of Blows	24	23

Blows	K-factor
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

Plastic Limit Test	1	2	3
Tare Number	178	81	31
Wt. of Tare & WS (gm)	24.66	24.86	23.32
Wt. of Tare & DS (gm)	23.52	23.56	22.26
Wt. of Tare (gm)	18.12	16.84	17.02
Wt. of Water (gm)	1.14	1.3	1.06
Wt. of DS (gm)	5.4	6.72	5.24
Moisture Content (%)	**21.1**	**19.3**	**20.2**

Liquid Limit Test Results

Test 1

LL = 46.9

LL^{CORR} = 47

Test 2

LL = 46.6

LL^{CORR} = 46

Summary

Liquid Limit (%)	47	Plasticity Index (%)	27
Plastic Limit (%)	20	USCS Symbol	CL

Tested By DA *Date* 6/9/99 *Checked By* LB *Date* 6-10-99

DCN: CT-S4A DAT 6-25-98 REVISION: 3

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LABORATORY TEST REPORT

June 22, 1999

Project No. 99139-02

Mr. Robbie Warner
PC Rizzo
Expo Mart Suite 270 E
105 Mall Blvd.
Monroeville, PA 15146

RE: Soils Testing – Santee Cooper / Winyah 99-1988.02

Transmitted herein are the results of the soils testing performed for PC Rizzo verified on the Project Verification Form, submitted June 11,1999. The testing was performed in accordance with the ASTM methods listed on the enclosed data sheets. The remaining sample materials for this project will be retained for a minimum of 90 days as directed by the Geotechnics' Quality Program.

Disclaimer

The test results are believed to be representative of the samples submitted but are indicative only of the specimens which were evaluated. Geotechnics has no direct knowledge of the origin of the samples, implies no position with regard to the disposition of the test results, i.e., pass/fail, and makes no claims as to the suitability of the material for its intended use.

The test data and all associated project information provided shall be held in strict confidence and disclosed to other parties only with authorization of the Client and Geotechnics. The test data submitted herein is considered integral with this report and is not to be reproduced except in whole and only with the authorization of the Client and Geotechnics.

We are pleased to provide these testing services. Should you have any questions or if we may be of further assistance, please do not hesitate to contact our office.

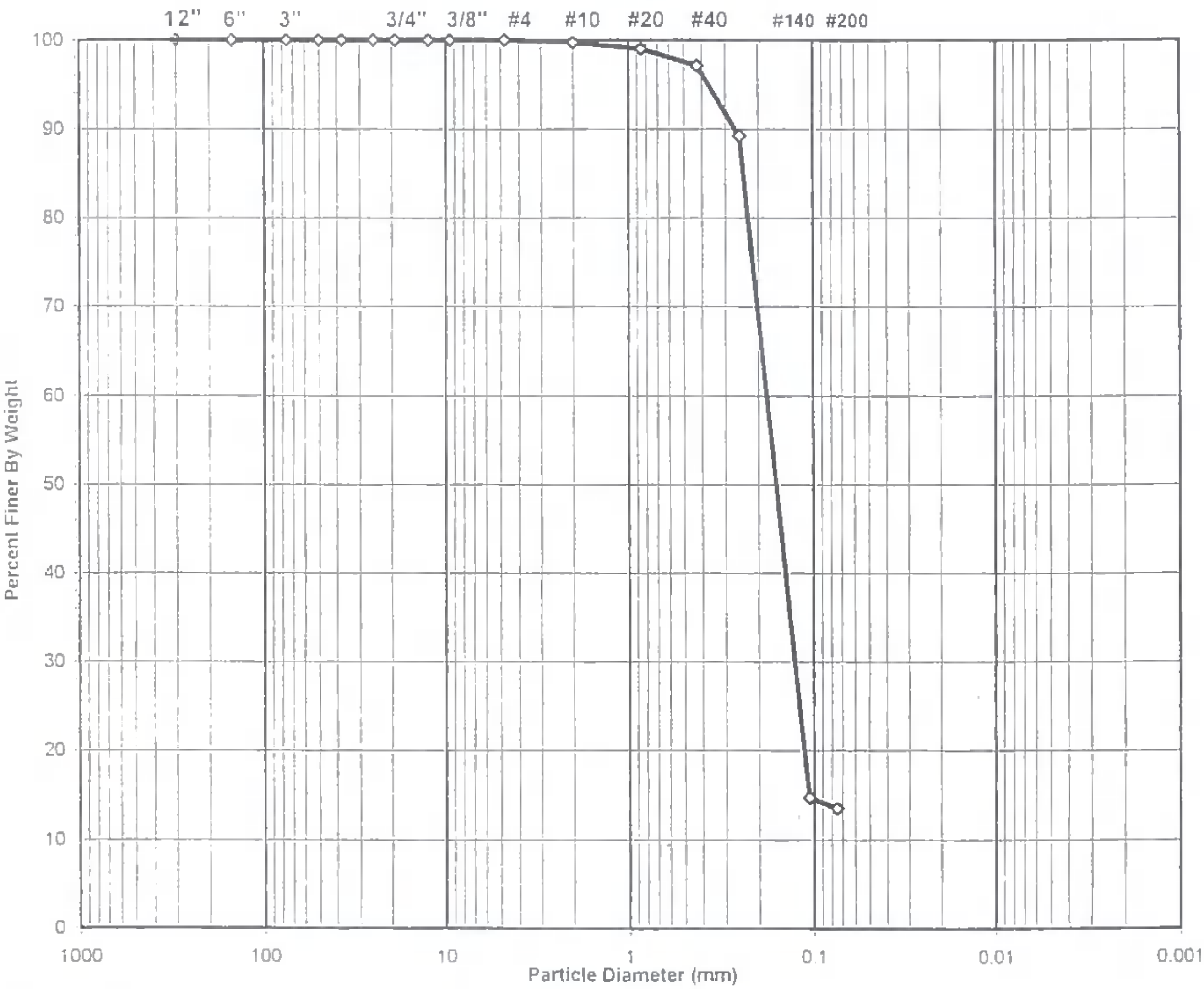
Respectively submitted,

David R. Backstrom
Laboratory Director

SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO ASSOC	Boring No.	B-205
Client Reference	SANTEE COOPER / WINYAH 99-1988.02	Depth (ft)	10-12
Project No.	99139-02	Sample No.	S-6
Lab ID	99139-02.004	Soil Color	BROWN

	SIEVE ANALYSIS		*HYDROMETER*
USCS	gravel	sand	silt and clay



USCS Symbol *sm, ASSUMED*

USCS Classification SILTY SAND

Tested By BS Date 6/14/99 Checked By [signature] Date 6/17/99

 DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MY DOCUMENTS\PrintQ\[C348.xls]Sheet1

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO ASSOC	Boring No.	B-205
Client Reference	SANTEE COOPER / WINYAH 99-1988.02	Depth (ft)	10-12
Project No.	99139-02	Sample No.	S-6
Lab ID	99139-02.004	Soil Color	BROWN

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	1063	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	327.70	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	301.74	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	102.76	Weight of Tare (gm)	NA
Weight of Water (gm)	25.96	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	198.98	Weight of Dry Soil (gm)	NA
Moisture Content (%)	13.0	**Moisture Content (%)**	NA

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	198.98
Dry Weight - 3/4" Sample (gm)	172.1	Weight of minus #200 material (gm)	26.92
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	172.06
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	0.00	0.00	0.00	100.00	**100.00**
#4	4.75	0.00	0.00	0.00	100.00	**100.00**
#10	2.00	0.53	0.27	0.27	99.73	**99.73**
#20	0.850	1.49	0.75	1.02	98.98	**98.98**
#40	0.425	3.62	1.82	2.83	97.17	**97.17**
#60	0.250	15.76	7.92	10.75	89.25	**89.25**
#140	0.106	148.29	74.53	85.28	14.72	**14.72**
#200	0.075	2.37	1.19	86.47	13.53	**13.53**
Pan	-	26.92	13.53	100.00	-	-

Tested By BS Date 6/14/99 Checked By [signature] Date 6/17/99

DCN: CT-S3A DATE 6-25-98 REVISION: 2

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MOISTURE CONTENT

ASTM D 2216 (SOP-S1)

Client	PC RIZZO ASSOC
Client Reference	SANTEE COOPER / WINYAH 99-1988.02
Project No.	99139-02

Lab ID	.005	.006	.007	.008
Boring No.	B-206	B-212	B-214	B-214
Depth (ft)	34-36	18-20	38-40	60-62
Sample No.	S-18	S-10	S-20	S-25
Tare Number	1310	11006	1871	55
Wt. of Tare & WS (gm)	155.59	157.36	90.88	163.36
Wt. of Tare & DS (gm)	142.83	146.59	65.86	145.85
Wt. of Tare (gm)	102.68	90.26	39.67	106.66
Wt. of Water (gm)	12.76	10.77	25.02	17.51
Wt. of DS (gm)	40.15	56.33	26.19	39.19
Water Content (%)	**31.8**	**19.1**	**95.5**	**44.7**

Notes : *NA*

Tested By *BS* *Date* *6/14/99* *Checked By* [signature] *Date* 6/16/99

 DCN CT-S1 DATE 5-30-98 REVISION 2 C:\MY DOCUMENTS\Print\Q\C321.xls]Sheet1

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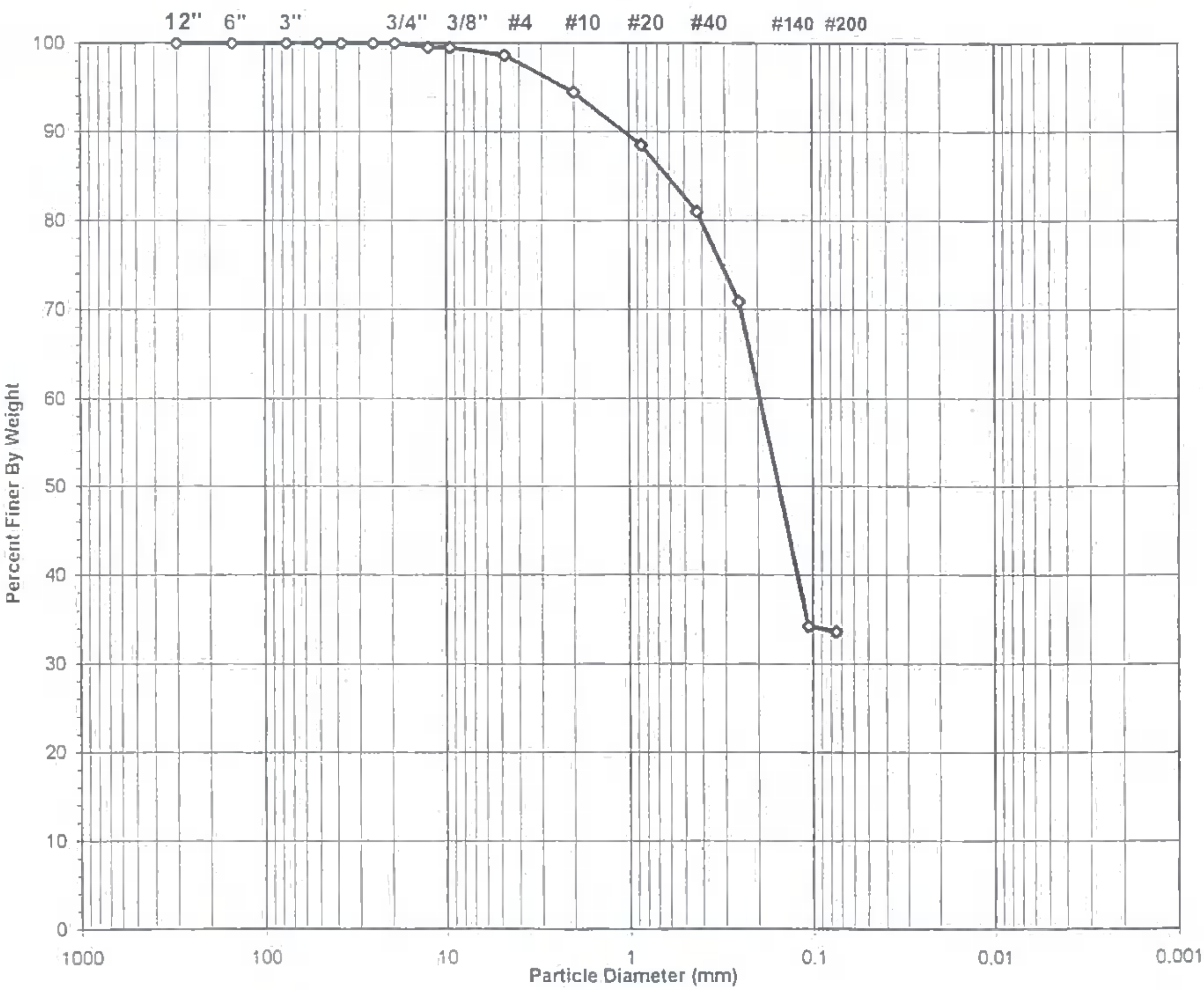
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-206
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	34-36
Project No.	99139-02	Sample No.	S-18
Lab ID	99139-02.005	Soil Color	GRAY

USCS	*SIEVE ANALYSIS*		*HYDROMETER*
	gravel	sand	silt and clay



USCS Symbol *SM, TESTED*

USCS Classification *SILTY SAND (NON-PLASTIC FINES)*

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-206
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	34-36
Project No.	99139-02	Sample No.	S-18
Lab ID	99139-02.005	Soil Color	GRAY

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	1679	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	336.17	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	312.04	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	97.46	Weight of Tare (gm)	NA
Weight of Water (gm)	24.13	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	214.58	Weight of Dry Soil (gm)	NA
Moisture Content (%)	**11.2**	**Moisture Content (%)**	**NA**

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	214.58
Dry Weight - 3/4" Sample (gm)	142.3	Weight of minus #200 material (gm)	72.26
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	142.32
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	1.05	0.49	0.49	99.51	**99.51**
3/8"	9.50	0.00	0.00	0.49	99.51	**99.51**
#4	4.75	1.77	0.82	1.31	98.69	**98.69**
#10	2.00	9.08	4.23	5.55	94.45	**94.45**
#20	0.850	12.72	5.93	11.47	88.53	**88.53**
#40	0.425	16.15	7.53	19.00	81.00	**81.00**
#60	0.250	21.72	10.12	29.12	70.88	**70.88**
#140	0.106	78.56	36.61	65.73	34.27	**34.27**
#200	0.075	1.27	0.59	66.32	33.68	**33.68**
Pan	-	72.26	33.68	100.00	-	-

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MSOFFICE\EXCEL\PrintQ\[C365.xls]Sheet1

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ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO ASSOC	Boring No.	B-206
Client Reference	SANTEE COOPER / WINYAH 99-1988.02	Depth (ft)	34-36
Project No.	99139-02	Sample No.	S-18
Lab ID	99139-02.005	Visual Description	GRAY SILT
			(MInus No. 40 sieve material, Airdried)

NON - PLASTIC MATERIAL

Tested By LB *Date* 6/17/99 *Checked By* [signature] *Date* 6/17/99

 DCN. CT-S4C DATE 7-11-97 REVISION : 2 C:\[Projectionbuttons.xls]Sheet1

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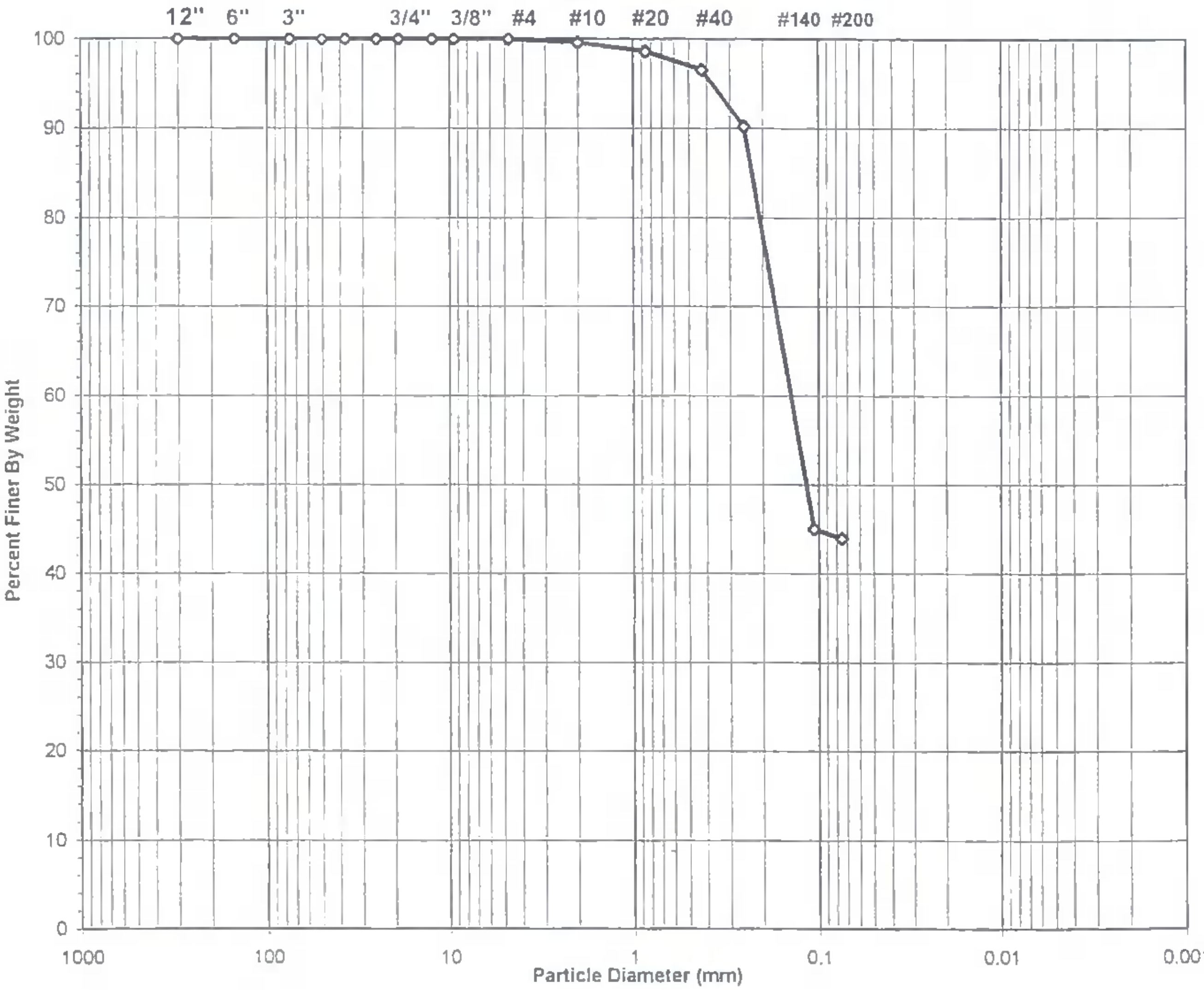
SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-212
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	18-20
Project No.	99139-02	Sample No.	S-10
Lab ID	99139-02.006	Soil Color	BROWN

USCS	*SIEVE ANALYSIS*		*HYDROMETER*
	gravel	sand	silt and clay



USCS Symbol *SC, TESTED*

USCS Classification *CLAYEY SAND*

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

 DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MSOFFICE\EXCEL\PrintQ\C369.xls]Sheet1

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-212
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	18-20
Project No.	99139-02	Sample No.	S-10
Lab ID	99139-02.006	Soil Color	BROWN

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	1633	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	327.60	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	327.60	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	96.22	Weight of Tare (gm)	NA
Weight of Water (gm)	0.00	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	231.38	Weight of Dry Soil (gm)	NA
Moisture Content (%)	**0.0**	**Moisture Content (%)**	**NA**

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	231.38
Dry Weight - 3/4" Sample (gm)	129.7	Weight of minus #200 material (gm)	101.65
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	129.73
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	**Accumulated Percent Finer (%)**
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	0.00	0.00	0.00	100.00	**100.00**
#4	4.75	0.00	0.00	0.00	100.00	**100.00**
#10	2.00	0.86	0.37	0.37	99.63	**99.63**
#20	0.850	2.31	1.00	1.37	98.63	**98.63**
#40	0.425	4.83	2.09	3.46	96.54	**96.54**
#60	0.250	14.68	6.34	9.80	90.20	**90.20**
#140	0.106	104.48	45.16	54.96	45.04	**45.04**
#200	0.075	2.57	1.11	56.07	43.93	**43.93**
Pan	-	101.65	43.93	100.00	-	-

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

DCN: CT-S3A DATE 6-25-98 REVISION: 2

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ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO	Boring No.	B-212
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	18-20
Project No.	99139-02	Sample No.	S-10
Lab ID	99139-02.006	Soil Description	BROWN LEAN CLAY

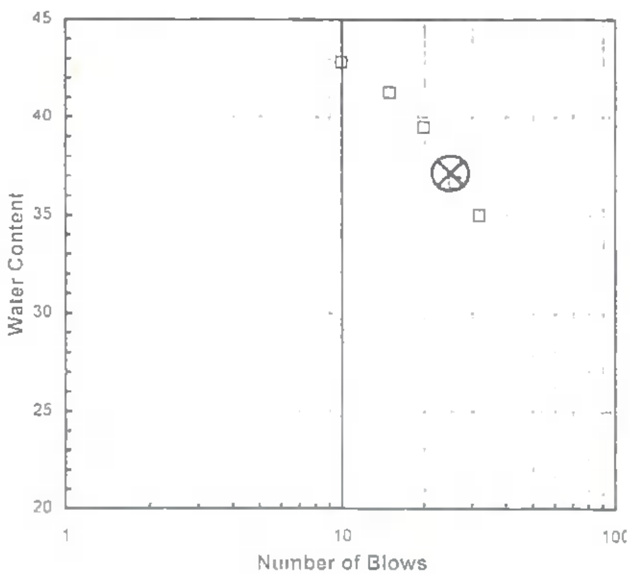
Note: The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description. (Minus No. 40 sieve material, Airdried)

Liquid Limit Test	1	2	3	4	5	MULTIPOINT
Tare Number	122	2032	131	2317	61	
Wt. of Tare & WS (gm)	38.11	35.78	41.27	36.43	37.58	
Wt. of Tare & DS (gm)	32.79	30.25	34.93	30.33	31.44	
Wt. of Tare (gm)	17.59	15.17	18.86	15.53	17.09	
Wt. of Water (gm)	5.32	5.53	6.34	6.1	6.14	
Wt. of DS (gm)	15.2	15.08	16.07	14.8	14.35	
Moisture Content (%)	35.0	36.7	39.5	41.2	42.8	
Number of Blows	32	26	20	15	10	

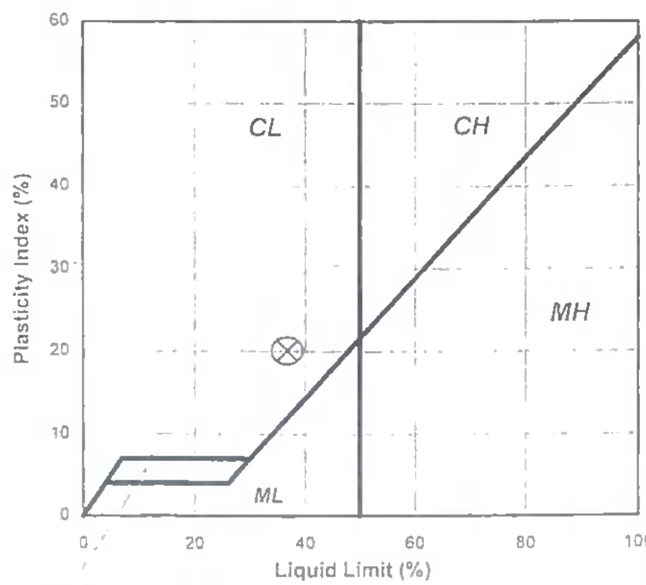
Plastic Limit Test	1	2	3
Tare Number	2235	99	2279
Wt. of Tare & WS (gm)	23.39	22.47	23.43
Wt. of Tare & DS (gm)	22.49	21.61	22.53
Wt. of Tare (gm)	17.13	16.38	17.13
Wt. of Water (gm)	0.9	0.86	0.9
Wt. of DS (gm)	5.36	5.23	5.4
Moisture Content (%)	16.8	16.4	16.7

Test Results	
Liquid Limit (%)	37
Plastic Limit (%)	17
Plasticity Index (%)	20
USCS Symbol	CL

Flow Curve



Plasticity Chart



Tested By LB *Date* 6/17/99 *Checked By* Jcm *Date* 6-21-99

DCN: CT-S4 DATE: 1/13/98 REVISION: 3 C:\MY DOCUMENTS\PrintQ\D67.xls]Sheet1

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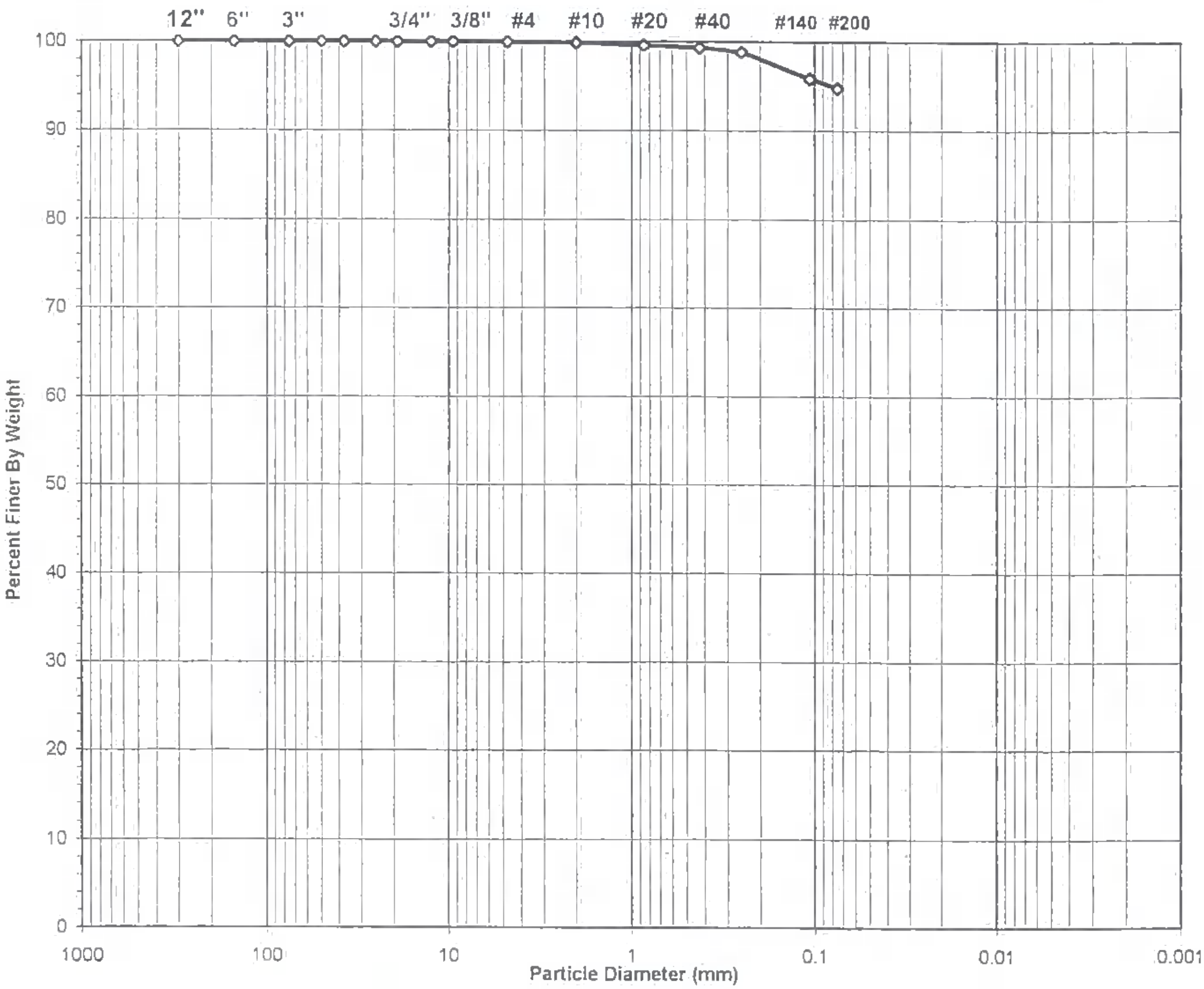
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-214
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	38-40
Project No.	99139-02	Sample No.	S-20
Lab ID	99139-02.007	Soil Color	GRAY

	SIEVE ANALYSIS		*HYDROMETER*
USCS	gravel	sand	silt and clay



USCS Symbol *cl, ASSUMED*

USCS Classification *LEAN CLAY*

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

DCN: CT-S3A DATE 6-25-98 REVISION: 2
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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-214
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	38-40
Project No.	99139-02	Sample No.	S-20
Lab ID	99139-02.007	Soil Color	**GRAY**

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	514	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	193.86	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	182.54	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	101.02	Weight of Tare (gm)	NA
Weight of Water (gm)	11.32	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	81.52	Weight of Dry Soil (gm)	NA
Moisture Content (%)	**13.9**	**Moisture Content (%)**	**NA**

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	81.52
Dry Weight - 3/4" Sample (gm)	4.2	Weight of minus #200 material (gm)	77.29
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	4.23
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	**Accumulated Percent Finer (%)**
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	0.00	0.00	0.00	100.00	**100.00**
#4	4.75	0.00	0.00	0.00	100.00	**100.00**
#10	2.00	0.10	0.12	0.12	99.88	**99.88**
#20	0.850	0.21	0.26	0.38	99.62	**99.62**
#40	0.425	0.23	0.28	0.66	99.34	**99.34**
#60	0.250	0.40	0.49	1.15	98.85	**98.85**
#140	0.106	2.42	2.97	4.12	95.88	**95.88**
#200	0.075	0.87	1.07	5.19	94.81	**94.81**
Pan	-	77.29	94.81	100.00	-	-

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DCN: CT-S3A DATE 6-25-98 REVISION: 2

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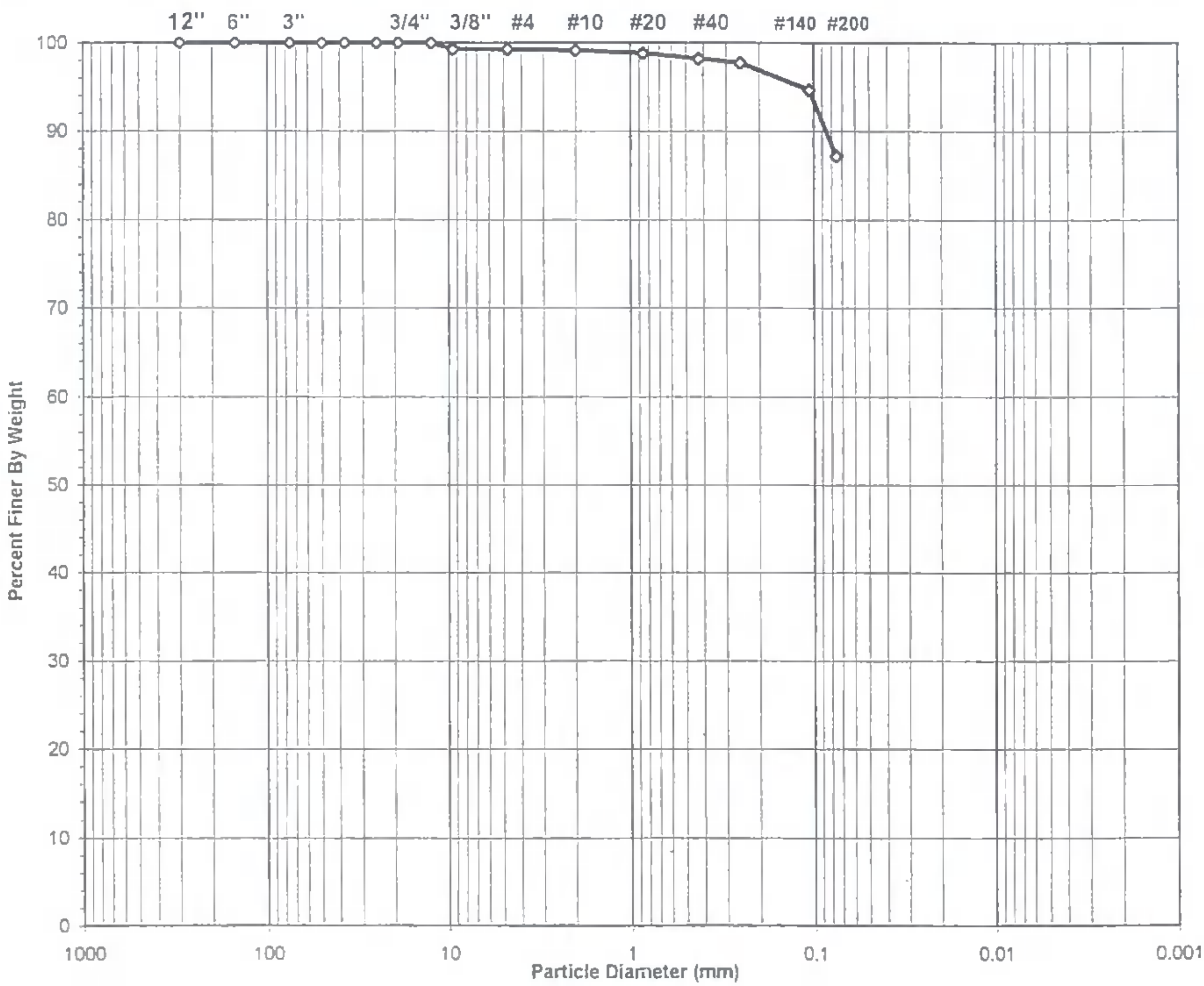
SIEVE ANALYSIS
ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-214
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	60-62
Project No.	99139-02	Sample No.	S-25
Lab ID	99139-02.008	Soil Color	GRAY

USCS	SIEVE ANALYSIS		HYDROMETER
	gravel	sand	silt and clay



USCS Symbol *CH, TESTED*

USCS Classification *FAT CLAY*

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

 DCN: CT-S3A DATE 6-25-98 REVISION: 2 C:\MSOFFICE\EXCEL\PrintQ\[C368.xls]Sheet1

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WASH SIEVE ANALYSIS

ASTM D 422-63 (SOP-S3)

Client	PC RIZZO	Boring No.	B-214
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	60-62
Project No.	99139-02	Sample No.	S-25
Lab ID	99139-02.008	Soil Color	**GRAY**

Moisture Content of Passing 3/4" Material		Water Content of Retained 3/4" Material	
Tare No.	1657	Tare No.	NA
Wgt.Tare + Wet Specimen (gm)	244.97	Wgt.Tare + Wet Specimen (gm)	NA
Wgt.Tare + Dry Specimen (gm)	244.97	Wgt.Tare + Dry Specimen (gm)	NA
Weight of Tare (gm)	93.77	Weight of Tare (gm)	NA
Weight of Water (gm)	0.00	Weight of Water (gm)	NA
Weight of Dry Soil (gm)	151.20	Weight of Dry Soil (gm)	NA
Moisture Content (%)	**0.0**	**Moisture Content (%)**	**NA**

Wet Weight -3/4" Sample (gm)	NA	Weight of the Dry Specimen (gm)	151.20
Dry Weight - 3/4" Sample (gm)	19.3	Weight of minus #200 material (gm)	131.87
Wet Weight +3/4" Sample (gm)	NA	Weight of plus #200 material (gm)	19.33
Dry Weight + 3/4" Sample (gm)	0.00		
Total Dry Weight Sample (gm)	NA		

Sieve Size	Sieve Opening (mm)	Wgt.of Soil Retained (gm)	Percent Retained (%)	Accumulated Percent Retained (%)	Percent Finer (%)	Accumulated Percent Finer (%)
12"	300	0.00	0.00	0.00	100.00	**100.00**
6"	150	0.00	0.00	0.00	100.00	**100.00**
3"	75	0.00	0.00	0.00	100.00	**100.00**
2"	50	0.00	0.00	0.00	100.00	**100.00**
1 1/2"	37.5	0.00	0.00	0.00	100.00	**100.00**
1"	25.0	0.00	0.00	0.00	100.00	**100.00**
3/4"	19.0	0.00	0.00	0.00	100.00	**100.00**
1/2"	12.50	0.00	0.00	0.00	100.00	**100.00**
3/8"	9.50	1.09	0.72	0.72	99.28	**99.28**
#4	4.75	0.00	0.00	0.72	99.28	**99.28**
#10	2.00	0.24	0.16	0.88	99.12	**99.12**
#20	0.850	0.48	0.32	1.20	98.80	**98.80**
#40	0.425	0.93	0.62	1.81	98.19	**98.19**
#60	0.250	0.71	0.47	2.28	97.72	**97.72**
#140	0.106	4.64	3.07	5.35	94.65	**94.65**
#200	0.075	11.24	7.43	12.78	87.22	**87.22**
Pan	-	131.87	87.22	100.00	-	-

Tested By BS Date 6/14/99 Checked By LB Date 6-21-99

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ONE POINT ATTERBERG LIMIT

ASTM D 4318-96 (SOP - S4)

Client	PC RIZZO	Boring No.	B-214
Client Reference	S.C./ WINYAH 99-1988.02	Depth (ft)	60-62
Project No.	99139-02	Sample No.	S-25
Lab ID	99139-02.008	Soil Description	**GRAY FAT CLAY**
			(Minus No. 40 sieve material, Airdried)

Liquid Limit Test	1	2
Tare Number	155	2306
Wt. of Tare & WS (gm)	29.64	31.72
Wt. of Tare & DS (gm)	24.2	25.28
Wt. of Tare (gm)	16.26	15.85
Wt. of Water (gm)	5.44	6.44
Wt. of DS (gm)	7.94	9.43
Moisture Content (%)	68.5	68.3
Number of Blows	26	27

Blows	K-factor
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

Plastic Limit Test	1	2	3
Tare Number	1882	2057	1883
Wt. of Tare & WS (gm)	25.61	25.83	18.02
Wt. of Tare & DS (gm)	24.09	24.3	16.6
Wt. of Tare (gm)	19.26	19.38	11.9
Wt. of Water (gm)	1.52	1.53	1.42
Wt. of DS (gm)	4.83	4.92	4.7
Moisture Content (%)	31.5	31.1	30.2

Liquid Limit Test Results

Test 1

LL = 68.5
LL^{CORR} = 69

Test 2

LL = 68.3
LL^{CORR} = 69

Summary

Liquid Limit (%)	69	Plasticity Index (%)	38
Plastic Limit (%)	31	USCS Symbol	CH

Tested By TO *Date* 6/17/99 *Checked By* LB *Date* 6-19-99

 DCN: CT-S4A DAT 6-25-98 REVISION: 3 C:\MY DOCUMENTS\PrintQ\D66.xls]Sheet1

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ATTACHMENT 5

Subsurface Stratigraphy and Material Properties

Geosyntec consultants

CALCULATION PACKAGE COVER SHEET

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

TITLE OF PACKAGE: **SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES: SLURRY POND**

Calculation Prepared by:	Signature	[signature]	10/12/2016
	Name	James McNash, P.E.	Date
Assumptions & Procedures Checked by: (peer reviewer)	Signature	[signature]	10/12/2016
	Name	Ming Zhu, Ph.D., P.E.	Date
Computations Checked by:	Signature	[signature]	10/12/2016
	Name	Weston Shin, Ph.D., P.E.	Date
Computations Back-checked by:	Signature	[signature]	10/12/2016
	Name	James McNash, P.E.	Date
Approved by: (pm or designate)	Signature	[signature]	10/12/2016
	Name	Fabian Benavente, P.E.	Date

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	J. McNash	Date:	10/11/2016	Reviewed by:	W. Shin/ M. Zhu	Date:	10/11/2016

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

SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES: SLURRY POND

INTRODUCTION

This calculation package was prepared to present the subsurface stratigraphy and material properties supporting the geotechnical analyses for the Slurry Pond 3 & 4 (Slurry Pond) located at Winyah Generating Station (WGS or "Site"), which is owned and operated by Santee Cooper. This calculation package is an attachment to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry 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) phreatic surface interpretation and current water levels; (iv) Standard Penetration Test (SPT) and Cone Penetration Test (CPT) interpretation; (v) laboratory testing program; (vi) in-situ testing interpretation; (vii) calibration borings; (viii) recommended material properties; and (ix) references.

SITE INVESTIGATIONS

Historical Investigations

Two investigations were performed in the vicinity of the Slurry Pond by Soil and Materials Engineers, Inc. (S&ME) in 1977 and 1978 and Paul C. Rizzo and Associates, Inc. (PCRA) in 1999, respectively.

In 1977 and 1978, S&ME performed a general subsurface investigation supporting the construction of CCR surface impoundments that are currently referred to as the Slurry Pond, the West Ash Pond, and the South Ash Pond. Information collected during the S&ME investigation, as detailed in the report titled "Subsurface Investigation: Ash and Slurry Pond Dikes" (S&ME, 1978), was originally utilized to assess the suitability of on-site materials for design and construction of the perimeter dike structures. The subsurface investigation in the vicinity of the Slurry Pond included seventeen soil test borings with SPTs generally at 5-ft depth intervals and nine test pits excavated 10-15 ft below ground surface (bgs). Representative samples were collected by a standard split spoon sampler or as bulk samples from test pit excavations. Additionally, thin-walled Shelby tubes were utilized to collect undisturbed samples for laboratory testing. The laboratory program consisted of index testing (natural moisture content, grain size distribution and Atterberg limit tests), unit weight testing, compaction testing, consolidation testing, and shear strength testing of select samples. Compacted samples were also tested to evaluate the material properties of the perimeter dike once constructed.

In 1999, PCRA conducted a geotechnical and hydrogeologic investigation at WGS primarily on the perimeter dike structures of the Slurry Pond and Unit 2 Slurry Pond (PCRA, 1999). The purpose of the 1999 site investigation was to evaluate the hydraulic performance of the dike structures. Eighteen soil test borings were advanced through the centerline of the perimeter dikes and divider dike forming the

Slurry Pond. Temporary piezometers (PZ-201/PZ-202) was installed near the northern perimeter dike toe, which has since been abandoned. Soil test borings were performed using the mud rotary drilling method with a side discharge bit. Two drilling subcontractors were utilized during this investigation: Carolina Drilling and Mid-Atlantic Drilling, Inc. (MAD). Carolina Drilling advanced eight borings using a CME-55 truck mounted rig and, in general, continuously sampled with SPT blow counts recorded until refusal on "limestone" (geologic interpretation of this stratum is discussed later within this calculation package) was encountered. Once "limestone" was encountered, a casing was installed to allow coring this stratum. These borings were generally terminated once the underlying "Black Mingo Formation" (geologic interpretation of this stratum is discussed subsequently within this calculation package) was encountered, and two of these borings penetrated the upper 5 to 10 ft of the formation. MAD advanced ten borings using a trailer mounted CME-45 rig without sampling (i.e., without SPT measurements) until "limestone" was encountered. In seven of these borings, MAD cored the "limestone" stratum and penetrated nearly 10 ft into the "Black Mingo Formation." Nine representative soil samples collected in the vicinity of the Slurry Pond were sent to a geotechnical laboratory for testing. Thin-walled Shelby tube samples collected during this investigation were tested to assess the hydraulic conductivity of underlying soils.

Geosyntec Investigations

From February 21st to March 6th, 2013, Geosyntec conducted a geotechnical site investigation in the vicinity of the Slurry Pond and West Ash Pond. This geotechnical site investigation program consisted of eleven soil test borings with SPTs and split spoon samples, fifty-three CPTs, and three test pit excavations performed along the Slurry Pond perimeter dikes, the downstream toe of the perimeter dikes, the divider dike, and within the Slurry Pond. The as-built location of each CPT sounding and boring performed during the Geosyntec site investigation is shown in Figure 1. Soil Consultants, Inc. (SCI) of Charleston, SC was subcontracted to advance soil test borings using a CME-550X drill rig during this investigation. Eight soil test borings were advanced through the Slurry Pond dike centerline and three soil test borings were advanced near the downstream toe of the perimeter dikes. Soil test borings were advanced using the "rotary wash" or "mud rotary" method with a side discharge bit and in accordance with recommendations detailed by Idriss and Boulanger (2008) as shown in Table 1. Representative samples were collected during SPTs using a standard split spoon sampler at 5-ft depth intervals. Borings were terminated when the cemented "Chicora Member" (geologic interpretation of this stratum is discussed later within this calculation package) was encountered, and a final SPT was recorded. In one boring (GSB-4), the split spoon penetrated the "Chicora Member" stratum, which was observed to be thinner in that area, and encountered the underlying "Williamsburg Formation Clay" (geologic interpretation of this stratum is discussed later within this calculation package). GSB-4 was advanced an additional 10 ft before termination of the boring. In addition to SPT sampling, thin-walled Shelby tube samples were collected in the vicinity of the Slurry Pond perimeter dikes. Select samples were shipped to a geotechnical laboratory for testing, as summarized in Appendix 1. The laboratory testing program will be described in detail subsequently within this calculation package. Boreholes located on the dike centerline were left open for two to three days prior to abandonment. Geosyntec recorded the depths to water levels before the boreholes were backfilled with cement-bentonite grout.

Twenty-three CPT soundings with pore pressure measurements were performed to refusal through the perimeter, divider, and finger dike centerlines. An additional twenty-five CPT soundings were performed to refusal at the toe of the perimeter dikes, which was defined as the top of "Chicora" stratum. Four CPT soundings were performed through the flue gas desulfurization (FGD) residuals to the bottom of Slurry Pond during this subsurface investigation.

At select four CPT locations along the perimeter dike centerline and two locations along the perimeter dike toe, shear wave velocity (V_s) testing was conducted at 5-ft depth intervals. Additionally, pore pressure dissipation tests were performed at four, three, and four locations along the dike centerline, dike toe, and within the CCR, respectively. Attachment 3 to the Safety Factor Assessment Report provides the raw shear wave velocity and pore pressure dissipation test data.

Two hollow stem auger borings (HSA-1 and HSA-2) were advanced 25 to 30 ft bgs through the perimeter dike centerline immediately adjacent to two CPT soundings (CPT-7 and CPT-24). The readings of depth to the phreatic surface within the boreholes were collected at 1 hour and 24 hours after the drilling was completed.

Three test pits were excavated 5-ft bgs within the Slurry Pond, located approximately 60 ft from the finger dike, by means of a long stick excavator operated by Santee Cooper construction staff. Test Pit-1 (TP-1) was excavated on the western side of the finger dike; while, TP-2 and TP-3 were excavated on the eastern side of the finger dike. Bulk samples were collected in 5-gallon buckets and transported to the laboratory for geotechnical testing. The approximate locations of these Test Pits are shown in Figure 1.

In October and November 2013, Geosyntec re-mobilized to WGS to collect geotechnical subsurface information through additional soil test borings and CPT soundings in support of evaluating preliminary and conceptual closure alternatives for each CCR surface impoundment at WGS. While the site investigation was focused in the vicinity of the South Ash Pond, Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B, as part of this site investigation five soil test borings (SPT-106, SPT-106A, SPT-106B, SPT-107A, SPT-107B) were advanced by mud rotary wash drilling method and six CPT soundings (CPT-116, CPT-116A, CPT-117, CPT-119, CPT-120, and CPT-121) were performed within the vicinity of the Slurry Pond. The purpose of this additional investigation within the Slurry Pond was to: (i) characterize impounded CCR; (ii) investigate soils underlying the "Chicora Member" stratum; and (iii) investigate the perimeter dikes near soil boring GSB-11, where the soft dike fill was encountered during the previous investigation by Geosyntec. To investigate the material properties of the "Williamsburg Formation Clay", SCI was subcontracted to Geosyntec to advance one boring (SPT-106) to a depth of 100 ft bgs through the Slurry Pond finger dike. To penetrate the "Chicora Member" stratum, SCI switched to a tri-cone rotary wash drill bit once the "Chicora Member" was encountered to reach the target depth. Thin-walled Shelby tubes were advanced using a pitcher sampler to attempt collection of undisturbed samples of the underlying "Williamsburg Formation Clay". The remaining four test borings were advanced by MAD to 20 to 25 ft bgs. Fifteen Shelby tube samples of CCR were collected during these four borings. Three CPT soundings (CPT-119, CPT-120, and CPT-121) were advanced by MAD within the Slurry Pond all of which collected V_s measurements in 5-ft depth intervals. The remaining three CPT soundings (CPT-116, CPT-116A, and CPT-117) were advanced on both sides of GSB-11 to further investigate the dike fill material within this area.

In February and March 2016, Geosyntec mobilized to WGS to further investigate subsurface conditions supporting the design of potential CCR landfills within the existing footprints of the Unit 2 Slurry Pond, Ash Pond A, and Ash Pond B. Additional soil test borings were also advanced in the western half of the South Ash Pond to further evaluate engineering properties of the soft clays previously encountered within the area. As part of this investigation, one soil boring (GSB-11A) was advanced by Terracon using mud rotary drilling techniques. The boring was located adjacent to the soil test boring GSB-11, which was completed by Geosyntec in 2013. Except when Shelby tubes were pushed to collect undisturbed soil samples, SPTs were performed continuously through the perimeter dike soils (i.e., upper 30 ft bgs) and at 5-ft intervals thereafter. Upon completion of the soil test boring, the depth to water level was measured as 8.2 ft bgs prior to abandonment with cement-bentonite grout. Since the measured groundwater depth at the time of boring was inconsistent with historical temporary piezometer measurements, an engineer with Santee Cooper supervised the advancement of a Geoprobe® boring without sampling to 30 ft bgs and installation of a temporary standpipe at this location. Ninety-six hours after the completion of the Geoprobe® boring, the depth to water within the standpipe was measured as 24.4 ft bgs before the temporary standpipe was removed and abandoned.

SUBSURFACE STRATIGRAPHY AND COAL COMBUSTION RESIDUALS

Subsurface Stratigraphy

The subsurface stratigraphy at WGS was developed based on information collected in the recent and historical geotechnical investigations. Boring logs from both the Geosyntec and historical investigations are provided in Attachment 2 to the Safety Factor Assessment Report. The subsurface stratigraphy was described as follows:

- Dike Fill Material: Materials within the dike consist typically of brown to gray, loose to medium dense, silty or clayey, fine to medium sands, and stiff sandy clays. The dike crest is located approximately between elevations 36 and 40 ft National Geodetic Vertical Datum of 1929 (NGVD29), and the dike toe is located typically between 10 to 19 ft NGVD29. The dike fill is typically 20 to 30-ft thick. The perimeter and divider dikes were originally constructed in the late 1978-1979 using excavated foundation materials (excavated to a minimum elevation of 10 ft NGVD29) that were compacted to form the perimeter dikes.

- Foundation Soils: Foundation soils encountered onsite consist typically of brown to gray, clayey sands, silty sands, and poorly graded fine to medium sands with varying amounts of shells. Several clay lenses or thin layers of clay were occasionally encountered in sandy foundation soils. The relative density of the foundation soils ranged from very loose to medium dense. The foundation soils were generally found to be 10 to 30 ft thick. In the majority of borings, the lower 5 to 10 ft of foundation soils consist heavily of shell fragments and shells intermixed with poorly graded to silty sands. This layer was described in the field typically as loose to medium dense, silty to clayey fine sand with interbedded shells.

- Chicora Member: A layer of dense to very dense, partially cemented to heavily cemented shells were encountered beneath the foundation soils during each of the historical and recent investigations. Blow counts in this layer exceeded typically 50 blows per less than 6 in. of advancement. The historical borings indicated that the thickness of this stratum typically ranged from 1 to 10 ft, and was fairly thin in the eastern corner of the Slurry Pond. Both borings GSB-4 and SC-27 were able to penetrate this stratum in this corner by means of a split spoon sampler and without the use of roller drill bits or rock coring methods. SPT-106 was able to penetrate this stratum using a roller drill bit. The PCRA (1999) report referred to this layer as limestone or "shell hash – Coquina – limestone." S&ME (2001) described this layer as "Coquina," a local name used to describe shell hash and partially cemented shells. The geologic map by Doar (2012) indicates this layer can be considered to be "Shell Hash" and refers to the stratum as "Chicora". Due to limited samples recovered from the Geosyntec investigations, Geosyntec primarily relied on the review of the literature listed above and information from the historical investigations and regional geology regarding the geologic interpretation of this layer. Based on these sources, this layer is defined as "Chicora" or "Chicora Member" of the Williamsburg Formation. The term "Chicora Member" is used to refer to this soil unit throughout this Safety Factor Assessment Report.

- Williamsburg Formation Clay: The Williamsburg Formation Clay was encountered beneath the Chicora Member located at top of the overall Williamsburg Formation and described as stiff to very hard, dark gray to black, and medium to high plasticity clay with sand. The Williamsburg Formation Clay has previously been referred to geologically as the "Black Mingo Clay" or "Black Mingo Formation." For the purposes of this Safety Factor Assessment Report, the term "Williamsburg Formation Clay" was utilized throughout the document.

Coal Combustion Residuals (CCR)

CCR have been stored at the WGS since operations began within the Slurry Pond. Several test pits, four soil test borings, and seven CPT soundings were performed by Geosyntec within the Slurry Pond. These materials are described as follows:

- CCR: The Slurry Pond has been utilized to store FGD residuals (off specification gypsum and calcium sulfite) that are not suitable for beneficial reuse without being co-mingled with other CCR. FGD residuals were originally deposited west of the finger dike and are composed primarily of calcium sulfite with some calcium sulfate. On the western side of the finger dike, the material at the surface was observed to be light brown, slightly sandy silt and was overgrown with shrubs and small plants. Deeper CCR encountered approximately 5 ft bgs or below was found to be black, saturated, and silty. On the eastern side of the finger dike, the CCR was found to be light brown, silty sand at the surface that transitions to dark black, sandy silt also around 5 ft bgs. CCR on the east side of the finger dike, while primarily silty, was found to contain a greater percentage of sand-sized particles than the materials on the west side of the dike. On both sides of the finger dike, the light brown colored materials near the surface appeared to be more oxidized than the CCR at deeper depths. FGD residual materials on the eastern side of the finger dike within the Slurry Pond were found to be soft, unconsolidated sediment-type materials that appeared to be sensitive. Soundings within the FGD residuals were terminated when the tip resistance and sleeve friction signatures indicated a material transition into more competent materials (i.e., foundation soils). Based on information provided by Santee Cooper, the FGD residuals on the western side of the finger dike are generally classified as calcium sulfite (FGD residuals generated before the plant started forced oxidation process) and the residual materials on the eastern side of the finger dike are generally classified as calcium sulfate (FGD residuals after the plant started forced oxidation process).

PHREATIC SURFACE INTERPRETATION AND CURRENT WATER LEVELS

As discussed previously, water levels from rotary wash borings located on the dike centerline were taken 24 hours after boring termination. Since the circulation of bentonite slurry during mud rotary drilling could have influenced the measured water level, two hollow stem borings were advanced through the perimeter dike to collect water level measurements. The depth to water in the hollow stem boreholes was recorded after 1 hour and 24 hours to compare with measurements from the mud rotary boreholes and CPT dissipation tests. The CPT soundings were performed with pore pressure measurements from a pore pressure transducer located behind the cone. These signatures were interpreted to locate the phreatic surface at the time of the sounding. Porewater pressure dissipation tests were conducted at several locations and held for 5 to 30 minutes depending on the rate of pore pressure dissipation. The measured phreatic surface level at each location is summarized in Table 2. It is noted that water level measurements in 2013 were utilized to interpret in-situ tests at the time of the site investigation. Current or more recent water levels were utilized for the liquefaction and safety factor assessments within the Safety Factor Assessment Report.

Dike Phreatic Surface

At the time of the 2013 Geosyntec field investigation, the free water level within the Slurry Pond was maintained at 34.25 ft NGVD29. The measured phreatic surface through the Slurry Pond perimeter dikes appeared to vary across the perimeter dikes; thus, the Slurry Pond perimeter dikes were divided into two representative areas (with respect to phreatic surface) for initial analyses (Geosyntec, 2013), and the typical phreatic surface in each area was selected based on data from mud rotary borings, hollow stem auger borings, and porewater pressure dissipation tests. Area 1 is located along the perimeter dikes on the plant side and includes borings GSB-3, GSB-4, GSB-5, GSB-7, and HSA-1. Area 2 is located along the northern to northwestern perimeter dikes of Slurry Pond and includes GSB-8, GSB-10, GSB-11, and HSA-2. The elevation of the phreatic surface estimated based on the 2013 Geosyntec investigation is plotted in Figures 2a and 2b for Areas 1 and 2, respectively. The representative phreatic surface through the dike centerlines in these areas was selected to be at 28 ft NGVD29 and 22 ft NGVD29 for Areas 1 and 2, respectively, and were used to interpret soil boings in during evaluations performed in 2013 (Geosyntec, 2013). These representative phreatic surfaces were selected primarily based on data collected from the hollow stem auger borings, as these values were considered to be reliable measurements. Figure 2c presents the phreatic surface within the Slurry Pond measured during the spring 2013 subsurface investigation.

Free Field Phreatic Surface

Borings located at the perimeter dike toe were abandoned immediately after reaching their target depths and measurements of the depth to water level were not collected. However, CPT pore pressure signatures and dissipation tests consistently showed water levels of 0.5 ft to 5 ft bgs surrounding the perimeter dikes. Along the northwestern and western perimeter dikes, the groundwater table was shown as being near the ground surface at the time of the sounding. The elevated phreatic surface may be explained by the short duration of dissipation tests where excess pore pressures were not able to be fully dissipated to hydrostatic conditions.

Water Levels since 2013 Investigations

The Slurry Pond free water level, which was measured by a staff gauge (W-SW-WSP), has been slowly lowered and managed by the Floating Pump Station since December 2014. The target Slurry Pond operating elevation was set at 18.0 ft NGVD29. Temporary piezometers (PPZ-31 and PPZ-32) installed in June 2014 through the perimeter dike were measured on a monthly basis until they were abandoned. In addition, water levels were measured intermittently from three temporary piezometers (PPZ-33, PPZ-34, and PPZ-34S-1) installed within the Slurry Pond. It is noted that a significant rainfall event (i.e., a 1,000-yr storm) occurred across South Carolina including WGS in early October 2015, which resulted in a rise of free water elevation from 20.0 to 32.0 ft NGVD29 in the Slurry Pond. The Slurry Pond free water level was returned to 19.6 ft NGVD29 by December 2015. The temporary piezometer data indicates that the phreatic surface through the western perimeter dike is approximately 16.0 ft NGVD29 and 12.6 ft NGVD29 when the Slurry Pond is operating with a free water level elevation of approximately 34.3 ft NGVD29 and approximately 19.6 ft NGVD29, respectively. These measurements were considered more representative of long-term phreatic conditions than the 24-hour depth to water measurements from hollow stem auger borings performed in 2013 as equilibrium conditions may not be achieved in 24-hours. Note that borings were interpreted based on the phreatic conditions at the time of the boring or sounding. For the purposes of evaluating liquefaction potential (Attachment 7) and the safety factor assessment (Attachment 8) of the perimeter dikes, the phreatic surface through the western Slurry Pond perimeter dike during steady state conditions corresponding to the "Maximum Normal Storage Pool" (19.6 ft NGVD29) and the "Maximum Surcharge Pool" (35.4 ft NGVD29 as computed in Attachment 1) were selected as 12.6 ft NGVD29 and 16.0 ft NGVD29, respectively. For the eastern Slurry Pond perimeter dikes, phreatic surfaces based on individual CPT soundings and depth to water measurements were utilized to develop the phreatic surface.

SPT AND CPT INTERPRETATION

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

Standard Penetration Test (SPT)

During the SPT, the number of "blows" or impacts from a standard, 140-lb hammer falling 30 inches that is needed to advance the split spoon sampler by 6 inches is recorded over 3 intervals for a total of 18 inches. The blows for the last two 6-inch intervals are summed and this value is referred to as an "N-value". Due to variations in drill rigs, hammer efficiency, and sampling methods, the field or measured value is usually 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) appears to be the most influential. This correction factor may be computed as follows:

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

where:

ER = energy ratio of the SPT hammer.

SCI provided calibration records for the hammer system of the CME-550X used in the borings during the spring 2013 and fall 2013 subsurface investigations based on calibration tests performed offsite on October 9th, 2012 and April 3rd, 2013, respectively. Energy Ratios (ERs) of 89 percent and 88 percent were computed for this drill rig, and the calibration records are provided in Tables 3 and 4. Drilling rig utilized during the 2016 Geosyntec investigation had an ER of 79.3 percent (Table 5). The energy ratios for hammers used in the S&ME (1978) and PCRA (1999) investigations were estimated (or calibrated) by comparing Geosyntec borings with historical borings. The estimation of energy ratio can be found later within this calculation package. Other correction factors were selected based on standard of practice (Idriss and Boulanger, 2008) and are provided in Table 6. N_{60} was computed based on a 4-inch borehole (101.6 mm) and a standard split spoon sampler. Rod length for the C_R conversion factor was selected based on the depth where the SPT blow counts were recorded, while considering a 5-ft stickup from the length of the drilling rod and anvil above the top of the borehole.

N_{60} is usually normalized based on in-situ stress state at the time of boring. The normalized blow count $(N_1)_{60}$ is computed as follows:

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

where:

C_N = stress normalization parameter.

The effective overburden 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 from 0.5 to 0.6 for sands. 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) Interpretation

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 utilized for each sounding.

From CPT sounding data, a Soil Behavior Type Index (I_c) was computed by using the normalized cone tip resistance and normalized sleeve friction ratio. The normalized cone tip resistance (Q) was computed as:

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

where:

Q	=	normalized cone resistance;
q_t	=	corrected cone tip resistance (tsf) (from Equation (5));
σ_{vo}	=	total vertical stress (tsf);
σ'_{vo}	=	effective vertical stress (tsf);
P_a	=	atmospheric pressure (tsf); and
n	=	coefficient dependent on soil type and stress level.

A coefficient, n, of 1 was selected when interpreting each CPT sounding. Furthermore, the normalized sleeve friction ratio was calculated as follows:

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

where:

F	=	normalized sleeve friction ratio;

f_s = measured sleeve friction (tsf);
q_t = corrected tip resistance (tsf) (from Equation (5)); and
σ_{vo} = total vertical stress (tsf).

Finally, the Soil Behavior Type Index (I_c) (Robertson and Cabal, 2012) was calculated as follows:

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

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 3. Additionally, Figure 3 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, and two examples of Geosyntec's interpretation (CPT-5 and CPT-17) are presented in Figure 4.

LABORATORY TESTING PROGRAM

In 2013, Geosyntec subcontracted Excel Geotechnical Testing, Inc. (EGT) of Roswell, Georgia to conduct a geotechnical laboratory testing on select representative and thin-walled Shelby tube samples collected within the dike fill, foundation materials, and FGD residuals. The testing program included index testing (grain size distribution, Atterberg limits, and natural water content tests), shear strength testing, unit weight testing, and specific gravity testing. Furthermore, the index, triaxial strength, and one dimensional (1-D) consolidation tests were performed on select CCR samples collected from Slurry Pond. In 2016, the grain size distribution, Atterberg limits, and CU tests were performed on soil samples collected from GSB-11A. Additionally, natural moisture content tests were performed on each soil sample from GSB-11A. Appendix 1 summarizes the index testing, unit weight testing, and strength testing results from each of Geosyntec's investigation and the prior historical investigations by other consultants, which are discussed further below.

Index Testing

Dike Fill and Foundation Materials

The index testing program on dike fill and foundation soils included thirty-seven grain size distribution tests including hydrometer testing. Grain size testing indicated that dike fill materials typically consist of 50 percent to 60 percent sand-sized particles (smaller than No. 4 sieve, but greater than No. 200 sieve) and 40 percent to 50 percent silt- and clay-sized particles. Foundation materials typically consist ranged between 22.3 percent and 98.8 percent sand, but the majority of tested samples were found to range between 60 percent to 90 percent sand-sized particles. Tested samples from historical investigations indicated similar compositions of soils. Grain size distribution testing results from both Geosyntec and historical investigations of the Slurry Pond are plotted in Figure 5. Additionally, fines content testing was performed on twenty-six samples to supplement grain size testing and support liquefaction potential analyses. Fines content data, including results from grain size distribution testing, were plotted against depth in Figure 6. Several of the tested soil samples from S&ME (1978) were collected from test pits to assess the suitability of material for dike construction. Thus, data collected from those locations were considered more representative of the dike fill materials than foundation materials situated beneath the perimeter dikes. Laboratory results indicate that the dike fill material typically contains greater than 35 percent fines, while foundation materials typically exceed 20 percent fines. In some isolated cases, the foundation materials are relatively clean sands (less than 5 percent fines).

Natural moisture content and Atterberg limit testing were performed on a range of samples. Geosyntec conducted twenty Atterberg limit tests on dike fill and foundation soils in the vicinity of the Slurry Pond. Generally, Atterberg limit tests were not performed on soils that were found in the field to be apparently non-plastic. Three samples collected by S&ME (1978) and two samples collected by PCRA (1999) were found to be non-plastic. A plot of the natural moisture content test results, including historical data, with respect to elevations is provided in Figure 7. These materials have natural moisture contents between 10 percent and 35 percent in general. Results of Atterberg limits (including historical data) are plotted with respect to approximate distance from the bottom of perimeter dike in Figure 8. The clayey sands (observed in dike fill and foundation soils) typically exhibited a plasticity index between 10 and 25, indicating low to medium plasticity.

Three specific gravity tests were performed on Shelby tube samples of dike fill and foundation soil, as part of the shear strength tests. Three tests were performed on dike fill soils; while two tests were performed on foundation materials. Measured specific gravity of these materials ranges from 2.64 to 2.69.

FGD Residuals

The index testing programs performed by Geosyntec on the FGD residuals consisted of grain size distribution tests. Two out of ten samples were obtained from test pit excavation. Eight of these tested samples included hydrometer testing. Grain size distribution test results indicated that the FGD residuals consisted of 0.1 percent to 19.4percent sand-sized particles and 73.5 percent to 99.9 percent silt- and clay-sized particles. Typically, the FGD residuals consisted of 95.4 percent to 99.9 percent fines except for one sample containing 80.6 percent. A plot of the grain size distribution testing results for the FGD residual samples is provided in Figure 9 of this Attachment. Four samples were tested for Atterberg limits. The results from these tests indicated that FGD residuals are non-plastic.

Williamsburg Formation Clay

The index testing program conducted on the Williamsburg Formation Clay consisted of four grain size distribution tests (three of which included hydrometer tests), four Atterberg limit tests, and six natural moisture content tests. Grain size distribution test results indicated that the Williamsburg Formation Clay consisted of 17.2 percent to 25.7 percent sand-sized particles, 32.7 percent to 42.2 percent silt-sized particles, and 34.0 percent to 41.3%percent clay-sized particles. A plot of the grain size distribution test results is included in Figure 5. The natural moisture content and Atterberg limit test results indicated that the Williamsburg Formation Clay had natural moisture contents between 28.4 percent and 45.3 percent, liquid limits between 61 and 70, plastic limits between 30 and 46, and plasticity indices between 24 and 38. Plots of the fines content, natural moisture content, and Atterberg limit test results are provided in Figures 6 through 8.

Total Unit Weight

Dike Fill and Foundation Materials

Total unit weight of the dike fill and foundation materials were measured for three thin-walled Shelby tube samples as part of Geosyntec shear strength testing program. For samples located within the dike, the measured total unit weight ranged from 120 to 132 pcf. The measured total unit weight of soils beneath the dike (predominantly silty sands) ranged from 126 to 130 pcf. An average total unit weight of 128 pcf was computed for these tested samples, one of which was situated within the foundation soils. Furthermore, S&ME (1978) collected both unit weight data of in-situ materials and performed standard Proctor tests on bulk samples (Appendix 1), as on-site clayey sands and sandy clays were compacted to construct the perimeter dikes. Total unit weights of these materials ranged from 111 to 124 pcf. Meanwhile, total unit weights based on the optimum moisture content and maximum dry density ranged from 122 to 131 pcf. Both Geosyntec and S&ME's data are plotted with sample elevations in Figure 10.

FGD Residuals

The dry unit weight and initial moisture content were measured from each of the shear strength tests and consolidation tests for the eight Shelby tube samples collected from the area inside the Slurry Pond. The total unit weight was calculated for each specimen using the measured dry unit weight and initial moisture content. The results indicated that the total unit weight of the FGD residuals ranged from 92.1 to 116.3 pcf, as shown in Figure 10.

Williamsburg Formation Clay

Total unit weight of the Williamsburg Formation Clay was measured by a bulk density test, which resulted in a natural moisture content of 40.5 percent and a total unit weight of 106.6 pcf. In addition, the dry unit weight and initial moisture content were measured as part of the consolidation testing and hydraulic conductivity testing on the sample of the Williamsburg Formation Clay. The total unit weights for the Williamsburg Formation Clay from both tests ranged from 104.0 to 106.6 pcf.

Undrained Shear Strength

Consolidated Undrained (CU) triaxial compression tests were performed on extruded thin-walled Shelby tube samples of the dike fill, foundation soils, and FGD residual materials. CU tests were performed on two samples of the dike fill materials and one sample of the foundation soils. Additionally, eight CU tests were performed on samples of FGD residuals collected using thin-walled Shelby tubes and two reconstituted samples from material collected during test pits (TP-1 and TP-2) in 2013. A description of the CU test and its interpretation is presented herein.

Methodology

During (CU) 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 stress. The initial effective confining stress applied in each test should generally be applied at the in-situ effective overburden stress or greater. A larger overburden stress states compensate for the effect of sample disturbance. The undrained shear strength (S_u) measured in each CU test corresponds to the initial effective confining stress applied to the specimens rather than the in-situ effective overburden stress 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 as explained below.

The undrained shear strength ratio defined as $S_u / \sigma_c^{'}$ 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, and $\sigma_c^{'}$ is the initial effective confining stress applied in the CU test. The calculated $S_u / \sigma_c^{'}$ is then corrected for the overconsolidation effect by multiplying by a factor of $OCR^{0.8}$, if the sample is overconsolidated (Kulhawy and Mayne, 1990). The $S_u / \sigma_c^{'}$, or the corrected $S_u / \sigma_c^{'}$ if soil is overconsolidated, can be applied directly to a slope stability analysis program. The program will calculate the effective stress for each slice and then assign appropriate S_u based on the undrained shear strength ratio.

Dike Fill and Foundation Materials

The undrained shear strength ratio was calculated for each test based on the calculated shear strength from each data point. The calculated undrained shear strength ratios for the dike fill materials are provided in Figure 11. An *OCR* of 1.0 was selected for the stress history. Undrained shear strength ratios were found to range from 2.74 to 0.18 for dike fill materials, where the lowest value was computed at very high confining stresses (i.e., greater than 7,000 psf). Additionally, three CU tests were performed by S&ME (1978) on compacted bulk samples. These tests were analyzed and the calculated undrained shear strength ratios ranged from 3.10 to 0.71, as plotted in Figure 11. The data in Figure 11 show that an undrained shear strength ratio of 0.65 is approximately the lower bound for dike fill materials over the current range of in-situ stresses (i.e., 0 psf to 3,200 psf) within the vicinity of the perimeter dikes.

The thin-walled Shelby tube sample collected within foundation soils were primarily silty sands and experienced cavitation during shearing. The CU test was interpreted for drained shear strength parameters, but was not utilized to evaluate the undrained shear strength due to cavitation.

FGD Residuals

As shown in Figure 11, the undrained shear strength ratio computed for FGD residuals ranged from 53.33 to 0.37 from eight CU test specimens on Shelby tube samples and two CU tests from reconstituted samples collected from test pits (TP-1 and TP-2). Samples were reconstituted from bulk samples by saturating and mixing the material into a slurry. Once the slurry was mixed, it was poured into a large container and allowed to consolidate by self-weight for 10 weeks. After 10 weeks, Shelby tubes were pushed into the container, carefully removed from the container, and extruded into individual test specimens for CU testing. The undrained shear strength ratio was calculated for these samples between 8.55 to 21.34 for the FGD residuals. A value of 0.50 was selected for the undrained shear strength ratio of the FGD residuals.

Drained Shear Strength

Dike Fill and Foundation Materials

The effective stress friction angles (ϕ') and cohesion intercept (c') of dike fill and foundation soils were estimated based on the CU test results. The ϕ' and c' were calculated using the effective stress Mohr's circle at failure for each CU test. The Mohr's circles for dike fill materials are plotted in Figure 12. Effective friction angles for dike fill materials were calculated to be 31.7° and 35.8° with corresponding effective cohesion intercepts of 0.0 and 0.67 psi, respectively. CU tests on foundation materials were performed on Shelby tube samples consisting of primarily silty sands. Two of these test specimens experienced cavitation during testing and were not utilized as part of this evaluation. One specimen of the GSB-8 Shelby tube did not experience cavitation and a friction angle of 32.3° without an effective cohesion intercept was computed.

FGD Residuals:

The Mohr's circles from the CU tests on the FGD residuals are plotted in Figure 14. Computed friction angles of samples ranged from 37.5° to 48.8° considering effective stress parameters at failure. CU triaxial tests conducted on reconsolidated samples collected within test pits (i.e., TP-01 and TP-02) resulted in friction angles ranging from 40.0° to 42.0°.

Consolidation Test Interpretation

FGD Residuals

1-D consolidation tests were conducted on four Shelby tube samples collected from within the Slurry Pond. The preconsolidation pressure (σ_p') estimated from these tests ranged from 1,100 to 1,600 psf. The strain versus the applied load plot for each test is provided in Figure 15. The modified coefficient of compression ($C_{c\varepsilon}$) and modified coefficient of recompression ($C_{r\varepsilon}$) were calculated from each test and ranged from 0.066 to 0.18 and from 0.013 to 0.046, respectively. Each consolidation test on FGD residuals was conducted with an unload-reload cycle. The modified swelling indices ($C_{s\varepsilon}$) computed during this unload-reload cycle ranged from 0.002 to 0.042. Samples collected from shallower depths were observed to be less compressible than samples collected at greater depths. Coefficient of consolidation (C_v) and modified coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) were calculated from each load increment and plotted as a function of σ_v'/σ_p'. Figures 16 and 17 display the C_v and $C_{\alpha\varepsilon}$ results for the FGD residuals.

Williamsburg Formation Clay

A 1-D consolidation test was conducted on one Shelby tube sample of the Williamsburg Formation Clay. The σ_p' estimated from this test was 8,600 psf. The strain versus the applied load plot is plotted in Figure 15. From this test, $C_{c\varepsilon}$ and $C_{r\varepsilon}$ were calculated to be 0.120 and 0.011, respectively. C_v and $C_{\alpha\varepsilon}$ were calculated from each load increment and plotted as a function of σ_v'/σ_p' in Figures 18 and 19, respectively.

Hydraulic Conductivity

One hydraulic conductivity test was performed on a Shelby tube sample of the Williamsburg Formation Clay located below the center of the Slurry Pond (SPT-106). This test indicated a hydraulic conductivity (k) of 4.5 x 10^{-8} cm/s.

IN-SITU TESTING INTERPRETATION

In-situ test results, i.e., SPT and CPT results, were interpreted using empirical correlations to estimate material parameters. The following section describes the methodology and correlations applied to the interpretation.

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 shear wave velocity data can be found within Attachment 3 to the Safety Factor Assessment Report. Furthermore, the field shear wave velocity testing data were supplemented with a correlation with adjacent CPT sounding data. 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 (m/s); and

f_s = sleeve friction (kPa).

Drained Friction Angle

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

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

ϕ' = effective stress friction angle (degrees); and

$(N_1)_{60}$ = stress normalized and energy corrected N-value (blows/ft).

Undrained Shear Strength Ratio

Undrained shear strength ratio as computed by the following correlation was compared with laboratory test data. Undrained shear strength 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 (tsf);

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

N_{kt} varies regionally and by material type with a typical range between 10 and 20, and a value of 15 was selected (FHWA, 2002) the interpretations presented herein.

N_{60} from CPT Soundings

CPT sounding data have been correlated in order to compute an "equivalent N_{60}" profile. This correlation was applied in calibrating Geosyntec borings and CPT soundings with historic borings from the S&ME and PCRA investigations. The correlation from Robertson (2012) is expressed 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.

CALIBRATION BORINGS

Geosyntec defined the investigations at two locations as "calibration borings" for the following purposes: (i) to directly compare soil test boring data with CPT sounding data; and (ii) to compare Geosyntec's investigation data with either S&ME or PCRA's boring data. One "calibration boring" (GSB-5/CPT-17) was located on the dike centerline directly adjacent to a PCRA boring (B-209) and the other "calibration boring" (GSB-2/CPT-5) was located at the toe of the perimeter dikes directly adjacent to an S&ME boring (SC-41). The location of S&ME boring SC-41 was selected because the current field conditions are similar to the field conditions when S&ME conducted their investigation at this location (i.e., a perimeter dike has not been constructed over the location). In S&ME's report (S&ME, 1978), the drilling methodology (i.e., mud rotary versus hollow stem) and SPT hammer calibration were not thoroughly discussed. PCRA used mud rotary method to advance the boreholes, but the SPT-hammer calibration was not provided in the PCRA (1999) report. These "calibration borings" were utilized by Geosyntec to make reasonable assumptions on the energy ratios for the historical SPT data for material parameter selection and liquefaction potential analyses presented in Attachment 7 of the Safety Factor Assessment Report (i.e., the second purpose described above).

Geosyntec Soil Test Boring and CPT Sounding Comparison

Geosyntec qualitatively compared the results of soil test borings and geotechnical laboratory tests with adjacent CPT soundings. Since samples cannot be collected during a CPT sounding, index testing from soil borings was compared with CPT-based SBT_N soil classifications. When compared, the boring log for GSB-5 agrees with correlated classifications from CPT-17 and the boring log for GSB-2 matches well with CPT-5. For instance, index testing results of samples collected from 30 to 36 ft bgs in GSB-5 indicated a sand with less than ten percent of fines content, which matches well with the SBT_N profile for CPT-17.

S&ME Boring Calibration

Descriptions of the boring procedure were not provided in the S&ME boring logs or engineering report (S&ME, 1978). Geosyntec used the N_{60} value (blows/ft) for GSB-2 and compared the data with the back calculated N_{60} value for SC-41 under an assumed set of conditions. It was originally assumed that S&ME advanced their borings by a hollow stem auger method that produced a borehole with a 6-inch diameter borehole and an initial Energy Ratio (ER) of 60 percent of theoretical maximum was assumed. N_{60} values for both Geosyntec and S&ME's borings were plotted with elevations, and the assumed ER and borehole diameter were adjusted until better agreement was met. Figure 20 depicts the N_{60} profile for GSB-2 and SC-41. Additionally, the N_{60} profile for CPT-5 was computed based on Equation 12. The S&ME's boring was calibrated, based on this figure, by increasing the ER of the hammer to 70 percent and adjusting the assumed borehole diameter to 4 inches. It was assumed that S&ME advanced their borings using a rotary wash method, because soils on-site were found to be loose to medium, silt sands, clayey sands, and poorly graded sands, which were anticipated to heave or collapse into the borehole. An ER of 60 percent was found to best match Geosyntec's data, and was subsequently applied in the evaluation of the S&ME SPT results.

PCRA Boring Calibration

The energy calibration for SPT hammer used in the PCRA subsurface investigation was not provided with the PCRA boring logs or reports documenting the subsurface investigation in 1999. Geosyntec used the N_{60} value (blows/ft) for GSB-5 and compared it with the back calculated N_{60} value for B-209, based on an initial assumed ER of 80 percent. PCRA advanced their borings with rotary wash using a side discharge drilling bit that produced a 4-inch diameter borehole. N_{60} values for both borings were plotted with elevations, and the assumed ER was adjusted until the Geosyntec investigation results agree with the N_{60} profile for B-209 in general. Figure 21 depicts the N_{60} profile for GSB-5 and B-209, and CPT-17. The N_{60} profile for CPT-17 was computed based on Equation 12. The PCRA's boring was calibrated by reducing the ER of the hammer to ER = 70 percent.

RECOMMENDED MATERIAL PROPERTIES

The following paragraphs describe the recommended parameters for analysis of the perimeter dikes surrounding the Slurry Pond. Table 7 summarizes the recommended parameters for analysis.

Total Unit Weight

As shown in Figure 10, the total unit weight for dike fill and foundation materials ranges from 115 to 132 pcf. Representative values of 125 pcf, 115 pcf, and 100 pcf were selected for dike fill materials, sandy foundation soils, and clayey foundation soils, respectively, for the safety factor assessment (Attachment 8). Note that the unit weight for clayey foundation soil was selected based on the Shelby tube samples collected for other areas of WGS. These results are provided within other safety factor assessments published within the operating record. The Chicora and Williamsburg Formation Clay unit weights were selected as 130 pcf and 105 pcf, respectively. The total unit weight of FGD Residuals was selected as 95 pcf, which was based on the unit weight measured during CU triaxial testing on Shelby tubes collected from within the Slurry Pond.

Undrained Shear Strength

As shown in Figure 11, undrained shear strength ratios computed from CU tests performed on undisturbed dike fill materials (by Geosyntec) and on recompacted samples (by S&ME) show that S_u/σ_c' asymptote is 0.65 over the maximum current stresses (i.e., 0 to 3,200 psf) experienced by the Slurry Pond perimeter dikes. Since no Shelby tubes were collected from clayey foundation soils, all of the CPTs on the perimeter dike centerline were selected and used to compute the undrained shear strength ratio for fine grained soils (where the I_c is greater than 2.60) using Equation 11 (assuming an $N_{kt} = 15$). The results are plotted with respect to elevations in Figure 22. The undrained shear strength ratios computed by this correlation range from 0.35 to 5.00. A value between 0.35 and 0.40 was recommended based on Geosyntec's CPT results for clayey foundation soils.

Undrained shear strength ratios were estimated from CU tests on the FGD Residuals. These tests were also plotted on Figure 11, and ranged from 3.45 to 0.83. A value of 0.50 was recommended for stability analyses, accounting for the current range of stresses experienced by the FGD residuals.

Drained Shear Strength

For dike fill materials of the Slurry Pond perimeter dikes, Geosyntec selected an effective friction angle of 33 degrees with an effective cohesion intercept of 100 psf (0.69 psi) based on CU triaxial test results. The effective stress strength parameters for dike fill materials are provided in Figure 23. Equation 10 was also applied for both Geosyntec and historical SPT measurements located within silty sands, slightly clayey sands, and poorly graded sands and the calculated effective friction angles were plotted with respect to elevations in Figure 24. Based on this plot and the typical elevation of sandy foundation soils (-5 to 10 ft NGVD29), an effective friction angle of 32 degrees (without an effective cohesion intercept) was recommended for these materials. For foundation soils that are predominantly clayey, an effective stress friction angle of 28 degrees was conservatively selected based a relationship with plasticity index (Figure 25). A plasticity index of 20 was applied to this chart. Use of an effective cohesion intercept c′ of 50 psf is considered appropriate for this stratum.

Dike fill materials observed in GSB-11 were found to be softer/looser than those of adjacent borings and soundings. The split spoon sampler was advanced the full 18 inches with only the weight of the hammer or the weight of rod (i.e., 0 blow count) of the drill rig within the majority of dike fill materials. These results were not included in Figure 24, as these materials were found to be predominantly clay and the applied relationship is applicable for sands. Further evaluation of this isolated area in the vicinity of boring GSB-11 is discussed subsequently within this calculation package.

From CU test results, Geosyntec selected an effective friction angle of 40° without an effective cohesion intercept for FGD Residuals. The effective stress strength parameter data for thin-walled Shelby tube and reconstituted samples of FGD residuals from the Slurry Pond are provided in Figure 26.

Consolidation Parameters

The following section describes the selected consolidation parameters of FGD Residuals and the Williamsburg Formation Clay developed from 1-D consolidation test results. The average $C_{c\varepsilon}$ values of 0.082 and 0.180 were selected for FGD Residuals in the upper 10 ft bgs and below 10 ft bgs of FGD Residuals, respectively. $C_{r\varepsilon}$ values of 0.012 and 0.038 were selected for FGD Residuals in the upper 10 ft bgs and below 10 ft bgs of FGD Residuals, respectively, considering the average of both $C_{r\varepsilon}$ and $C_{s\varepsilon}$ for each test as $C_{s\varepsilon}$ is approximately equal to $C_{r\varepsilon}$. As there is no clear trend between σ_N'/σ_P' less than 1.0 and the ratio greater than 1.0, a representative value of 6.2 mm^2/s was selected for the C_v of FGD Residuals. The C_v values are often difficult to compute for many CCR materials, as they drain rapidly. Representative values of 0.25 percent and 0.47 percent were selected for $C_{\alpha\varepsilon}$ of the FGD Residuals at the σ_N'/σ_P' less than 1.0 and σ_N'/σ_P' greater than 1.0, respectively. Selected C_v and $C_{\alpha\varepsilon}$ values for FGD Residuals are shown in Figures 16 and 17, respectively. The overconsolidation ratio (OCR) for FGD Residuals was selected as 1.0 (i.e., normally consolidated), since these materials were sluiced into the Slurry Pond and have not been externally loaded during the operational life of WGS.

The OCR of the Williamsburg Formation Clay was calculated as 2.7, but conservatively selected to be 2.0 considering only limited information is available. From a single 1-D consolidation test, $C_{c\varepsilon}$ and $C_{r\varepsilon}$ values of 0.120 and 0.011, respectively, were selected for the Williamsburg Formation Clay.

Representative C_v values of 2.80 and 1.00 mm^2/s were selected for σ_N'/σ_P' less than 1.0 and σ_N'/σ_P' greater than 1.0, respectively. Representative $C_{\alpha\varepsilon}$ values of 0.030 percent and 0.200 percent were selected for the stress ratios less than 1.0 and the stress ratios greater than 1.0, respectively. Selected C_v and $C_{\alpha\varepsilon}$ values for the Williamsburg Formation Clay are presented in Figures 18 and 19, respectively.

GSB-11 Evaluation

During the spring 2013 geotechnical investigation, Geosyntec observed very soft clays and very loose, clayey sands within the perimeter dike fill materials exhibiting "weight-of-hammer" SPT measurements at GSB-11. Subsequently in fall 2013, Geosyntec advanced CPT-116, CPT-116a, and CPT-117 within 100-ft of soil test boring GSB-11 to evaluate the presence and lateral extent of this soft/loose zone. Soil behavior index (I_c), effective friction angle (ϕ'), and undrained shear strength ratio (S_u/σ'_v) were evaluated and compared with CPTs advanced through perimeter dikes in other areas during the spring 2013 investigation. A weak zone was not observed in CPT-116, CPT-116a, or CPT-117.

Geosyntec remobilized to WGS to investigate the Slurry Pond perimeter dikes adjacent to GSB-11 with a single soil test boring (GSB-11A) and to inspect the condition of the downstream perimeter dikes. The soil test boring encountered Dike Fill material with measured N-values between 2 and 20 blows per foot with a majority of the soil intervals exceeding 6 blows per foot. A CU test on the dike fill material from GSB-11A resulted in interpreted shear strength parameters of ϕ' =31.7 degrees and c' = 0 psf. Meanwhile, the natural moisture content test results on dike fill soils at GSB-11A ranged generally between 17 percent and 27 percent and were consistent with moisture content test results on soil samples collected from other sections of the Slurry Pond perimeter dikes, i.e., elevated moisture content measurements were not identified within the Dike Fill or Foundation Soils. The water level measured within the temporary standpipe installed in the GSB-11 area was consistent with adjacent historical temporary piezometers, indicating a phreatic surface below the base of the dike fill material.

Representative Profiles for Site Response Analyses

Shear wave velocities profiles, soil plasticity, and unit weight are input parameters for site response analyses presented in Attachment 6 of the Safety Factor Assessment Report. Therefore, three representative profiles were developed for sections of the perimeter dike structures based on the height of the perimeter dikes and the properties of the underlying soils. Shear wave velocity profiles were developed from seismic CPT tests performed in 5-ft depth intervals during select CPT soundings and the correlated V_s (by Equation 9) from CPT sounding sleeve friction (f_s). The raw V_s measurements and interpretation of these measurements are provided in Attachment 3 of the Safety Factor Assessment Report. The locations of these profiles are provided in Figure 27. The developed V_s

profiles (by elevation) are provided in Figures 28 through 30, and summarized within Table 8. Furthermore, selection of the shear wave velocity of the Chicora and Williamsburg Formation Clay strata is discussed in Attachment 6 of the 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

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Table 2. Summary of Water Level Measurements

Boring ID	Method	Location	Ground Surface El.	Depth to Water (24-hr)	Depth of Dissipation Test	Measured Hydrostatic Pressure	Phreatic Surface Elevation
			ft NGVD29	ft bgs	ft bgs	ft	ft NGVD29
HSA-1	Hollow Stem	Centerline	38.51	10.0	-	-	28.51
HSA-2	Hollow Stem	Centerline	38.61	16.0	-	-	22.61
GSB-3	Mud Rotary	Centerline	38.39	11.3	-	-	27.09
GSB-4	Mud Rotary	Centerline	38.66	5.5	-	-	33.16
GSB-5	Mud Rotary	Centerline	38.05	10.0	-	-	28.05
GSB-7	Mud Rotary	Centerline	38.16	8.0	-	-	30.15
GSB-8	Mud Rotary	Centerline	39.19	21.0	-	-	18.19
GSB-10	Mud Rotary	Centerline	38.86	19.5	-	-	19.36
GSB-11	Mud Rotary	Centerline	37.78	21.0	-	-	16.78
GSB-11A	Geoprobe®	Centerline	37.78	24.4[2]	-	-	13.38
CPT-24	Dissipation	Centerline	38.61	-	32.48	8.28	14.41
CPT-24	Dissipation	Centerline	38.61	-	37.41	13.38	14.57
CPT-24	Dissipation	Centerline	38.61	-	42.49	19.29	15.41
CPT-28	Dissipation	Centerline	38.61	-	46.42	22.85	15.03
CPT-31	Dissipation	Dike Toe	6.45	-	13.45	15.23	8.23
CPT-32	Dissipation	Dike Toe	10.68	-	14.93	17.54	13.29
CPT-33	Dissipation	Centerline	38.98	-	42.16	16.38	13.21
CPT-38	Dissipation	Centerline	37.82	-	27.40	2.04	12.46
CPT-38	Dissipation	Centerline	37.82	-	42.16	16.94	12.60
CPT-40	Dissipation	Dike Toe	10.84	-	17.39	15.58	9.03
CPTO-86	Dissipation	Slurry Pond	38.86	-	12.63	8.93	35.16
CPTO-86	Dissipation	Slurry Pond	38.86	-	17.55	13.21	34.52
CPTO-87	Dissipation	Slurry Pond	38.31	-	12.47	9.55	35.39
CPTO-87	Dissipation	Slurry Pond	38.31	-	17.55	14.70	35.46
CPTO-88	Dissipation	Slurry Pond	37.69	-	12.47	5.86	31.08
CPTO-88	Dissipation	Slurry Pond	37.69	-	17.55	9.78	29.92
CPTO-89	Dissipation	Slurry Pond	37.72	-	12.47	4.96	30.21
CPTO-89	Dissipation	Slurry Pond	37.72	-	17.55	9.28	29.45

Notes:

1. Depth to water levels in mud rotary boreholes may not be representative of existing conditions due to borehole collapse or the influence of drilling mud on measured depth to water levels.
2. The water level within GSB-11A was measured within a temporary standpipe approximately 96-hours after completion of the boring.

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

SCI Construction Materials / Non Destructive / Geotechnical / Environmental

SPT HAMMER EFFICIENCY

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

Test Date: 10/9/12
Project No.:
Location: SCI Yard
Drilling Method: Mud Rotary

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

Depth: 40 ft
LE: 43 ft
Blow Count: 1, 1, 2

Blow No.	Energy	Blow No.	Energy
1	0.318	26	
2	0.306	27	
3	0.311	28	
4	0.308	29	
5		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: 0.311 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 89%
Std. Deviation: 0.005 kip-ft

Depth: 45 ft
LE: 48 ft
Blow Count: 10, 15, 15

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

Average Energy: 0.311 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 89%
Std. Deviation: 0.002 kip-ft

Depth: 50 ft
LE: 53 ft
Blow Count: 9, 13, 14

Blow No.	Energy	Blow No.	Energy
1	0.322	26	0.309
2	0.321	27	0.314
3	0.313	28	0.314
4	0.318	29	0.310
5	0.316	30	0.313
6	0.315	31	0.315
7	0.317	31	0.311
8	0.314	33	0.316
9	0.313	34	0.313
10	0.312	35	0.314
11	0.310	36	0.315
12	0.313	37	
13	0.315	38	
14	0.315	39	
15	0.316	40	
16	0.318	41	
17	0.312	42	
18	0.314	43	
19	0.313	44	
20	0.314	45	
21	0.317	46	
22	0.316	47	
23	0.313	48	
24	0.316	49	
25	0.316	50	

Average Energy: 0.315 kip-ft
Max. Rated Energy: 0.350 kip-ft
Efficiency: 90%
Std. Deviation: 0.003 kip-ft

Average efficiency from all tests: 89%

Written by: **J. McNash** Date: **10/11/2016** Reviewed by: **W. Shin/ M. Zhu** Date: **10/11/2016**

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

Table 4. CME-550X Energy Ratio Calibration (provided by Soil Consultants, Inc.) for Fall 2013 Investigation

SPT HAMMER EFFICIENCY

SCI Construction Materials | Non Destructive | Geotechnical | Environmental

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

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

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

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

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

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

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

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

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

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

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

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

Average efficiency from all tests: 88%

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

Table 5. CME-45C Energy Ratio Calibration for 2016 Investigation (Provided by Bridger Drilling Enterprises, Inc. (Carolina Drilling))

SPT ENERGY TESTING

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

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

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

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

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

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

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

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

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

Average Hammer Efficiency: 79.3 %

EFV is method for determining energy

ETR is energy transfer ratio

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

Table 6. 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): $C_S = 1.1$ for $(N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100}$ for $10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3$ for $(N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Table 7. Recommended Material Parameters for Analysis

Material	Total Unit Weight (pcf)[1]	Drained Parameters		Undrained Parameters		Consolidation Parameters [2]				
		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 Material	125	33	100	0.65	100	-	-	-	-	-
Foundation Soils (Clayey)	100	28	0[3]	0.35 to 0.40	100	-	-	-	-	-
Foundation Soils (Sandy)	115	32	0	-	-	-	-	-	-	-
Chicora[4]	130	50	0	-	-	-	-	-	-	-
Williamsburg Formation Clay [5]	105	50	0	-	-	0.120	0.011	0.200	1.00	2.0
FGD Residuals [6]	95	40	0	0.50	-	0.082 / 0.180	0.012 / 0.038	0.470	6.20	1.0

Notes:

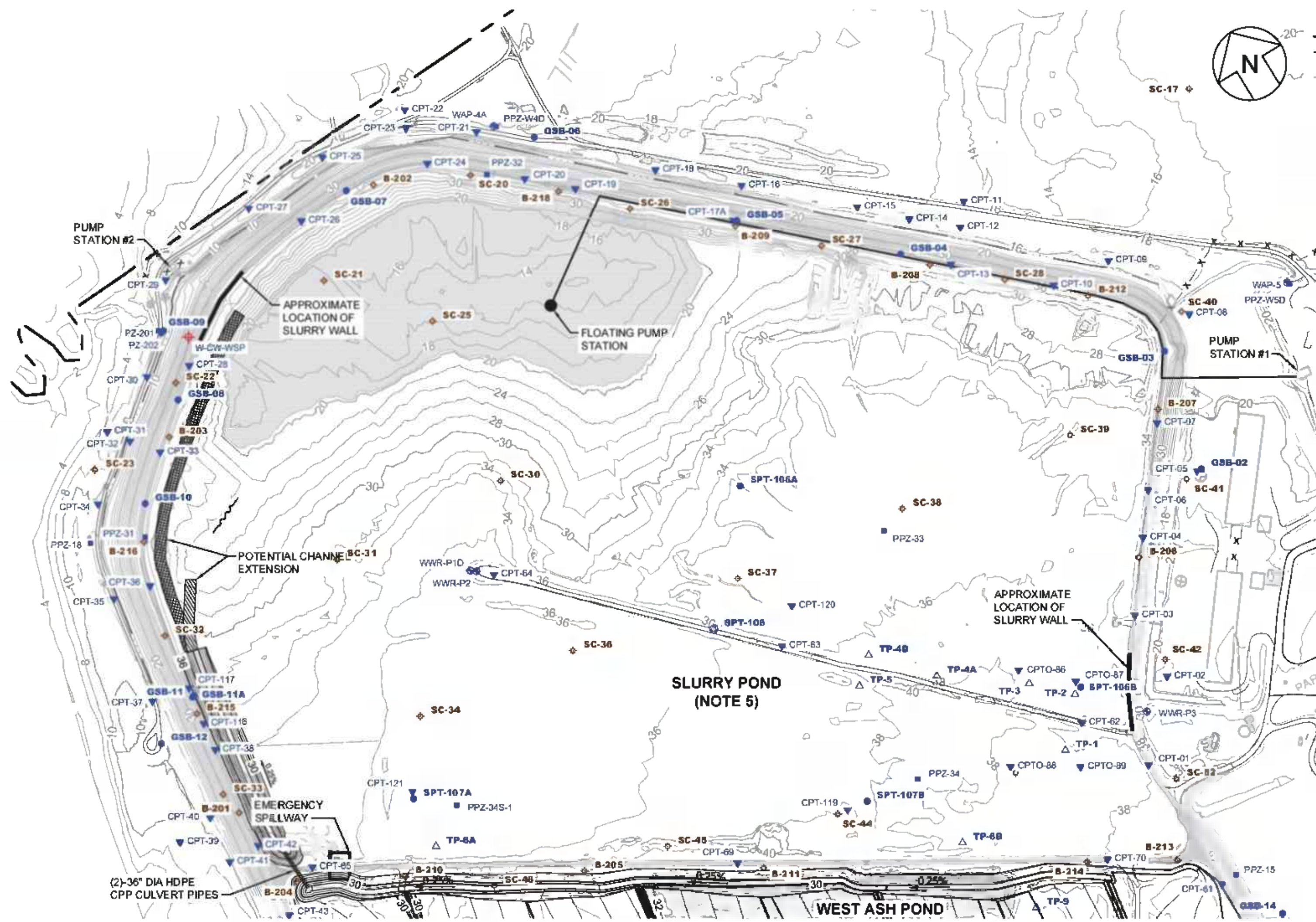
1. Total unit weights presented herein were utilized in the safety factor assessment (Attachment 8). For the liquefaction potential evaluation (Attachment 7), total unit weights were selected based on the I_c.
2. C_v and $C_{\alpha\varepsilon}$ values are provided assuming soils are normally consolidated in situ and additional loading would yield a stress ratio greater than 1.0 (i.e., $\sigma_v' / \sigma_p' > 1.0$).
3. The effective cohesion intercept (c') was conservatively assumed to be zero; however, use of c' = 50 psf is considered appropriate for this stratum.
4. Effective stress parameters for Chicora were assumed due to the cementation and SPT refusal typically observed during site investigations.
5. 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.
6. $C_{c\varepsilon}$ and $C_{r\varepsilon}$ for FGD Residuals varied between samples collected at shallow depths (<10 ft bgs) and larger depths (>10 ft bgs). A value for each of those two cases was selected.

Table 8. Summary of Representative Shear Wave Velocity Profiles

Profile 1 (Dike Centerline)		Profile 2 (Dike Centerline)		Profile 3 (Dike Centerline)	
Elev. (ft)	V_s (ft/s)	Elev. (ft)	V_s (ft/s)	Elev. (ft)	V_s (ft/s)
-12 to -4	700	-12 to -4	650	-12 to -4	650
-4 to 18	750	-4 to 18	700	-4 to 8	700
18 to 38	900	18 to 38	850	8 to 38	800

Notes:
1. Figure 27 depicts the location of each section.
2. Elevations are provided in NGVD29.

FIGURES



LEGEND

30	DESIGN MAJOR GRADE CONTOUR
10	EXISTING MAJOR GRADE CONTOUR
W-SW-WSP	EXISTING STAFF GAUGE
B-201, SC-20	BORING BY OTHERS
CPT-01, CPTO-86	GEOSYNTEC CONE PENETRATION TEST
GSB-02	GEOSYNTEC SOIL BORING
SPT-106	GEOSYNTEC STANDARD PENETRATION TEST
WAP-4A, WWR-P1D	MONITORING WELL
PZ-201, PPZ-15, PPZ-W4D	PIEZOMETER
TP-1	TEST PIT

NOTES:

1. TOPOGRAPHIC SURVEY PROVIDED BY THOMAS & HUTTON DATED 06/29/11 AND REVISED ON 01/14/12.
2. TEMPORARY PPZ-31 AND PPZ-32 WERE INSTALLED IN JUNE 2014 TO MEASURE THE PHREATIC SURFACE WITHIN THE SLURRY POND PERIMETER DIKES. PPZ-31 AND PPZ-32 WERE DESTROYED IN OCTOBER 2015 AND JUNE 2013, RESPECTIVELY.
3. ELEVATIONS FROM THIS SURVEY ARE REFERENCED TO NGVD 1929 DATUM AS DERIVED FROM NGS MONUMENT PID#DD1957.
4. 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.
5. THE FREE WATER LEVEL IN THE SLURRY POND IS MAINTAINED AT AN ELEVATION OF 19.6 FT NATIONAL GEODETIC VERTICAL DATUM OF 1928 (NGVD29) BY A FLOATING PUMP STATION. THE MAXIMUM SURCHARGE POOL WITHIN THE SLURRY POND WAS COMPUTED BASED ON THE INFLOW DESIGN FLOOD (IDF) AS 35.3 FT NGVD29.
6. PZ-201 AND PZ-202 WERE INSTALLED BY PAUL C. RIZZO AND ASSOCIATES (PCRA) IN 1999 AND HAVE SINCE BEEN ABANDONED.
7. CPT-17A REPLACED CPT-17 WHICH EXPERIENCED AN EQUIPMENT MALFUNCTION DURING ADVANCEMENT OF THE LOCATION.

0 300' 600'

SCALE IN FEET

WGS - SLURRY POND
BORING LOCATION MAP

Geosyntec consultants

FIGURE 1

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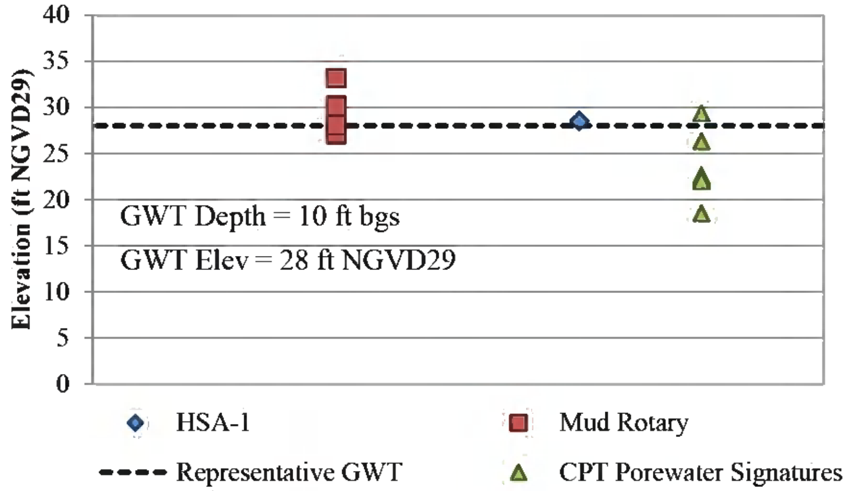

Figure 2a. Measured Phreatic Surface Elevations for Area 1 in 2013
(Area 1 spans from GSB-3 to GSB-7)

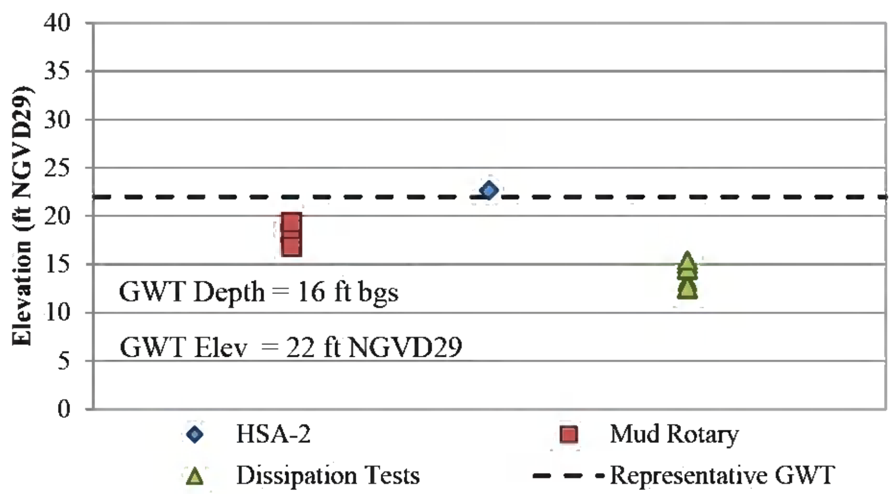

Figure 2b. Measured Phreatic Elevations for Area 2 in 2013
(Area 2 spans from GSB-8 to GSB-11)

Note:

1. Representative Groundwater Table (GWT) was selected based on the measured water levels at HSA-2. Phreatic surface selection for the evaluations within this Safety Factor Assessment Report is based on individual CPT and piezometer measurements.

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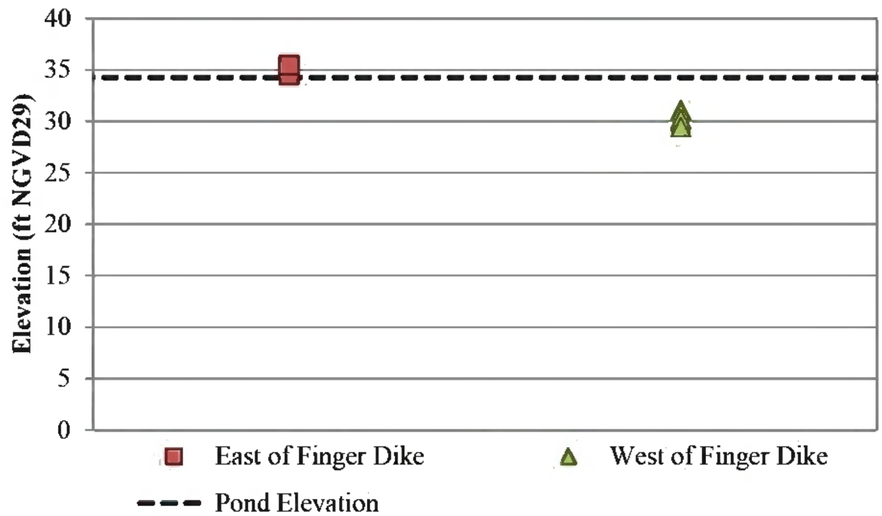
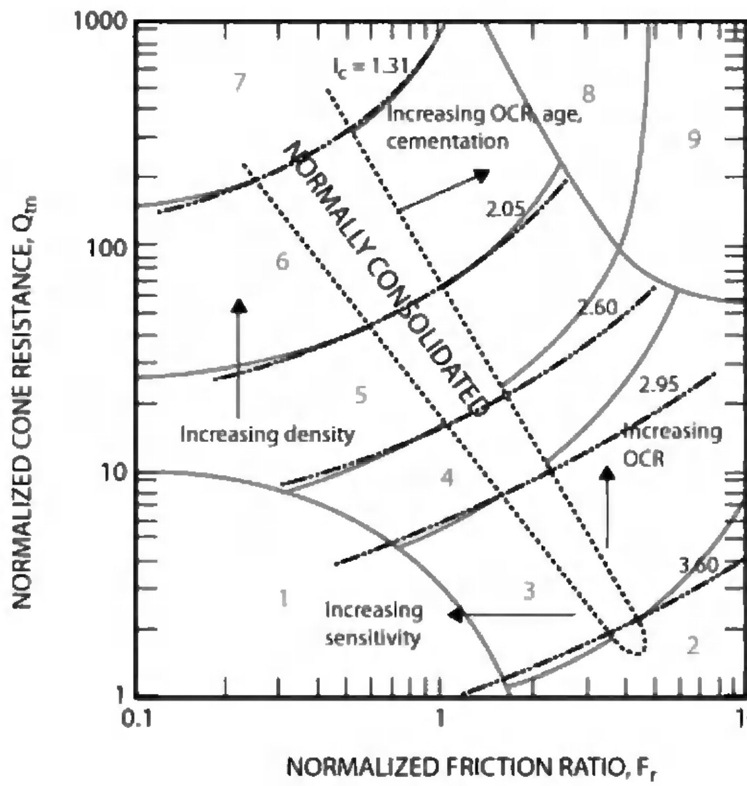


Figure 2c. Measured Phreatic Surface Elevations within Slurry Pond CCR in 2013 (by Dissipation Tests Only)

Note:

1. Pond elevation relates to the surface water level within the Slurry Pond in 2013.



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

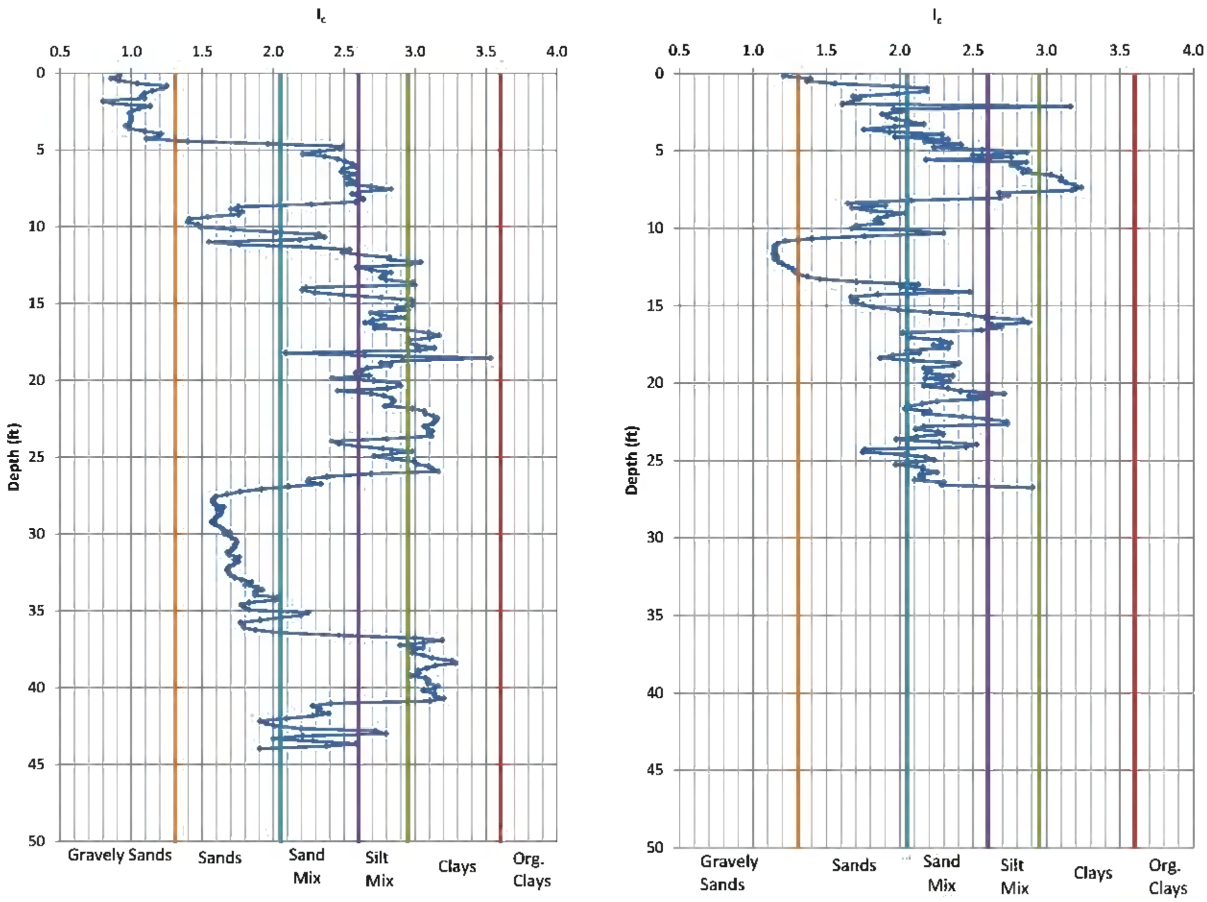


Figure 4. Example SBT_N classification profiles of CPT-17A (Left) and CPT-5 (Right)

Notes:

1. I_c – Soil Behavior Index by Robertson and Cabal (2012).
2. CPT-5 is located at the Slurry Pond Perimeter Dike Toe near the Cooling Towers; while CPT-17 is located through the eastern perimeter dike.

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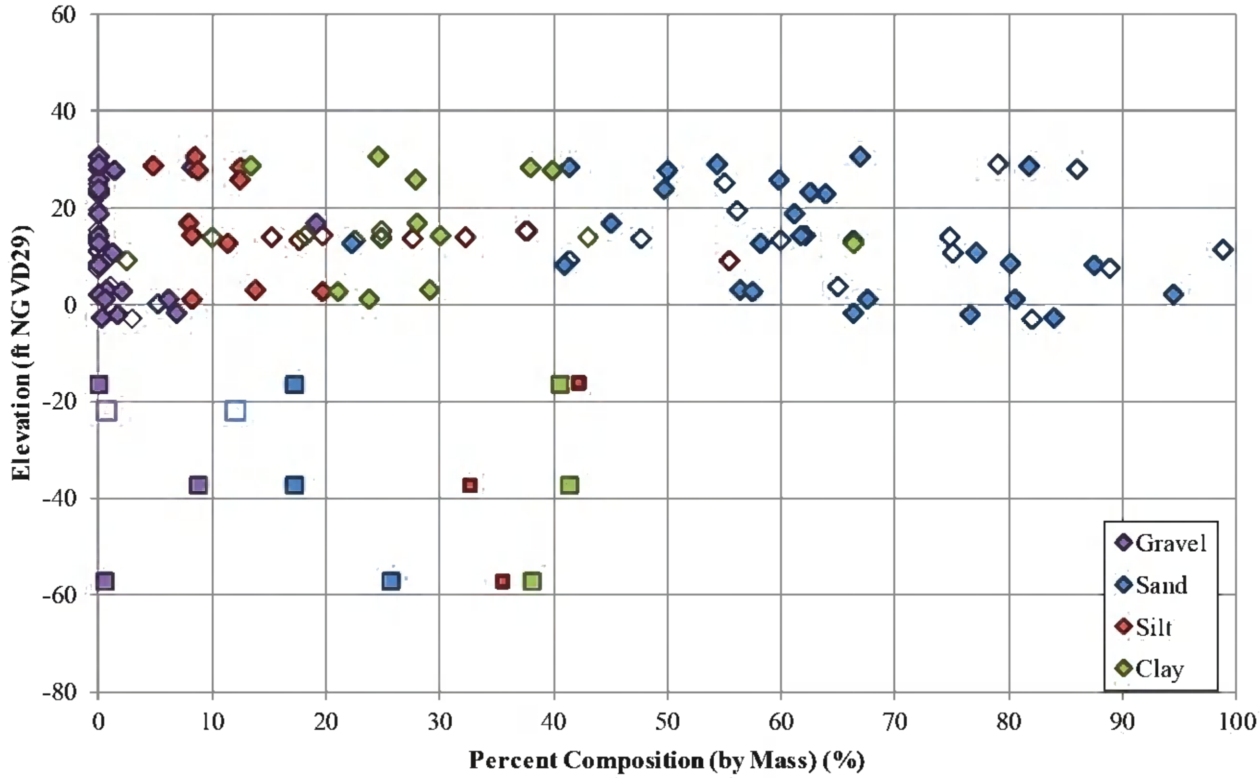

Figure 5. Geosyntec and Historical Grain Size Distribution Test Results

Notes:

1. Diamond points represent data for dike fill and foundation materials. Square points represent the data for Williamsburg Formation Clay.
2. Solid points represent Geosyntec's data; while, hollow points are historical data.
3. The historical grain size distribution test (Elevation -21.9 ft NGVD29) on Williamsburg Formation Clay did not include a hydrometer test. Thus, percent silt and clay sized material was not provided within this figure.

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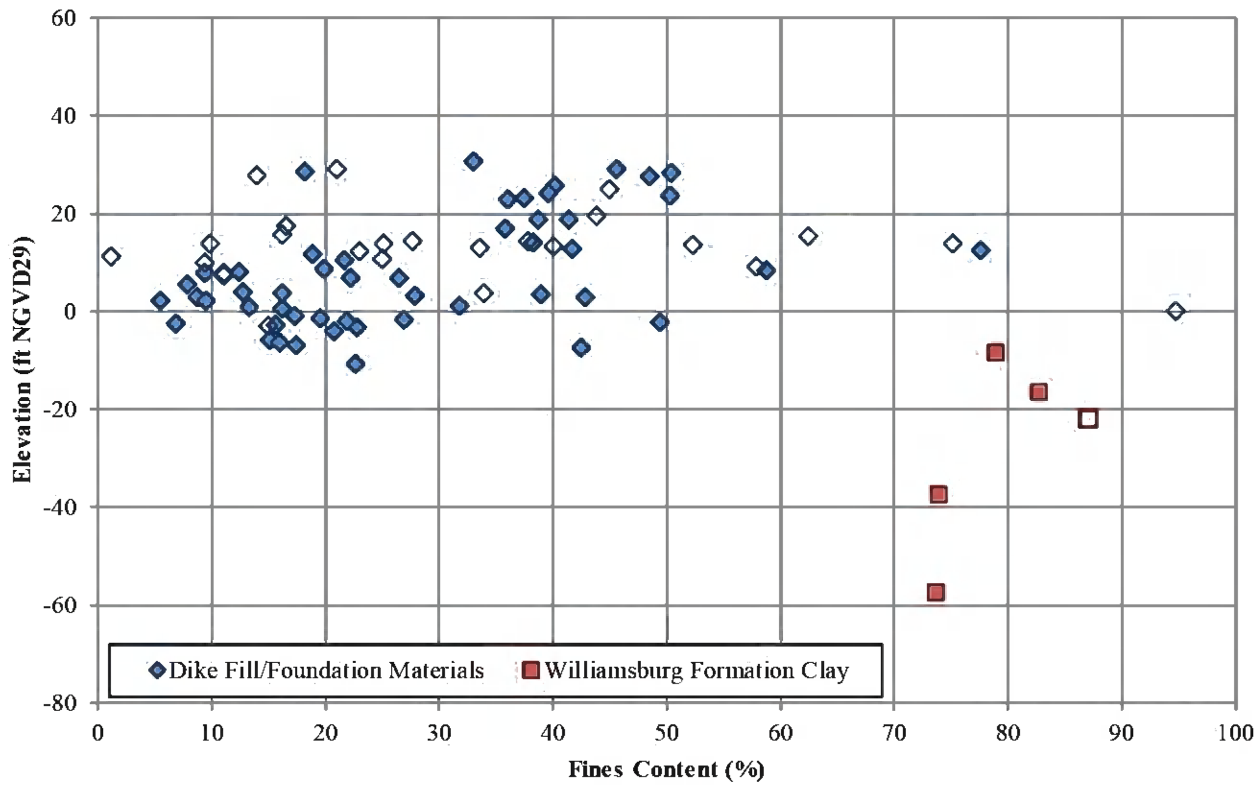

Figure 6. Geosyntec and Historical Fines Content Data

Notes:

1. Diamond points represent data for dike fill and foundation materials. Square points represent the data for Williamsburg Formation Clay.
2. Solid points represent Geosyntec's data; while, hollow points are historical data.

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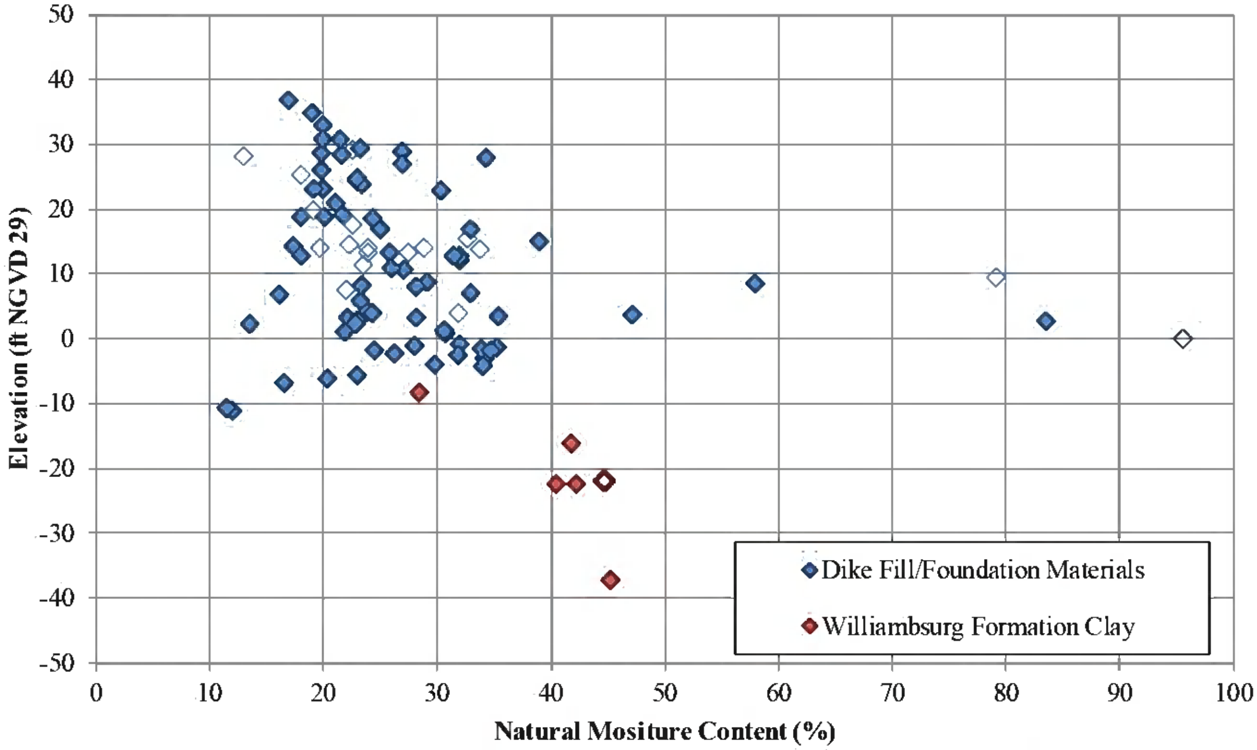

Figure 7. Geosyntec and Historical Natural Moisture Content Data

Note:

1. Solid points represent Geosyntec's data; while, hollow points are historical data.

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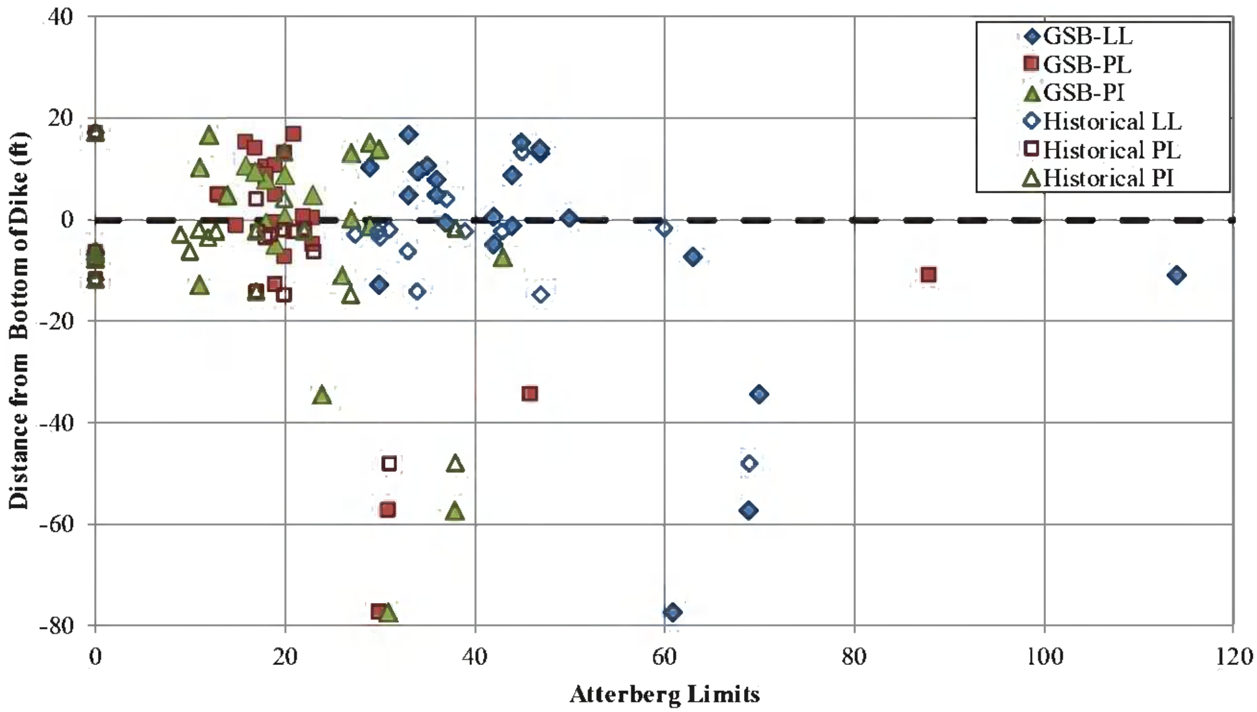

Figure 8. Geosyntec and Historical Atterberg Limit Test Results

Notes:

1. Atterberg limit data was plotted with respect to the distance from approximate bottom of the dike. It is noted that S&ME (historical) data was collected prior to the construction of the dikes. Thus, those data points are typically plotted in the range from elevation 0 to -10 ft NGVD29. It was assumed that these materials were excavated and compacted for dike construction.
2. Non-plastic (NP) samples were plotted as with Liquid Limits (LL), Plastic Limits (PL), and Plasticity Indices (PI) of 0. It is noted that Geosyntec did not typically assign Atterberg limit tests on materials that were anticipated to be non-plastic based on field observations.
3. Index test taken on the Williamsburg Formation Clay are found to be on samples located below -30 ft beneath the dike.

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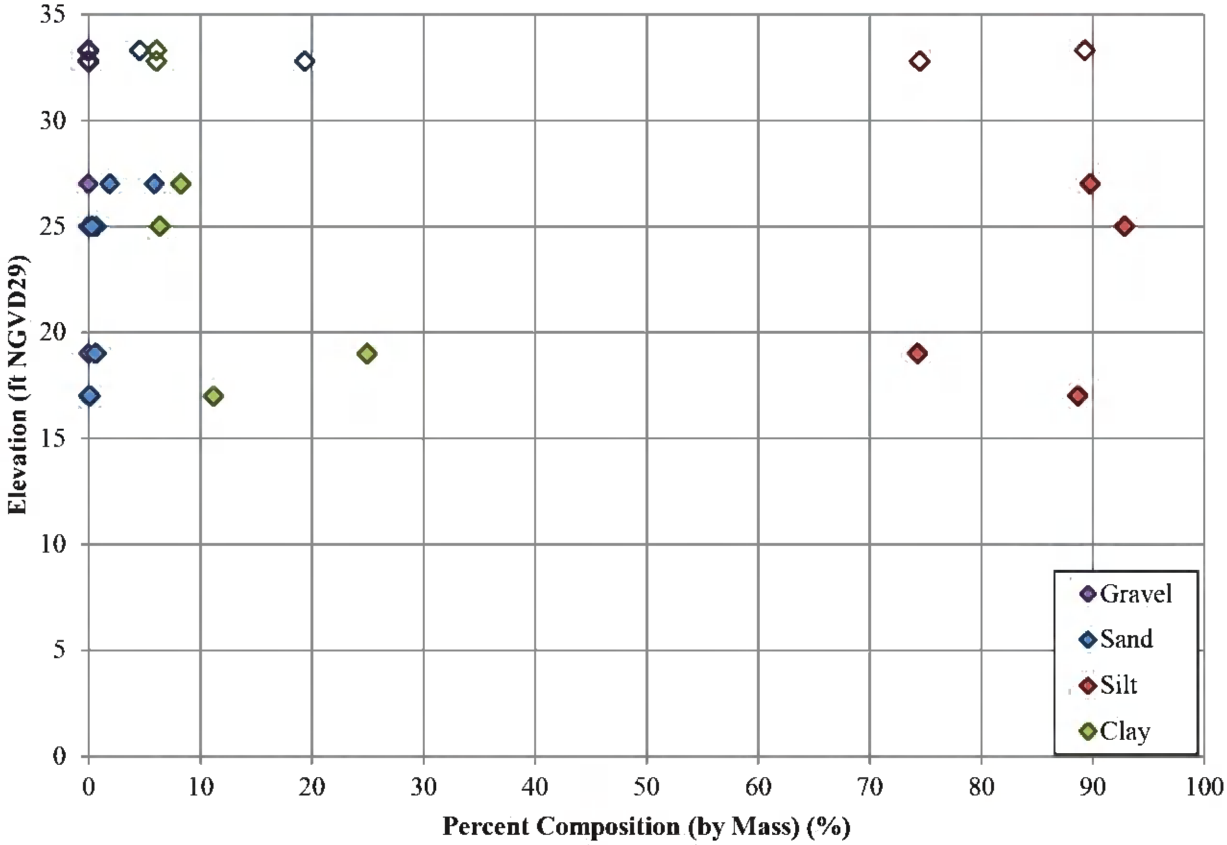

Figure 9. FGD Residuals Grain Size Distribution Test Results

Note:

1. Grain size distribution test results taken from test pits (TP-1 and TP-2) within the Slurry Pond are plotted as hollow diamonds. Grain size distribution results from Shelby tube samples were plotted as solid diamonds.

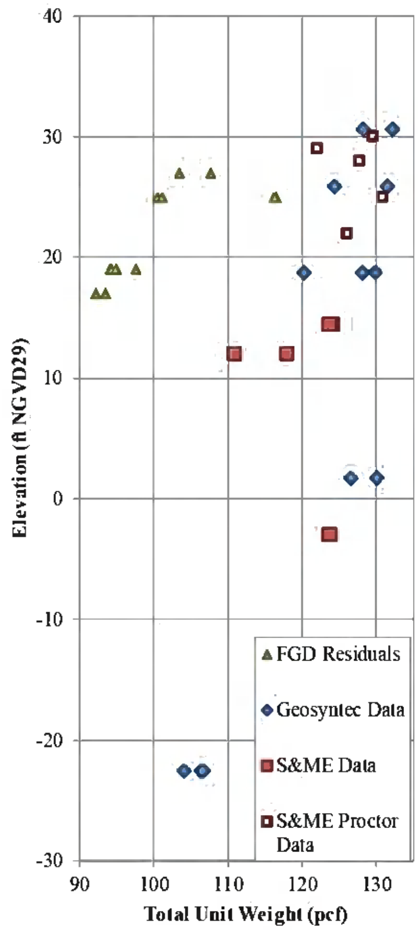

Figure 10. Total Unit Weight Measured during CU and Proctor Testing

Notes:

1. The total unit weight from bulk density tests from the same Shelby tube were averaged together and plotted above. Likewise, total unit weights collected from triaxial tests were similarly averaged and plotted above.
2. The elevations of the total unit weight data points from the S&ME Proctor tests were assumed.

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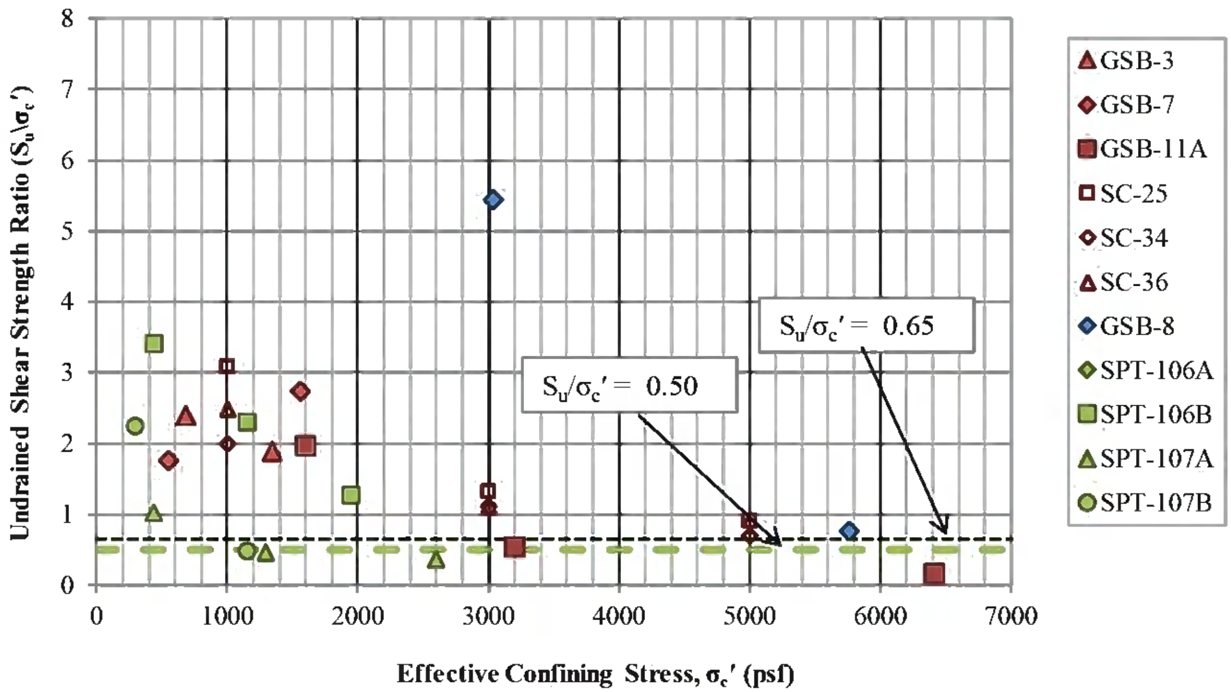

Figure 11. Undrained Shear Strength Ratio from CU tests

Notes:

1. GSB-3, GSB-7, and GSB-11A are CU triaxial tests taken from the dike fill (clayey sand to sandy clay) materials within the Slurry Pond perimeter dikes and are represented in red. CU tests on foundation soils (GSB-08) within the vicinity of the Slurry Pond are represented in blue. Several CU tests on foundation soils experienced cavitation, and thus were considered unreliable; and therefore not included in the undrained shear strength interpretation.
2. Test results for FGD Residual materials (SPT-106A/SPT106B/SPT-107A/SPT-107B) collected from within the interior of the Slurry Pond are represented in green. It is noted that SPT-106A results indicated undrained shear strength ratios were greater than 10, and thus was not plotted in the figure.
3. SC-25, SC-34, and SC-36 are CU triaxial tests on recompacted bulk samples of silty/clayey sands collected from bulk samples during the 1978 investigation by S&ME. These materials were used in dike construction.
4. Representative values of S_u / σ'_c were selected based on the current overburden stress state of the Slurry Pond Dike Fill and FGD Residual materials.
5. Reconstituted samples of FGD Residuals (TP-1 and TP-2) were computed with undrained shear strength ratios greater than 8 and were not plotted within Figure 12.

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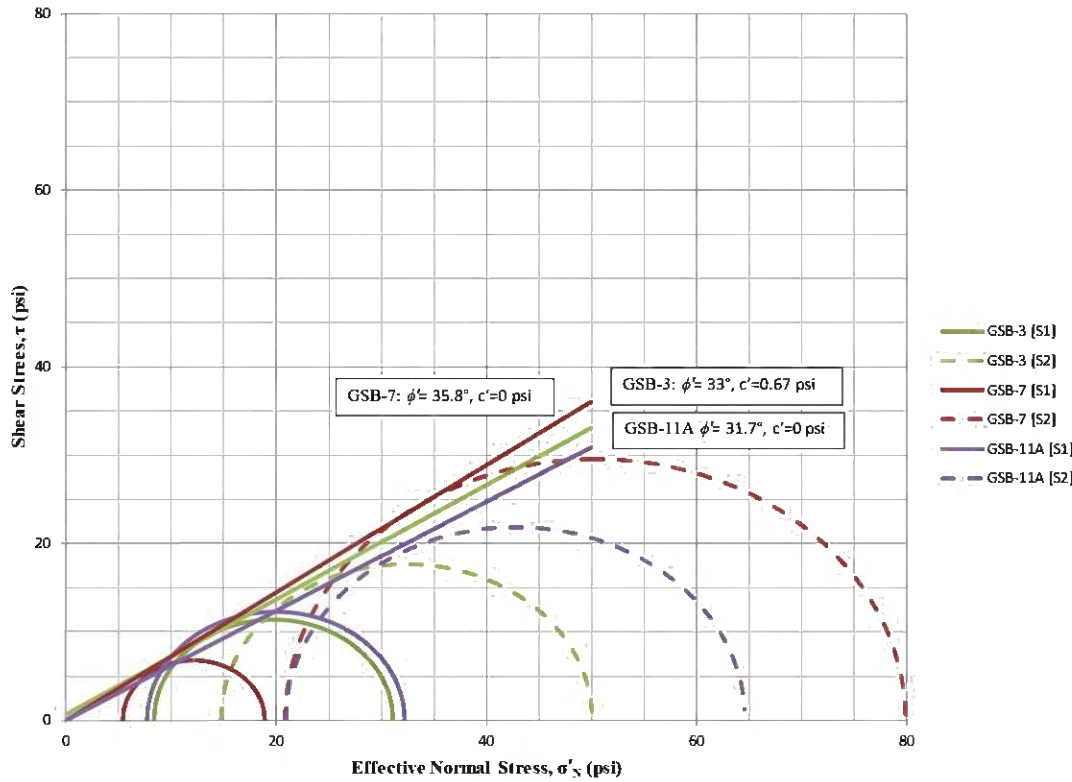

Figure 12. Mohr's Circles from the CU Tests on Dike Fill Materials

Note:

1. Samples from GSB-3, GSB-7, and GSB-11 were collected from 11.5 to 13.5 ft bgs, 6.5 to 8.5 ft bgs, and 18 to 20 ft bgs, respectively.

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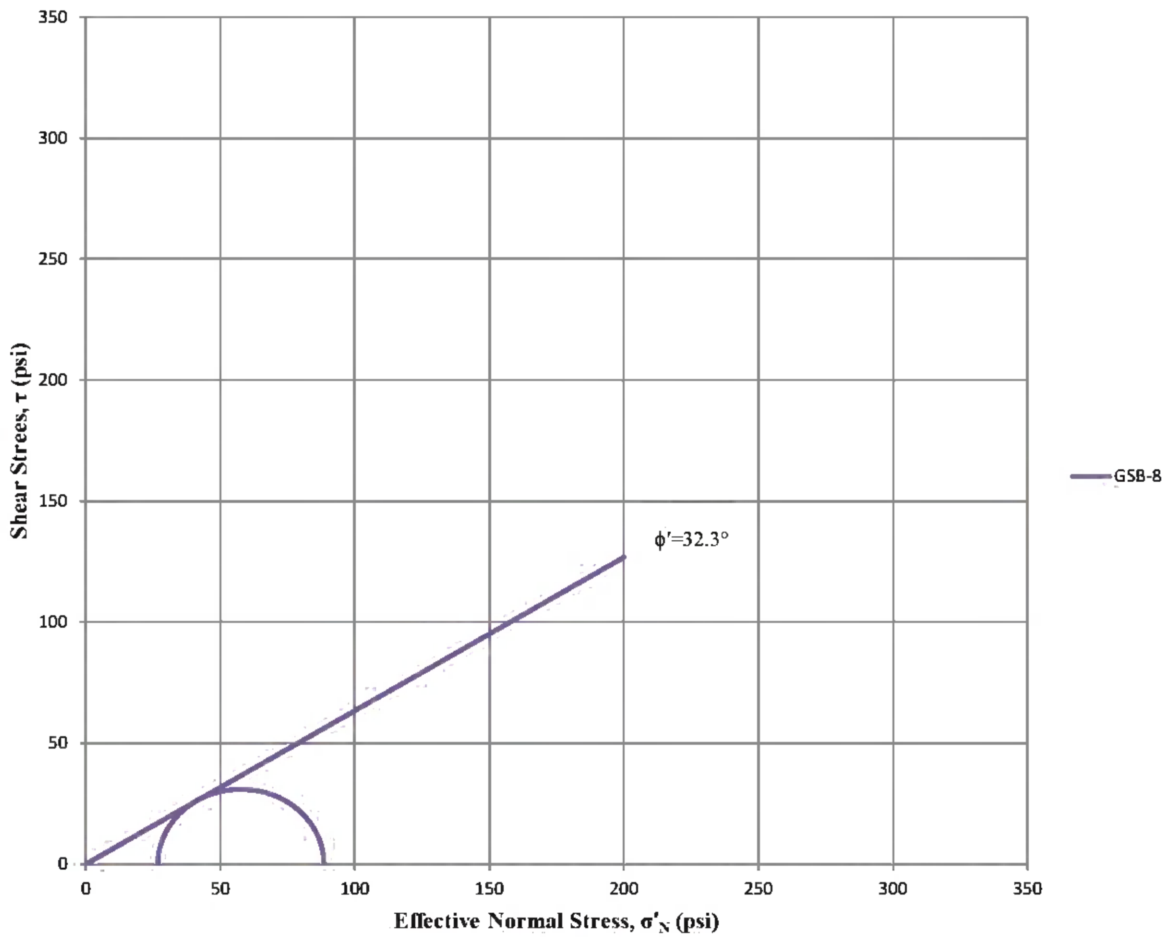

Figure 13. Mohr's Circles from the CU Tests on Foundation Soils (GSB-8)

Notes:

1. Only one collected specimen on sandy Foundation Soils did not experience cavitation during CU testing. This CU test results were supplemented with in-situ testing results during selection of drained strength parameters.
2. The tested sample from GSB-8 was collected from 36.5 to 38.5 ft bgs.

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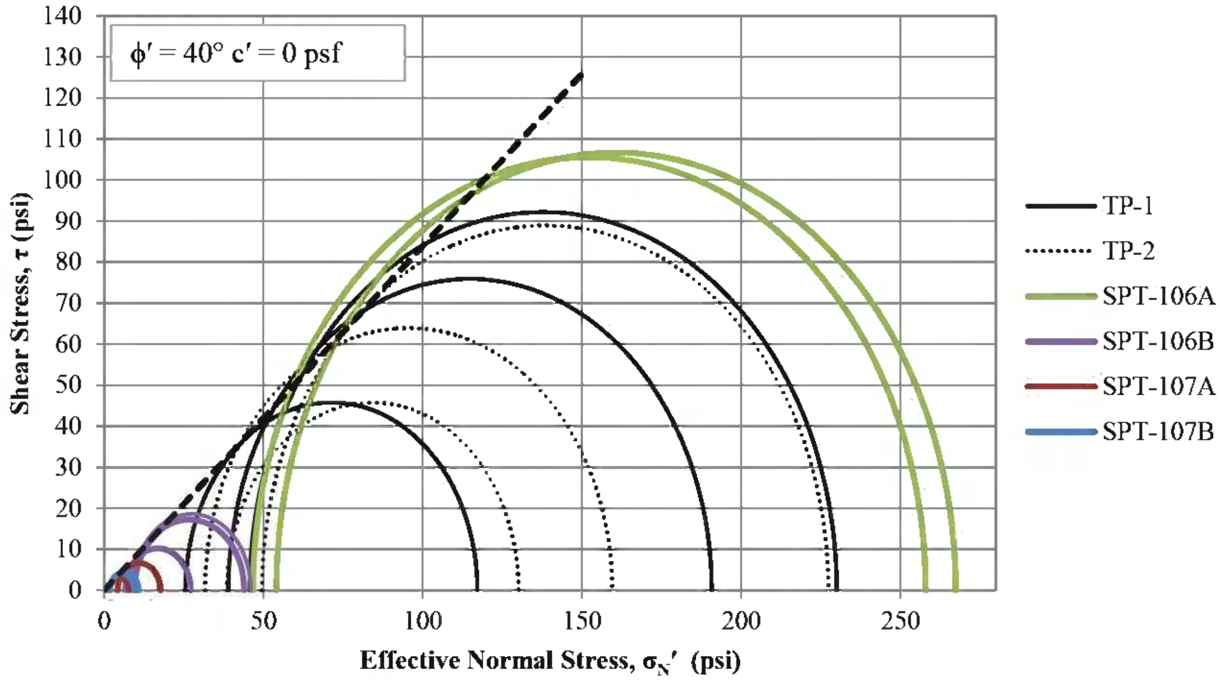

Figure 14. Mohr's Circles from the CU Tests on FGD Residuals

Notes:

1. Samples from SPT-106A, SPT-106B, SPT-107A and SPT-107B consisted of FGD Residuals. SPT-106A: S-3, SPT-106B: S-1; SPT-107A: S-1 and SPT-107:S-3, were collected 16 to 18 ft bgs, 8 to 10 ft bgs, 8 to 10 ft bgs, and 16 to 18 ft bgs, respectively.
2. TP-1 and TP-2 refer to tests performed on reconstituted samples collected from tests pits.

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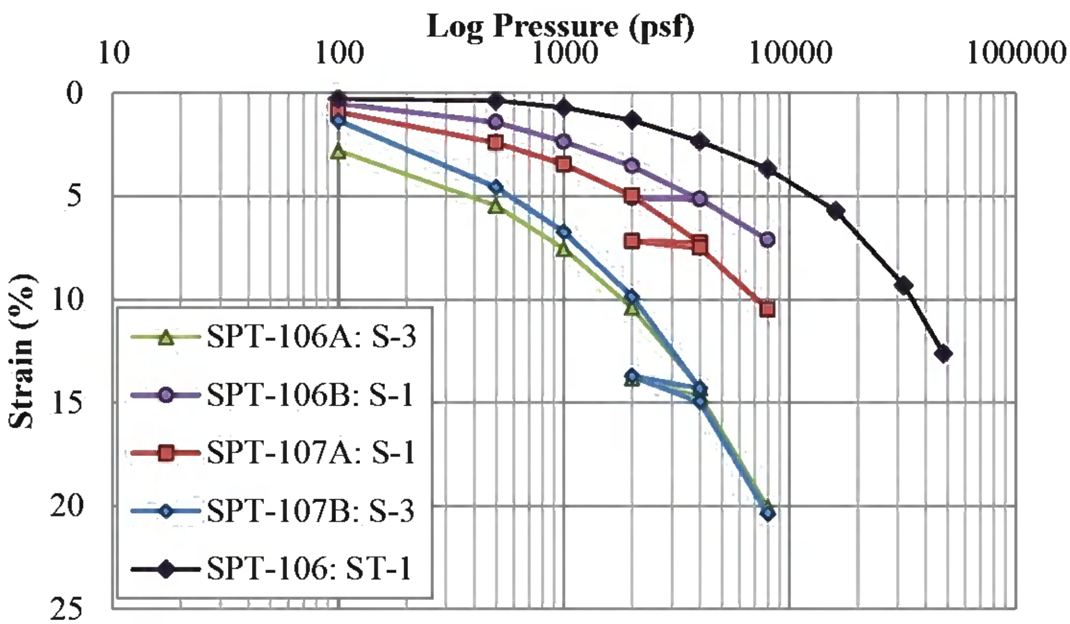

Figure 15. Load-Strain Curves from the 1-D Consolidation Tests on FGD Residuals and Williamsburg Formation Clay

Notes:

1. Samples from SPT-106A, SPT-106B, SPT-107A and SPT-107B consisted of FGD Residuals. SPT-106A: S-3, SPT-106B: S-1; SPT-107A: S-1 and SPT-107: S-3, were collected 16 to 18 ft bgs, 8 to 10 ft bgs, 8 to 10 ft bgs, and 16 to 18 ft bgs, respectively.
2. The test on SPT-106 was conducted on Williamsburg Formation Clay. Sample SPT-106: ST-1 was collected 60 to 62 ft bgs.

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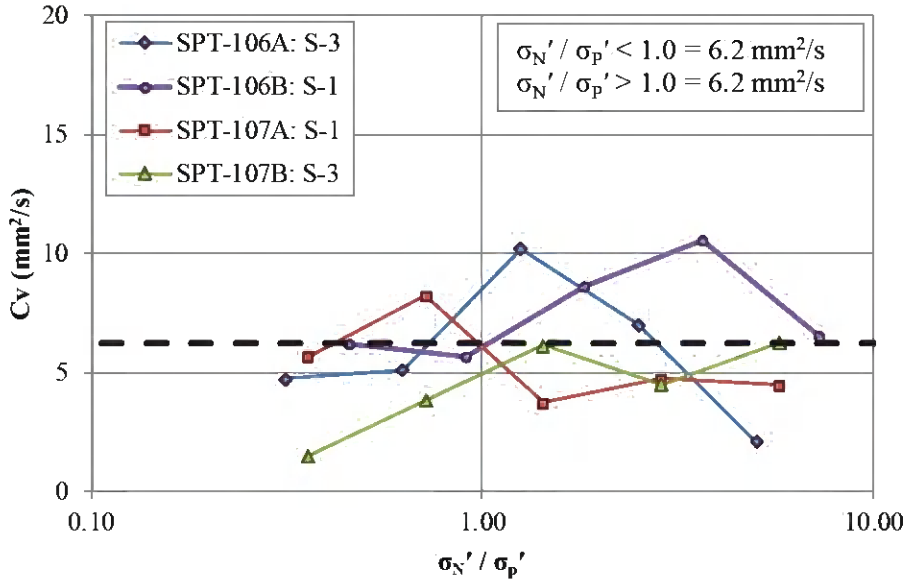

Figure 16. Coefficient of Consolidation (C_V) for FGD Residuals

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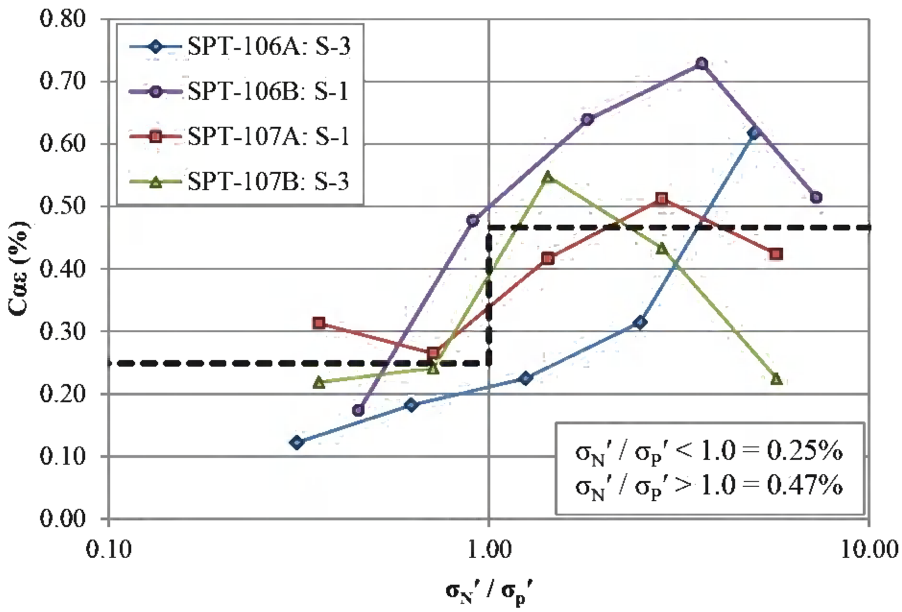

Figure 17. Secondary Compression Ratio ($C_{\alpha\varepsilon}$) for FGD Residuals

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

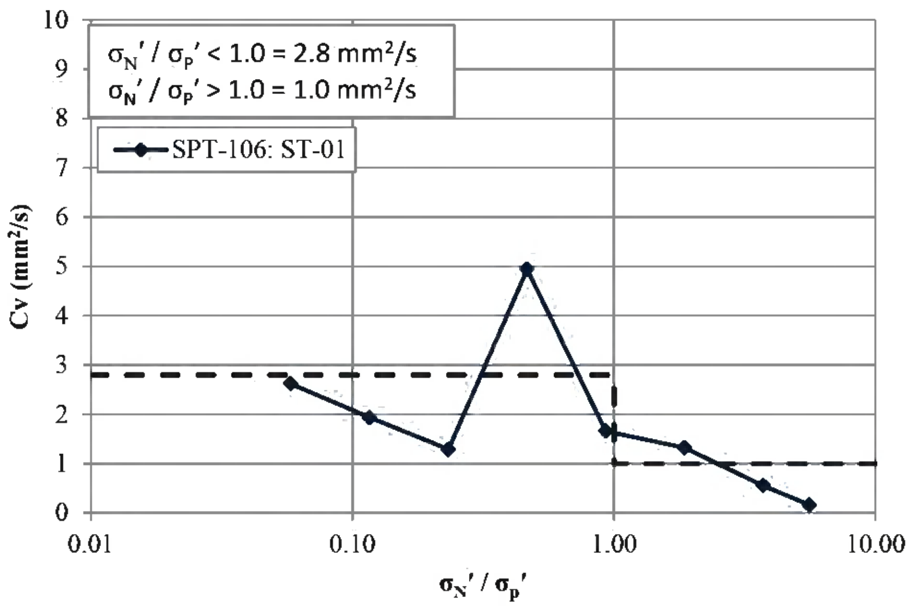

Figure 18. Coefficient of Consolidation (C_V) for Williamsburg Formation Clay

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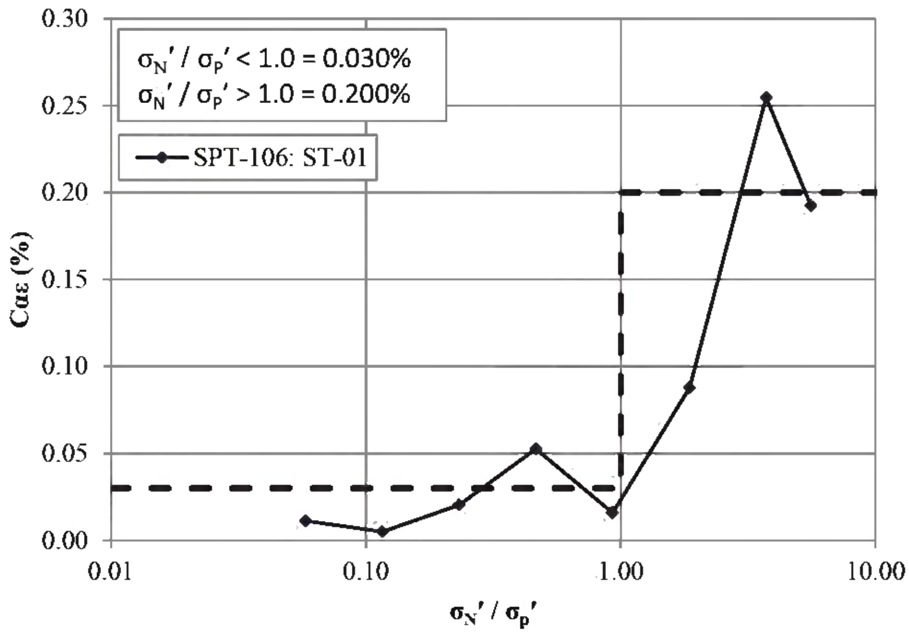

Figure 19. Secondary Compression Ratio ($C_{\alpha\varepsilon}$) for Williamsburg Formation Clay

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

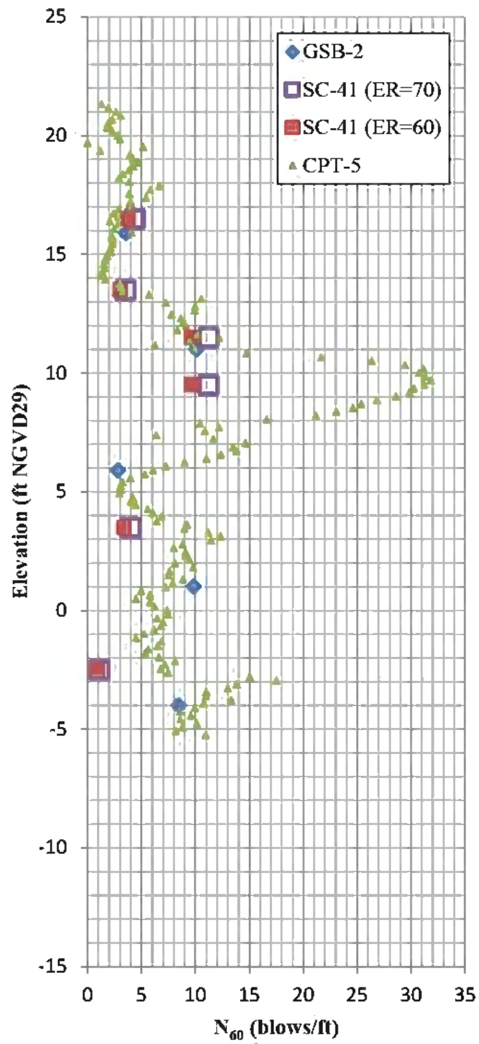

Figure 20. Geosyntec-S&ME Calibration Boring

Note:

1. A computed N_{60} – profile is provided for CPT-17, which is directly adjacent to GSB-2 and SC-41.

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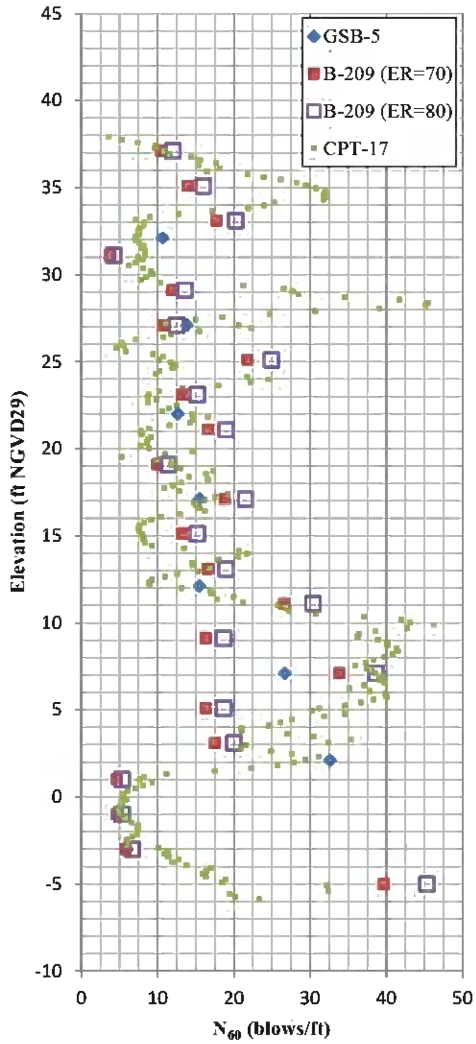

Figure 21. Geosyntec-PCRA Calibration Boring

Notes:

1. A computed N_{60} – profile is provided for CPT-17, which is directly adjacent to GSB-5 and B-209.
2. It is noted that as-built locations from historical borings were not available, but were approximated from available figures. Thus, CPT-17 and GSB-15 may not be positioned exactly at B-209.

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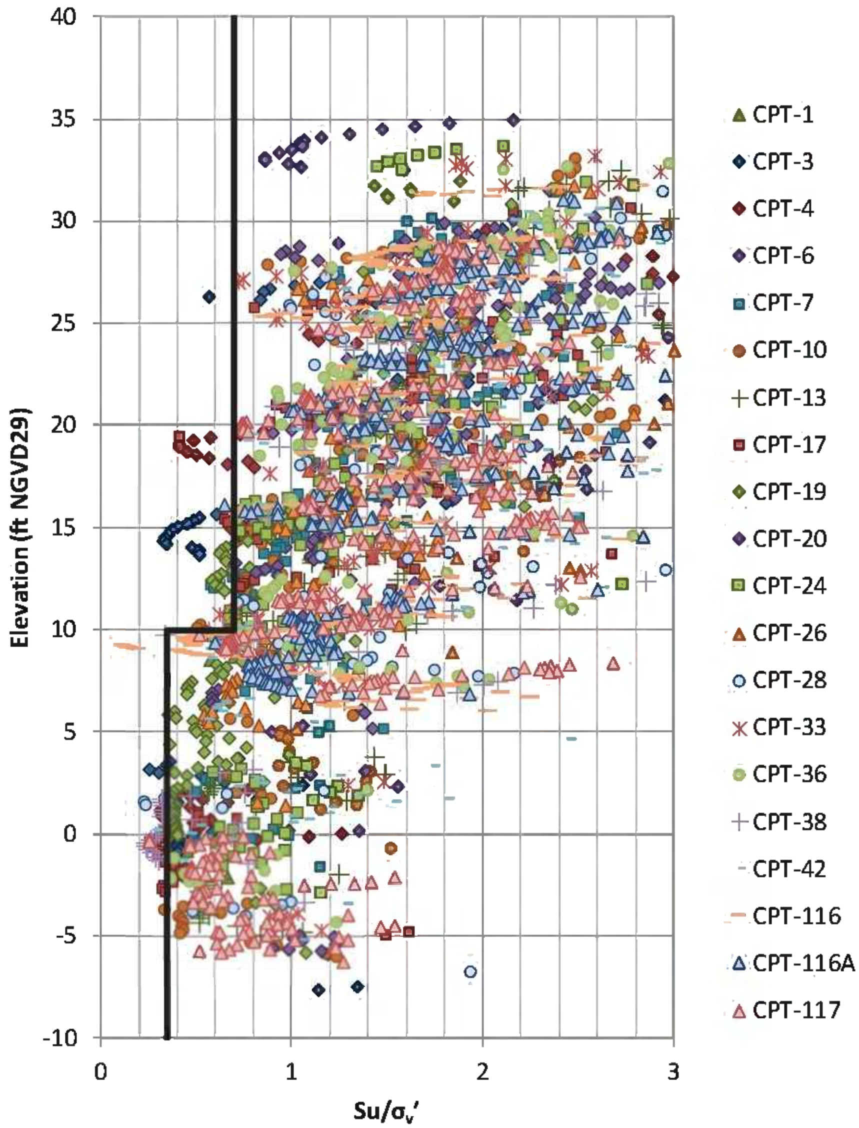

Figure 22. Undrained Shear Strength Ratio Estimated from Dike Crest CPT Soundings

Note:

1. Values S_u/σ'_{vo} between 0.35 and 0.40 were selected for clayey foundation soils during the safety factor assessment depending on the location of the evaluated cross sections.

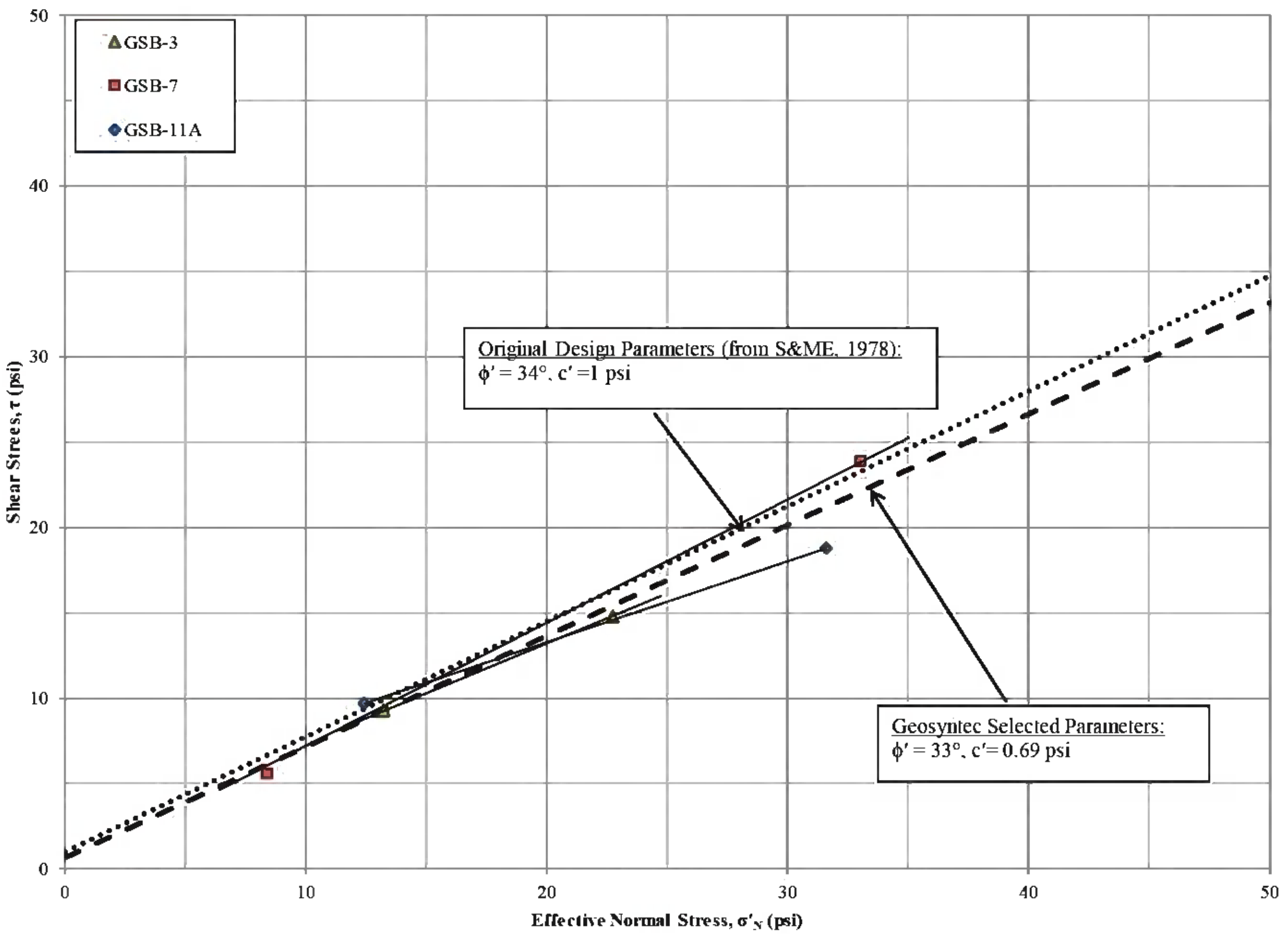

Figure 23. Effective Stress Strength Parameters for Dike Fill Materials

Note:

1. During the design of the Slurry Pond, S&ME selected $\phi' = 34°$ and $c' = 1$ psi for perimeter Dike Fill soils which is shown within this figure for comparison.

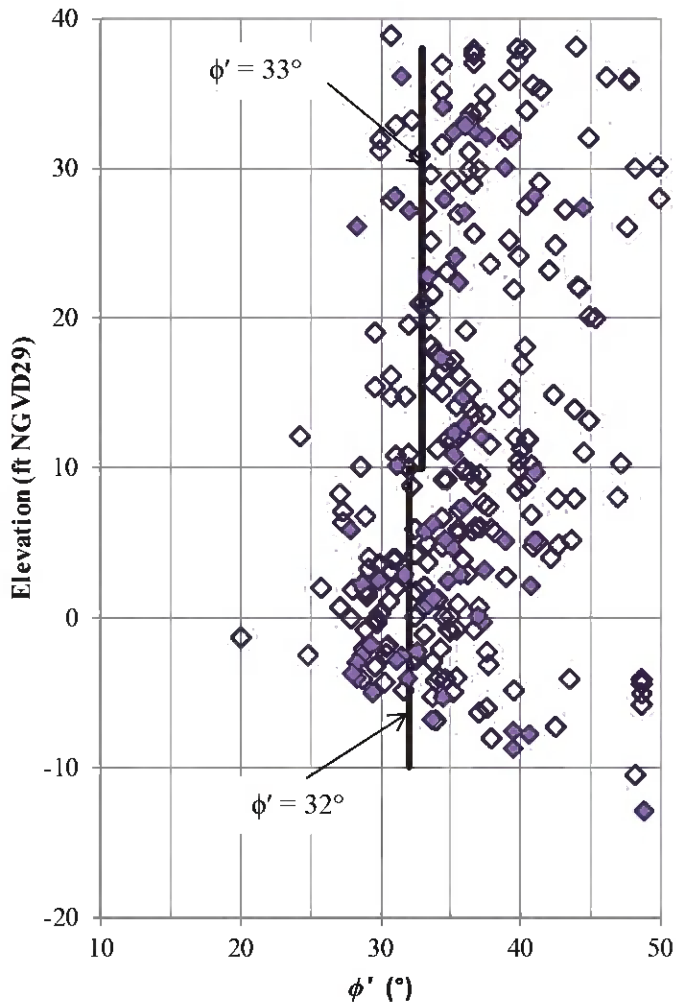

Figure 24. Effective Friction Angle from SPT-based Correlation

Notes:

1. Historical data points were plotted as hollow points, while Geosyntec data points were plotted as solid points.
2. Effective friction angle was plotted for materials classified as sands only.

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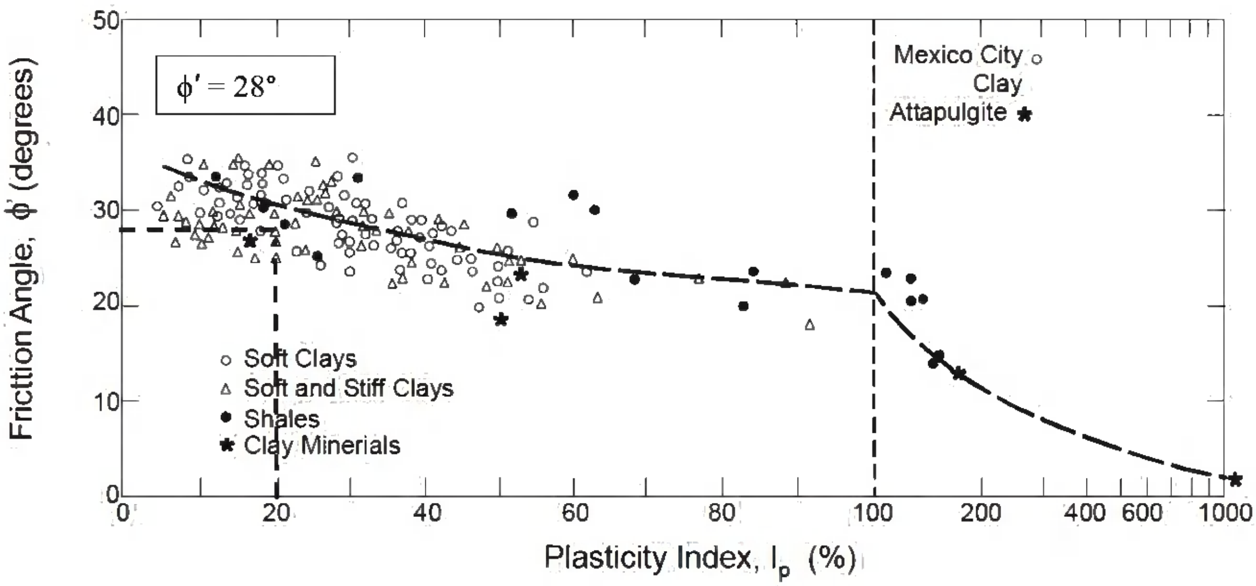

Figure 25. Effective Friction Angle for Clayey Foundation Soils from FHWA (2002) (originally by Terzaghi, Mesri, Peck, 1996)

Note:

1. While most foundation soils are classified as silty to clean sands, several soundings encountered clayey materials. With a Plasticity Index of 20, a lower bound effective friction angle for zones of more clayey foundation soils was selected as 28 degrees. A c' of 50 psf was considered appropriate for clayey foundation materials.

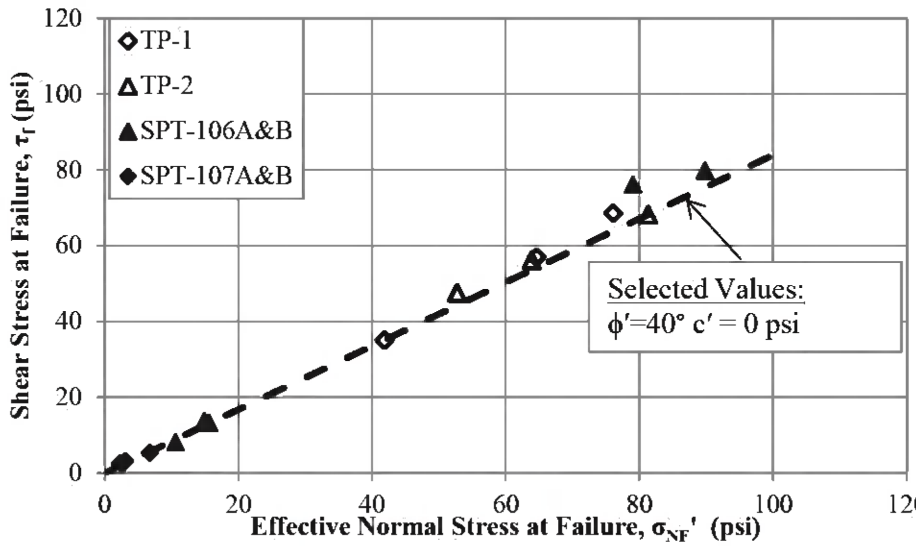

Figure 26. Failure Envelope Estimated from CU Testing for the FGD Residuals

Written by:	J. McNash	Date:	10/11/2016	Reviewed by:	W. Shin/ M. Zhu	Date:	10/11/2016
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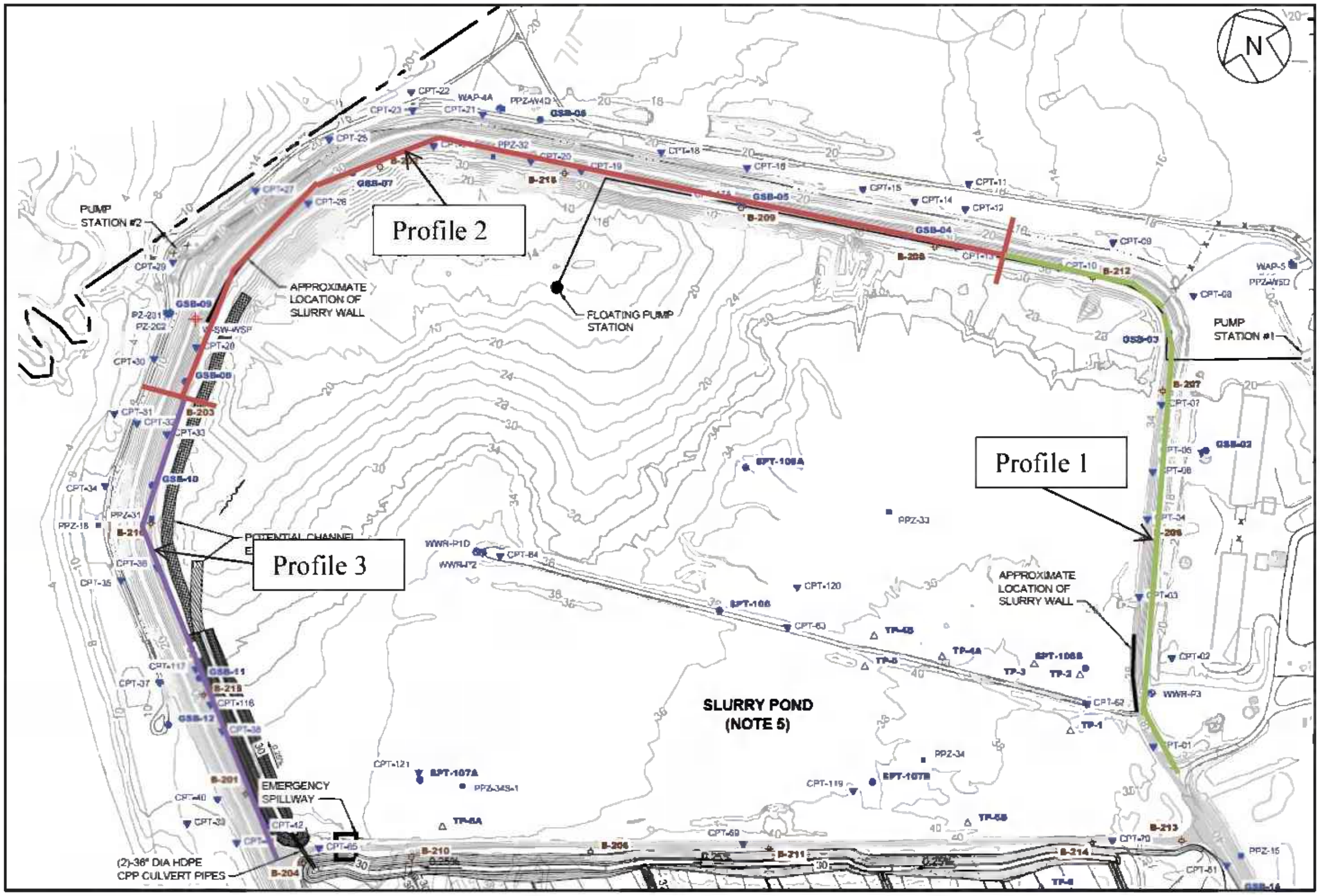

Figure 27. Location of Shear Wave Velocity Profiles

Written by:	J. McNash	Date:	10/11/2016	Reviewed by:	W. Shin/ M. Zhu	Date:	10/11/2016

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

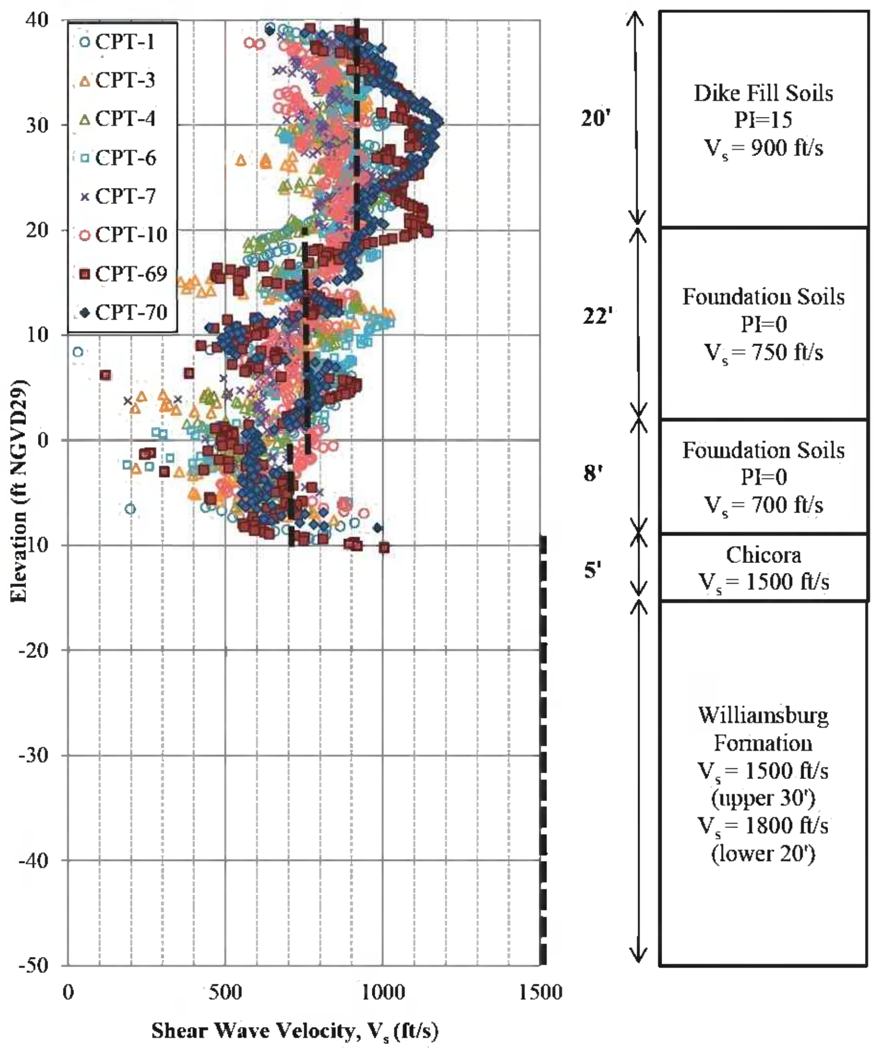

Figure 28. Representative Shear Wave Velocity Profile 1 (Dike Centerline)

Note:

1. 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.

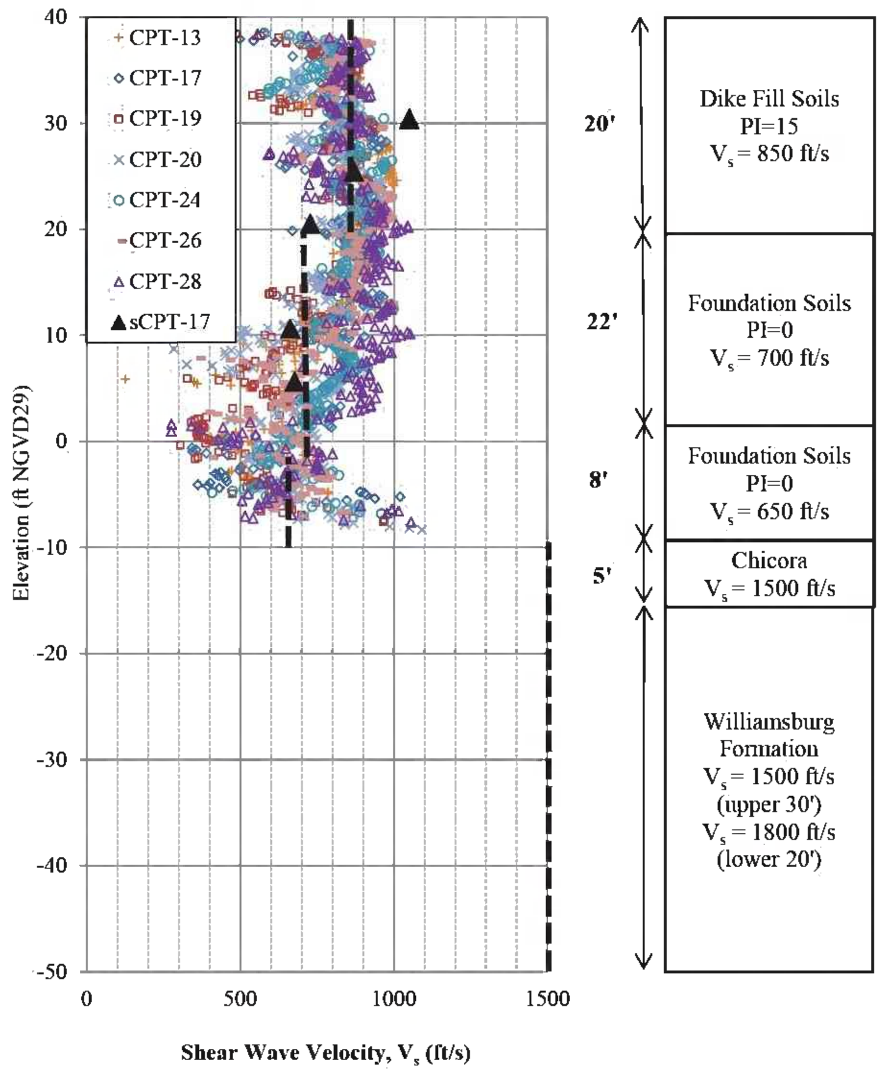

Figure 29. Representative Shear Wave Velocity Profile 2 (Dike Centerline)

Note:

1. 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.

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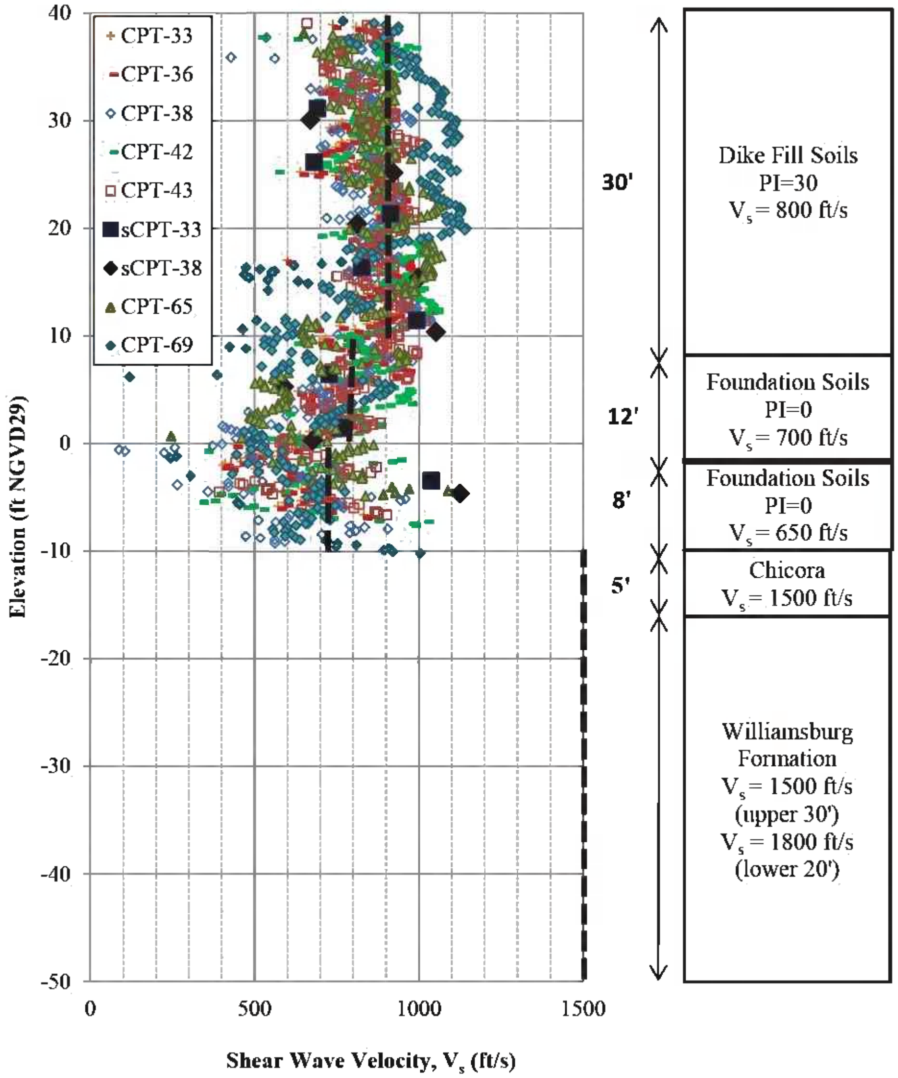

Figure 30. Representative Shear Wave Velocity Profile 3 (Dike Centerline)

Note:

1. 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

Written by: J. McNash Date: 10/11/2016 Reviewed by: W. Shin/ M. Zhu Date: 10/11/2016

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

Table 1-1. Summary of Index Testing

Boring ID	Source	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	S.G.
Units	-	-	ft bgs	ft NGVD29	%	%	%	%	%	%	%	%	%	-
GSB-02	Geosyntec 2013	SS	5	16.93	32.9	42	23	19	19.1	45.0	7.9	28.0	35.9	-
GSB-02	Geosyntec 2013	SS	10	11.93	32.0	-	-	-	-	-	-	-	18.9	-
GSB-02	Geosyntec 2013	SS	15	6.93	33.0	-	-	-	-	-	-	-	22.3	-
GSB-02	Geosyntec 2013	SS	25	-3.07	34.2	-	-	-	-	-	-	-	22.8	-
GSB-03	Geosyntec 2013	SS	10	28.39	21.6	29	18	11	8.2	41.3	12.5	38.0	50.5	-
GSB-03	Geosyntec 2013	ST	12.5	25.89	19.8	36	18	18	0.0	59.8	12.4	27.8	40.2	2.67
GSB-03	Geosyntec 2013	SS	20	18.39	24.3	50	23	27	-	-	-	-	-	-
GSB-03	Geosyntec 2013	SS	30	8.39	58.0	-	-	-	0.2	41.0	-	-	58.8	-
GSB-03	Geosyntec 2013	SS	35	3.39	35.4	-	-	-	-	-	-	-	28.0	-
GSB-03	Geosyntec 2013	SS	40	-1.61	33.9	-	-	-	6.8	66.3	-	-	26.9	-
GSB-04	Geosyntec 2013	SS	10	28.66	19.9	35	19	16	0.0	81.8	4.8	13.4	18.2	-
GSB-04	Geosyntec 2013	SS	20	18.66	20.1	42	22	20	-	-	-	-	-	-
GSB-04	Geosyntec 2013	SS	30	8.66	29.1	-	-	-	0.0	80.1	-	-	19.9	-
GSB-04	Geosyntec 2013	SS	35	3.66	47.2	-	-	-	-	-	-	-	39.0	-
GSB-04	Geosyntec 2013	SS	40	-1.34	35.3	-	-	-	-	-	-	-	19.6	-
GSB-04	Geosyntec 2013	SS	47	-8.34	28.4	-	-	-	-	-	-	-	79.0	-
GSB-04	Geosyntec 2013	SS	55	-16.34	41.9	70	46	24	0.0	17.2	42.2	40.6	82.8	-
GSB-05	Geosyntec 2013	SS	15	23.05	20.0	33	19	14	0.0	63.9	-	-	36.1	-
GSB-05	Geosyntec 2013	SS	30	8.05	28.2	-	-	-	-	-	-	-	9.5	-
GSB-05	Geosyntec 2013	SS	35	3.05	22.1	-	-	-	-	-	-	-	8.8	-
GSB-05	Geosyntec 2013	SS	40	-1.95	24.5	-	-	-	1.6	76.5	-	-	21.9	-
GSB-06	Geosyntec 2013	SS	10	10.68	27.1	-	-	-	1.2	77.1	-	-	21.7	-
GSB-06	Geosyntec 2013	SS	15	5.68	23.2	-	-	-	-	-	-	-	7.9	-
GSB-06	Geosyntec 2013	SS	20	0.68	30.8	-	-	-	-	-	-	-	16.3	-
GSB-07	Geosyntec 2013	ST	7.5	30.66	21.5	33	21	12	0.0	66.9	8.5	24.6	33.1	2.69
GSB-07	Geosyntec 2013	SS	15	23.16	19.1	34	17	17	0.0	62.5	-	-	37.5	-

Written by: **J. McNash** Date: **10/11/2016** Reviewed by: **W. Shin/ M. Zhu** Date: **10/11/2016**

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

Boring ID	Source	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	S.G.
Units	-	-	ft bgs	ft NGVD29	%	%	%	%	%	%	%	%	%	-
GSB-07	Geosyntec 2013	SS	25	13.16	25.8	37	19	18	-	-	-	-	-	-
GSB-07	Geosyntec 2013	SS	30	8.16	23.4	-	-	-	0.0	87.5	-	-	12.5	-
GSB-07	Geosyntec 2013	SS	35	3.16	28.2	-	-	-	0.7	56.4	13.8	29.1	42.9	-
GSB-07	Geosyntec 2013	SS	45	-6.84	16.6	-	-	-	-	-	-	-	17.5	-
GSB-08	Geosyntec 2013	SS	10	29.19	23.2	45	16	29	0.0	54.3	-	-	45.7	-
GSB-08	Geosyntec 2013	SS	15	24.19	23.0	-	-	-	-	-	-	-	39.6	-
GSB-08	Geosyntec 2013	SS	25	14.19	17.4	-	-	-	0.0	61.7	8.2	30.1	38.3	-
GSB-08	Geosyntec 2013	SS	35	4.19	23.9	-	-	-	-	-	-	-	12.8	-
GSB-08	Geosyntec 2013	ST	37	2.19	13.6	-	-	-	0.0	94.5	-	-	5.5	-
GSB-08	Geosyntec 2013	ST	38	1.19	30.6	30	19	11	0.5	67.6	8.2	23.7	31.9	2.64
GSB-08	Geosyntec 2013	SS	40	-0.81	32.0	-	-	-	-	-	-	-	17.4	-
GSB-08	Geosyntec 2013	SS	45	-5.81	23.0	-	-	-	-	-	-	-	15.1	-
GSB-09	Geosyntec 2013	SS	10	1.04	21.9	-	-	-	6.2	80.5	-	-	13.3	-
GSB-09	Geosyntec 2013	SS	15	-3.96	29.8	-	-	-	-	-	-	-	20.8	-
GSB-10	Geosyntec 2013	SS	15	23.86	23.4	47	20	27	0.0	49.7	-	-	50.3	-
GSB-10	Geosyntec 2013	SS	20	18.86	21.7	-	-	-	-	-	-	-	41.4	-
GSB-10	Geosyntec 2013	SS	32	6.86	16.2	-	-	-	-	-	-	-	26.5	-
GSB-10	Geosyntec 2013	SS	35	3.86	24.4	-	-	-	-	-	-	-	16.3	-
GSB-10	Geosyntec 2013	SS	45	-6.14	20.4	-	-	-	-	-	-	-	16.0	-
GSB-11	Geosyntec 2013	SS	10	27.78	34.3	47	17	30	1.4	50.0	8.8	39.8	48.6	-
GSB-11	Geosyntec 2013	SS	15	22.78	30.4	44	18	26	-	-	-	-	-	-
GSB-11	Geosyntec 2013	SS	25	12.78	32.0	44	15	29	0.0	58.2	-	-	41.8	-
GSB-11	Geosyntec 2013	SS	35	2.78	83.5	114	88	26	2.0	57.4	19.6	21.0	40.6	-
GSB-11	Geosyntec 2013	SS	40	-2.22	168.5	-	-	-	-	-	-	-	49.5	-
GSB-11	Geosyntec 2013	SS	45	-7.22	140.6	-	-	-	-	-	-	-	42.6	-
GSB-12	Geosyntec 2013	SS	10	2.31	22.9	-	-	-	-	-	-	-	9.6	-
GSB-12	Geosyntec 2013	SS	15	-2.69	31.8	-	-	-	0.3	84.0	-	-	15.7	-

Written by: J. McNash Date: 10/11/2016 Reviewed by: W. Shin/ M. Zhu Date: 10/11/2016

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

Boring ID	Source	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	S.G.
Units	-	-	ft bgs	ft NGVD29	%	%	%	%	%	%	%	%	%	-
GSB-12	Geosyntec 2013	SS	23	-10.69	11.5	-	-	-	-	-	-	-	22.7	-
SPT-106	Geosyntec 2013	SS	25.75	12.69	31.4	63	20	43	0.0	22.3	11.3	66.3	77.7	
SPT-106	Geosyntec 2013	SS	40.75	-2.31	26.2	-	-	-	-	-	-	-	6.9	
SPT-106	Geosyntec 2013	ST	61.00	-22.56	40.5	-	-	-	-	-	-	-	-	
SPT-106	Geosyntec 2013	SS	75.75	-37.31	42.2	69	31	38	8.8	17.2	32.7	41.3	74.0	
SPT-106	Geosyntec 2013	SS	95.75	-57.31	45.3	61	30	31	0.6	25.7	35.6	38.1	73.7	
SPT-106A	Geosyntec 2013	ST	9.00	25.00	25.8	-	-	-	0.0	0.7	92.9	6.4	99.3	
SPT-106A	Geosyntec 2013	ST	17.00	17.00		NP	NP	NP	0.0	0.2	-	-	99.8	
SPT-106B	Geosyntec 2013	ST	7.00	27.00	56.2	-	-	-	0.0	5.9	-	-	94.1	
SPT-106B	Geosyntec 2013	ST	15.00	19.00	-	-	-	-	0.0	0.7	74.3	25.0	99.3	
SPT-107A	Geosyntec 2013	ST	7.00	27.00	-	-	-	-	0.0	1.9	89.8	8.3	98.1	
SPT-107A	Geosyntec 2013	ST	15.00	19.00	-	-	-	-	0.0	0.6	-	-	99.4	
SPT-107B	Geosyntec 2013	ST	9.00	25.00	-	-	-	-	0.0	0.3	-	-	99.7	
SPT-107B	Geosyntec 2013	ST	17.00	17.00	-	NP	NP	NP	0.0	0.1	88.7	11.2	99.9	
TP-1[1]	Geosyntec 2013	Bulk	5	-	44.7	NP	NP	NP	0.0	4.6	89.3	6.1	95.4	2.38
TP-2[1]	Geosyntec 2013	Bulk	5	-	46.8	NP	NP	NP	0.0	19.4	74.5	6.1	80.6	2.39
GSB-11A	Geosyntec, 2016	SS	1.0	36.78	17.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	3.0	34.78	19.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	5.0	32.78	20.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	7.0	30.78	20.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	9.0	28.78	27.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	11.0	26.78	27.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	13.0	24.78	23.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	17.0	20.78	21.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	ST	19.0	18.78	18.0	36	13	23	0.0	61.2	-	-	38.8	
GSB-11A	Geosyntec, 2016	SS	21.0	16.78	25.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	23.0	14.78	39.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	25.0	12.78	18.0	-	-	-	-	-	-	-	-	-

Written by: J. McNash Date: 10/11/2016 Reviewed by: W. Shin/ M. Zhu Date: 10/11/2016

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

Boring ID	Source	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	S.G.
Units	-	-	ft bgs	ft NGVD29	%	%	%	%	%	%	%	%	%	-
GSB-11A	Geosyntec, 2016	SS	27.0	10.78	26.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	35.0	2.78	23.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	39.0	-1.22	28.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	42.0	-4.22	34.0	-	-	-	-	-	-	-	-	-
GSB-11A	Geosyntec, 2016	SS	49.0	-11.22	12.0	-	-	-	-	-	-	-	-	-
SC-20	S&ME 1978	SS	1.75	15.75	-	-	-	-	-	-	-	-	16.3	-
SC-20	S&ME 1978	SS	5.25	12.25	-	-	-	-	-	-	-	-	23.0	-
SC-20	S&ME 1978	SS	7.5	10.00	-	-	-	-	-	-	-	-	9.4	-
SC-25	S&ME 1978	Bulk	3.5	14.40	-	-	-	-	-	-	-	-	27.7	-
SC-25	S&ME 1978	Bulk	7	10.90	-	-	-	-	-	-	-	-	25.0	-
SC-30	S&ME 1978	Bulk	2.5	13.30	27.5	30	17	13	0.0	59.9	17.6	22.5	40.1	-
SC-31	S&ME 1978	Bulk	3	13.90	19.7	27	18	9	0.0	74.8	15.2	10.0	25.2	-
SC-34	S&ME 1978	Bulk	2.25	13.25	24.0	31	20	11	0.0	66.4	-	-	33.6	-
SC-34	S&ME 1978	Bulk	7.5	8.00	-	-	-	-	-	-	-	-	11.1	-
SC-35	S&ME 1978	Bulk	8	7.50	22.0	NP	NP	NP	0.0	88.90	-	-	11.1	-
SC-36	S&ME 1978	Bulk	2.5	15.40	32.7	39	22	17	0.0	37.50	37.70	24.8	62.5	-
SC-36	S&ME 1978	Bulk	3.5	14.40	22.3	30	18	12	0.0	62.10	19.70	18.2	37.9	-
SC-36	S&ME 1978	Bulk	6.5	11.40	23.5	NP	NP	NP	0.0	98.8	-	-	1.2	-
SC-38	S&ME 1978	Bulk	2.5	13.70	33.8	43	21	22	0.0	47.6	27.6	24.8	52.4	-
SC-39	S&ME 1978	Bulk	1.75	14.05	28.8	60	22	38	0.0	24.8	32.2	43.0	75.2	-
SC-39	S&ME 1978	Bulk	6.5	9.30	79.2	33	23	10	0.8	41.3	55.4	2.5	57.9	-
SC-45	S&ME 1978	SS	1.75	17.45	22.6	-	-	-	-	-	-	-	16.6	-
SC-45	S&ME 1978	SS	5.25	13.95	23.9	-	-	-	-	-	-	-	9.9	-
B-201	PCRA 1999	SS	9	29.10	22.6	NP	NP	NP	0.0	79.0	-	-	21.0	-
B-201	PCRA 1999	SS	13	25.10	18.1	45.0	20.0	20.0	0.0	55.0	-	-	45.0	-
B-201	PCRA 1999	SS	41	-2.90	-	47.0	20.0	27.0	3.0	82.0	-	-	15.0	-
B-205	PCRA 1999	SS	11	28.00	13.0	-	-	-	0.0	86.0	-	-	14.0	-

Written by: **J. McNash** Date: **10/11/2016** Reviewed by: **W. Shin/ M. Zhu** Date: **10/11/2016**

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

Boring ID	Source	Sample Type	Depth	Elev.	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	Silt	Clay	Fines Content	S.G.
Units	-	-	ft bgs	ft NGVD29	%	%	%	%	%	%	%	%	%	-
B-206	PCRA 1999	SS	35	3.90	31.8	NP	NP	NP	1.0	65.0	-	-	34.0	-
B-212	PCRA 1999	SS	19	19.60	19.1	37.0	17.0	20.0	0.0	56.07	-	-	43.93	-
B-214	PCRA 1999	SS	39	0.10	95.5	-	-	-	-	5.19	-	-	94.81	-
B-214	PCRA 1999	SS	61	-21.90	44.7	69.0	31.0	38.0	0.7	12.0	-	-	87.2	-

Note:

1. SS = split spoon sample; ST = thin-walled Shelby tube sample; Bulk = bulk sample; ft bgs = feet below ground surface; Elev. = Elevation; NGVD 29 = National Geodetic Vertical Datum of 1929; S.G. = specific gravity.

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Table 1-2. Summary of Geosyntec Triaxial Testing

Boring ID	Depth	Elev.	MC[1]	Dry Unit Weight	Wet Unit Weight	σ_{consol}'	$\sigma_{1,f}'$	$\sigma_{3,f}'$	S_u	S_u / σ_c'	OCR	S_u / σ_c'	ϕ'	c'
Units	ft bgs	ft NGVD 29	%	pcf	pcf	psi	psi	psi	psi	-	-	-	°	psi
GSB-3	12.5	25.89	20.2	109.4	131.50	4.7	31.1	8.4	11.35	2.41	1.00	2.41	33.0	0.68
GSB-3	12.5	25.89	22.1	101.8	124.30	9.3	50.1	14.7	17.7	1.90	1.00	1.90		
GSB-07	7.5	30.66	15.5	111	128.21	3.8	18.9	5.4	6.75	5.44	1.00	5.44	35.8	0.00
GSB-07	7.5	30.66	17.1	112.8	132.09	10.8	79.9	20.8	29.55	0.77	1.00	0.77		
GSB-08	37.5	1.69	25.0	101.2	126.50	21.1	250	20.5	114.75	2.41	1.00	2.41	33.1	0.00
GSB-08	37.5	1.69	22.3	106.2	129.88	40.0	88.6	26.8	30.9	1.90	1.00	1.90		
GSB-11A	19.0	18.78	18.2	109.8	129.78	11.1	64.63	20.9	21.865	1.97	1.00	1.97	25.5	3.75
GSB-11A	19.0	18.78	18.2	108.3	128.01	22.2	32.19	7.7	12.245	0.55	1.00	0.55		
GSB-11A	19.0	18.78	30.3	92.2	120.14	44.5	39.33	23.4	7.965	0.18	1.00	0.18		
TP-01	-	-	51.4	70.5	106.7	3.5	117.1	25.5	45.8	13.09	1.00	13.09	41.48	0.00
TP-01	-	-	51.0	71.8	108.4	6.3	190.7	38.9	75.9	12.05	1.00	12.05		
TP-01	-	-	51.4	71.9	108.9	10.4	230	45.6	92.2	8.87	1.00	8.87		
TP-02	-	-	53.0	68.9	105.4	3.5	187.9	38.5	74.7	21.34	1.00	21.34	40.80	0.00
TP-02	-	-	53.1	68.6	105.0	6.3	159.5	31.6	64.0	10.15	1.00	10.15		
TP-02	-	-	52.9	69	105.5	10.4	227.4	49.5	89.0	8.55	1.00	8.55		
SPT-106A: S1	9.0	25.0	25.3	94	117.8	2.0	267.3	54.0	106.7	53.33	1.00	53.33	42.6	0.00
SPT-106A: S1	9.0	25.0	37.6	83.5	114.9	6.0	257.9	46.7	105.6	17.60	1.00	17.60		
SPT-106B: S3	15.0	19.0	55.5	66.2	102.9	3.0	27.1	6.6	10.3	3.42	1.00	3.42	40.6	0.00
SPT-106B: S3	15.0	19.0	82.5	51.8	94.5	8.0	46	8.9	18.6	2.32	1.00	2.32		
SPT-106B: S3	15.0	19.0	68.9	56.1	94.8	13.5	43.9	9.5	17.2	1.27	1.00	1.27		
SPT107A: S3	15.0	19.0	71.4	57.4	98.4	3.0	7.8	1.6	3.1	1.03	1.00	1.03	33.6	0.79
SPT107A: S3	15.0	19.0	89.1	51.1	96.6	9.0	10.2	1.7	4.3	0.47	1.00	0.47		
SPT107A: S3	15.0	19.0	91.2	46.8	89.5	18.0	17.6	4.2	6.7	0.37	1.00	0.37		
SPT-107B: S1	9.0	25.0	60.6	65	104.4	2.0	10.8	1.8	4.5	2.25	1.00	2.25	34.1	1.06
SPT-107B: S2	9.0	25.0	75.1	55.8	97.7	8.0	9.2	1.3	4.0	0.49	1.00	0.49		

Notes:

1. Initial moisture content of sample prior to shearing.
2. Undrained shear strength ratio, friction angle, and cohesion intercepts were computed by Geosyntec.
3. Drained parameters for CU test on GSB-11A were computed based on the results for the first two specimens due to sample quality.

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Table 1-3. Summary of S&ME Triaxial Testing (within the Slurry Pond)

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'	OCR	S_u / σ_c'	ϕ'	c'
Units	ft bgs	ft NGVD 29	%	pcf	pcf	psi	psi	psi	psi	-	-	-	°	psi
SC-25	3.5	14.4	17.8	105.3	124.0	-	50.00	6.94	21.53	3.10	1.00	3.10	25.8	7.64
SC-25	3.5	14.4	17.6	105.4	124.0	-	76.39	20.83	27.78	1.33	1.00	1.33		
SC-25	3.5	14.4	17.4	105.6	124.0	-	98.61	34.72	31.94	0.92	1.00	0.92		
SC-34	2.25	12.0	16.4	95.2	110.8	-	41.67	6.94	17.36	2.50	1.00	2.50	36.9	1.04
SC-34	2.25	12.0	17.2	100.6	117.9	-	67.36	20.83	23.26	1.12	1.00	1.12		
SC-34	2.25	12.0	16.4	95.2	110.8	-	100.00	34.72	32.64	0.94	1.00	0.94		
SC-36	3.5	14.4	17.8	105.3	124.0	-	34.72	6.94	13.89	2.00	1.00	2.00	40.0	1.18
SC-36	3.5	14.4	17.6	105.3	123.8	-	67.71	20.83	23.44	1.13	1.00	1.13		
SC-36	3.5	14.4	17.4	105.3	123.6	-	84.03	34.72	24.65	0.71	1.00	0.71		
SC-41	20.5	-3.0	10.0	112.3	123.6	-	24.30	20.83	1.74	-	-	-	-	1.73
SC-41	20.5	-3.0	11.0	111.5	123.8	-	37.50	34.72	1.39	-	-	-		

Notes:

1. Principal stresses were estimated from S&ME testing results. Strength parameters were selected by S&ME, and these results are provided within Attachment 4.
2. SC-41 was conducted as an UU test (Unconsolidated Undrained) and used to compute the undrained shear strength. The material tested was described as sandy clay with a significant fraction of shells.
3. Moisture content provided prior to shearing of the sample.
4. Undrained shear strength ratio was computed by Geosyntec; friction angle and effective cohesion intercept were provided in S&ME (1978) Report.

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Table 1-4. Summary of S&ME Standard Proctor Testing

Boring ID	Depth	Optimum Moisture Content	Dry Unit Weight	Wet Unit Weight
Units	ft bgs	%	pcf	pcf
SC-25	3.5	16.0	112.8	130.8
SC-25	7.0	15.8	108.8	126.0
SC-34	13.3	15.6	110.5	127.7
SC-34	7.5	15.0	106.0	121.9
SC-36	14.4	14.5	113.1	129.5

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: SLURRY POND**

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Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS: SLURRY 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 Slurry Pond 3 & 4 (Slurry Pond) at 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 Slurry 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 this 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) 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 damping (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: Slurry Pond*" (Data Package). Information that is specific to the site response analysis is presented herein. To develop representative soil profiles, the Slurry Pond perimeter dike was divided into three sections depending on the depth of the dike fill and the V_s profile of the subsurface as shown in Figure 5. The top of the perimeter dike is relatively flat, whereas the base of the dike toe undulates with free-field ground surface elevation. In general, the base of the northwestern dike is lower (i.e., the dike is taller) than the rest of the dike (Profile 3 in Figure 5). The remainder of the perimeter dike was divided into two sections based on the V_s profile. As such, three representative profiles to 100 ft below ground surface (bgs) were developed for the perimeter dike (Profiles 1, 2, and 3), and are shown in Figure 6. For all three profiles, the water table was assumed to be at a depth of 15 ft bgs.

Profiles 1, 2, and 3 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 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 weights of the dike and foundation soils were 125 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 through 9c show calculated maximum shear strains and shear stresses for Profiles 1 through 3, 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 0.16 g. 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 through 9c 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.

REFERENCES

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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 Cyclic Shear Stress Envelopes

Profile 1		Profile 2		Profile 3	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	39	2.5	37	2.5	45
7.5	89	7.5	89	7.5	90
12.5	144	12.5	131	12.5	127
17.5	197	17.5	178	17.5	169
22.5	251	22.5	227	22.5	206
27.5	299	27.5	263	27.5	243
32.0	327	32.0	288	32.0	280
36.0	344	36.0	301	36.0	299
40.0	354	40.0	307	40.0	307
44.0	353	44.0	304	44.0	304
48.0	363	48.0	313	48.0	313
52.5	442	52.5	375	52.5	386
60.0	548	60.0	492	60.0	488
70.0	630	70.0	572	70.0	572
80.0	714	80.0	663	80.0	661
90.0	808	90.0	760	90.0	761
100.0	909	100.0	841	100.0	831

Figures

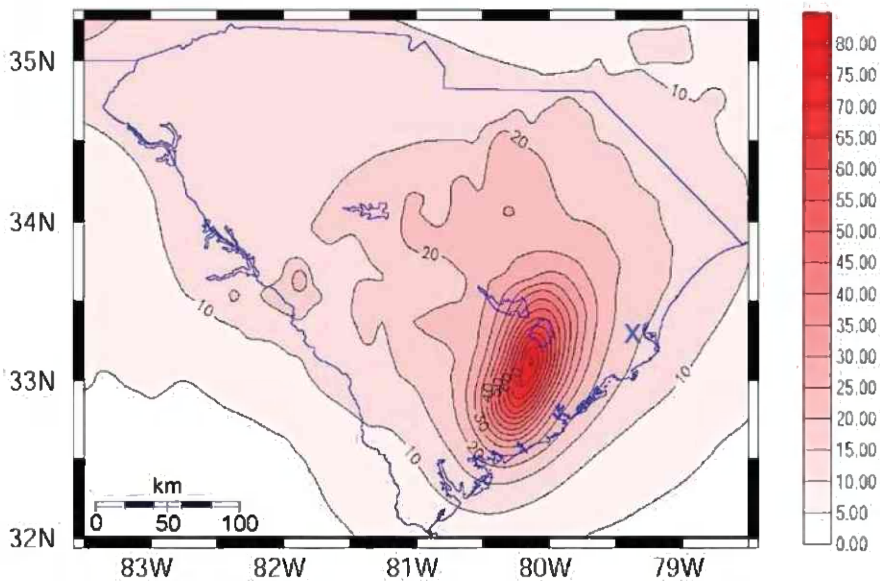

Figure 1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Chapman and Talwani, 2006)

Note:

1. PGA for WGS was selected as 0.16g.

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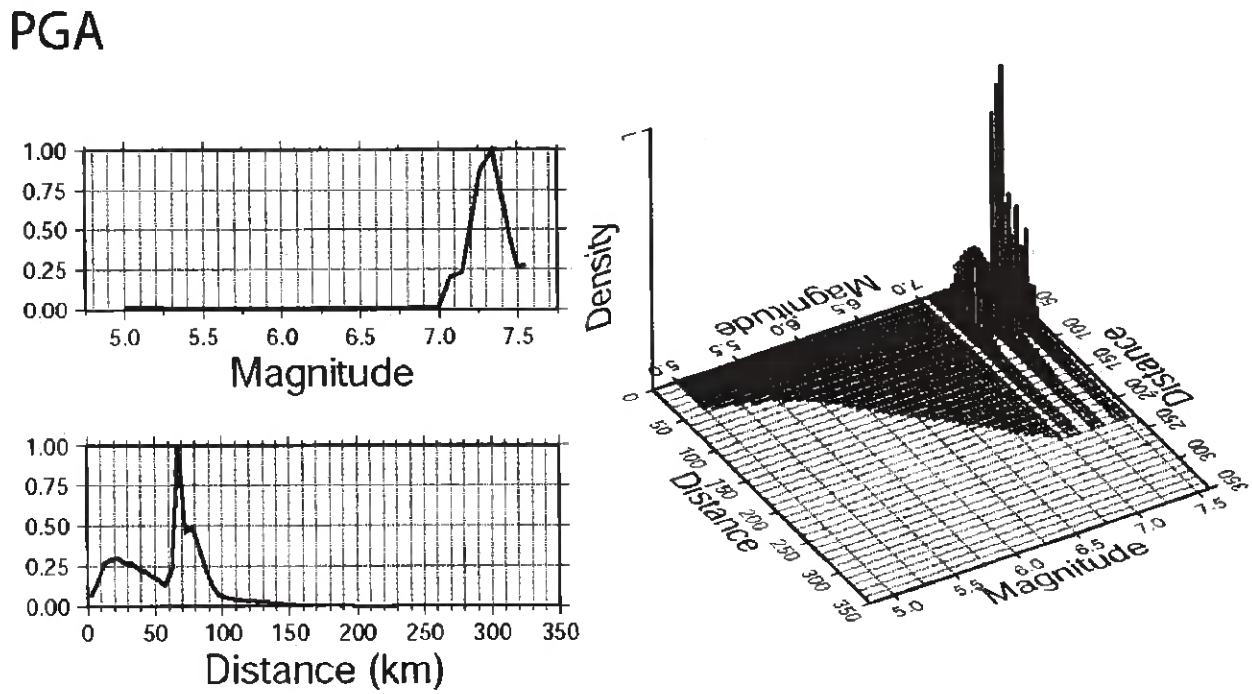

Figure 2. Deaggregation of 2 Percent Probability of Exceedance in 50 Years for PGA near Georgetown, South Carolina

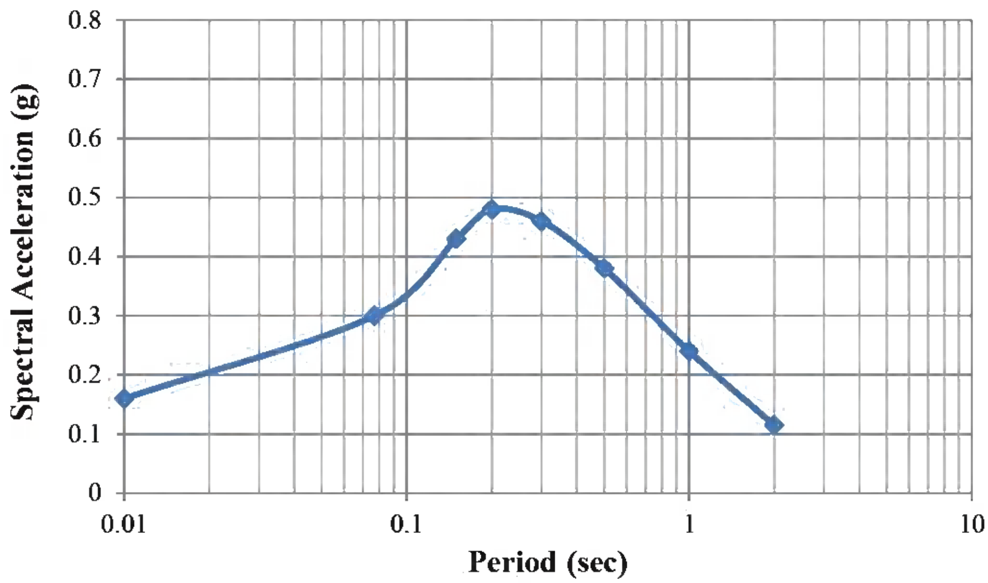

Figure 3. "Geologically Realistic" Target Response Spectrum for WGS

Notes:

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

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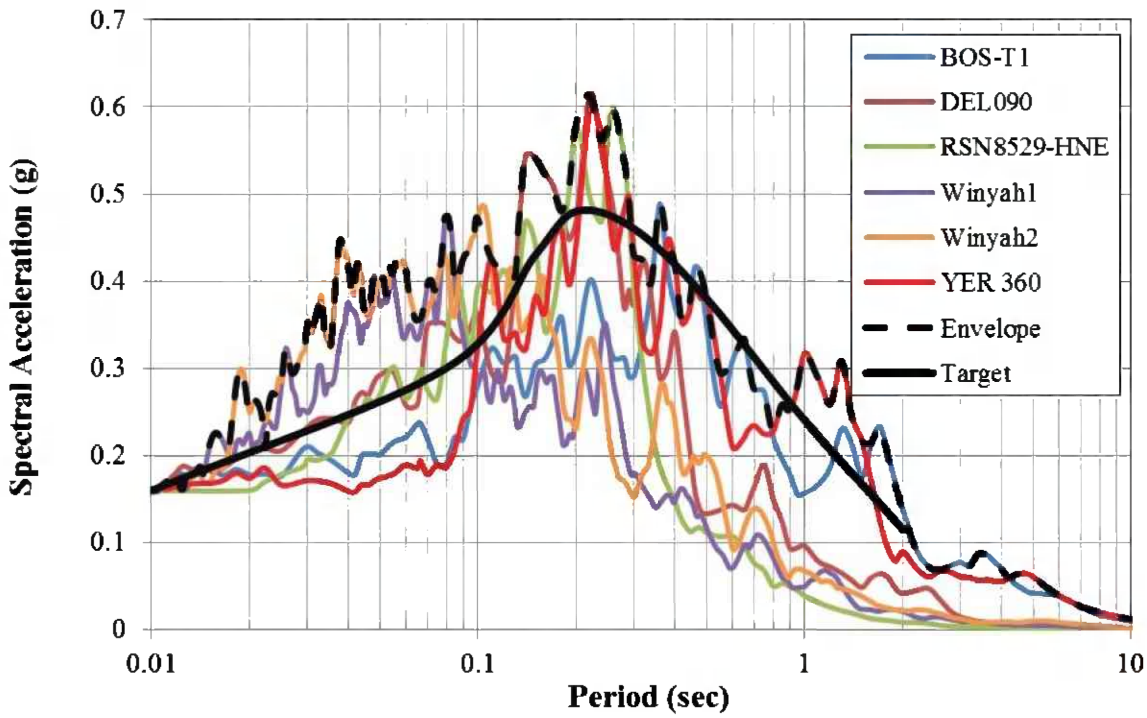

Figure 4. Response Spectra of Scaled Time Histories Selected for Seismic Evaluations

Note:

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

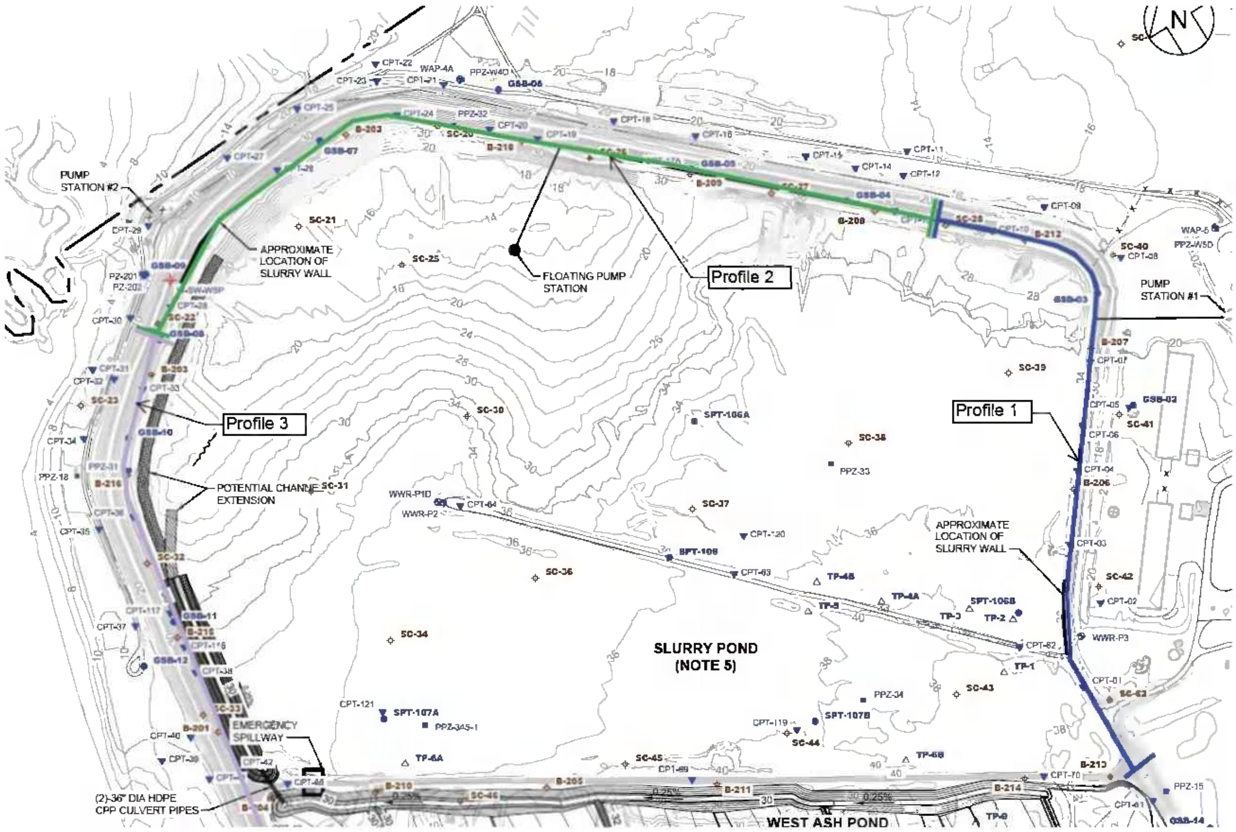

Figure 5. Locations of Representative Soil Profiles

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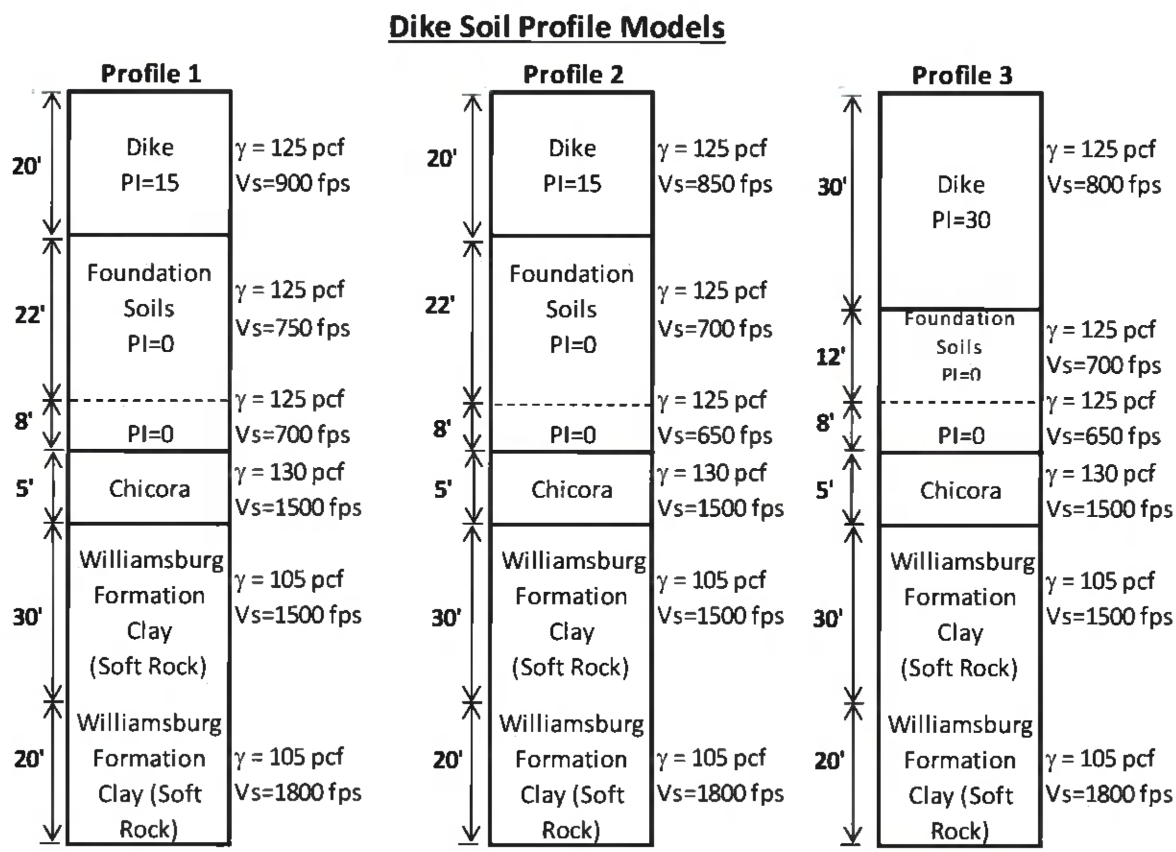

Figure 6. DEEPSOIL® Soil Profile Models for the Three Representative Dike Profiles

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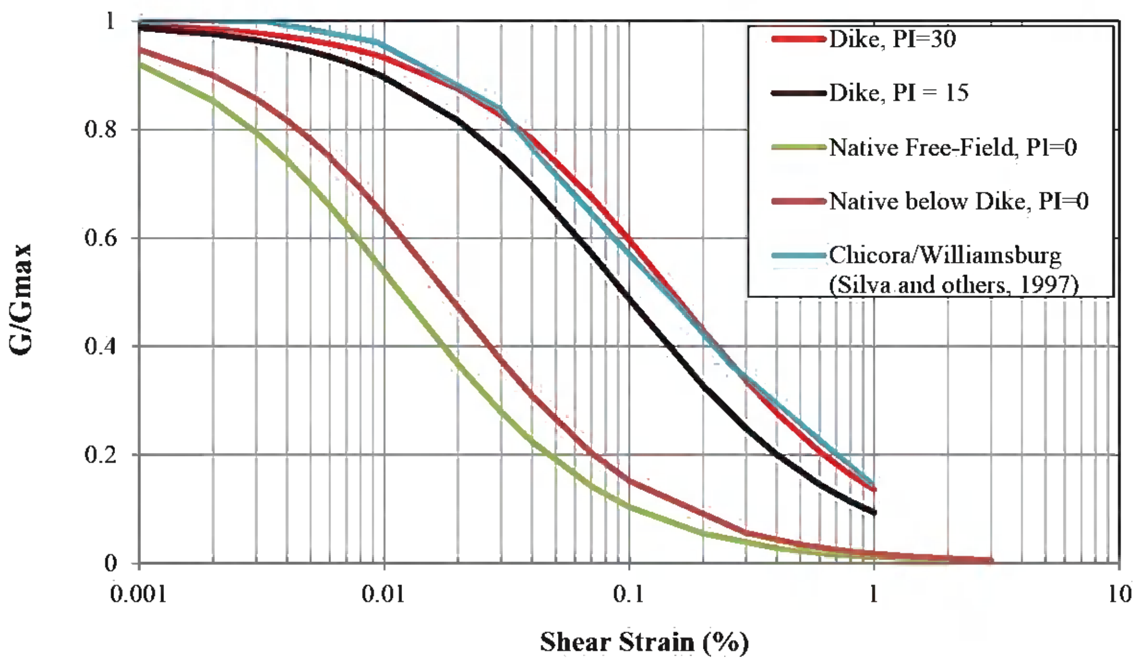

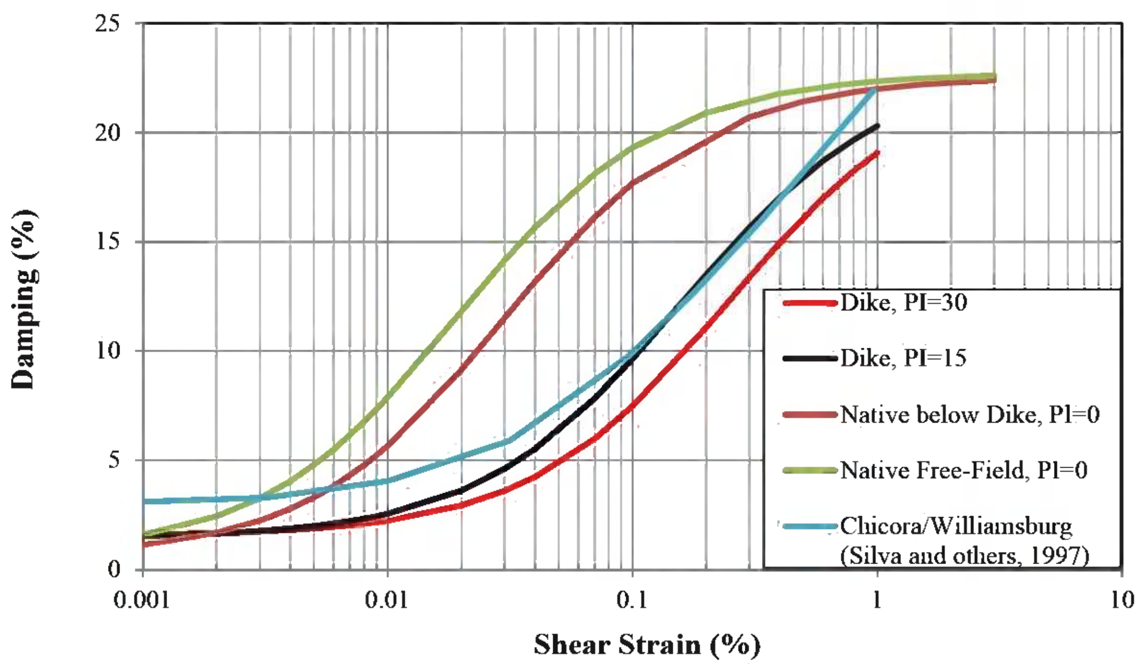

Figure 7. Shear Modulus Reduction (G/G_{max}) and Damping Curves for Soils Used in Site Response Analyses

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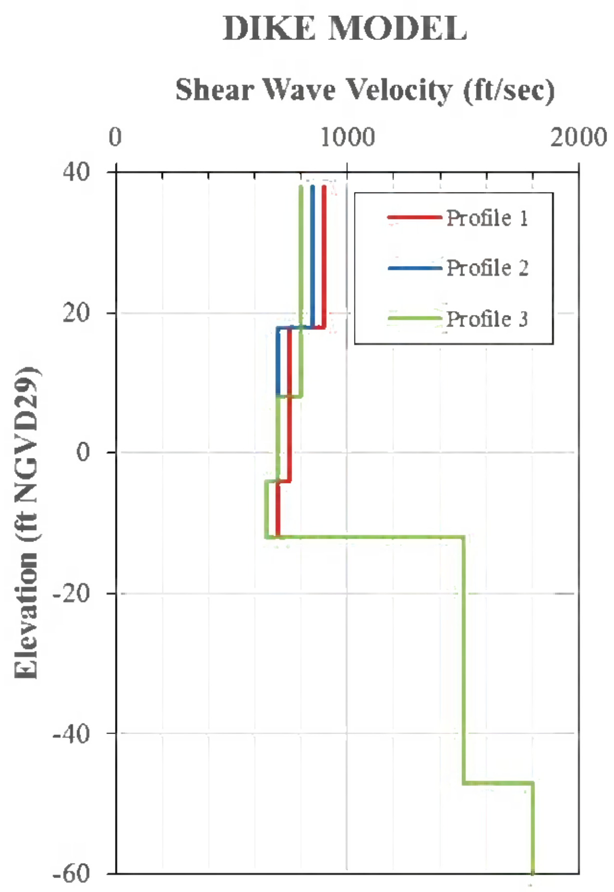

Figure 8. Selected Shear Wave Velocity (V_s) Profiles for Site Response Analyses

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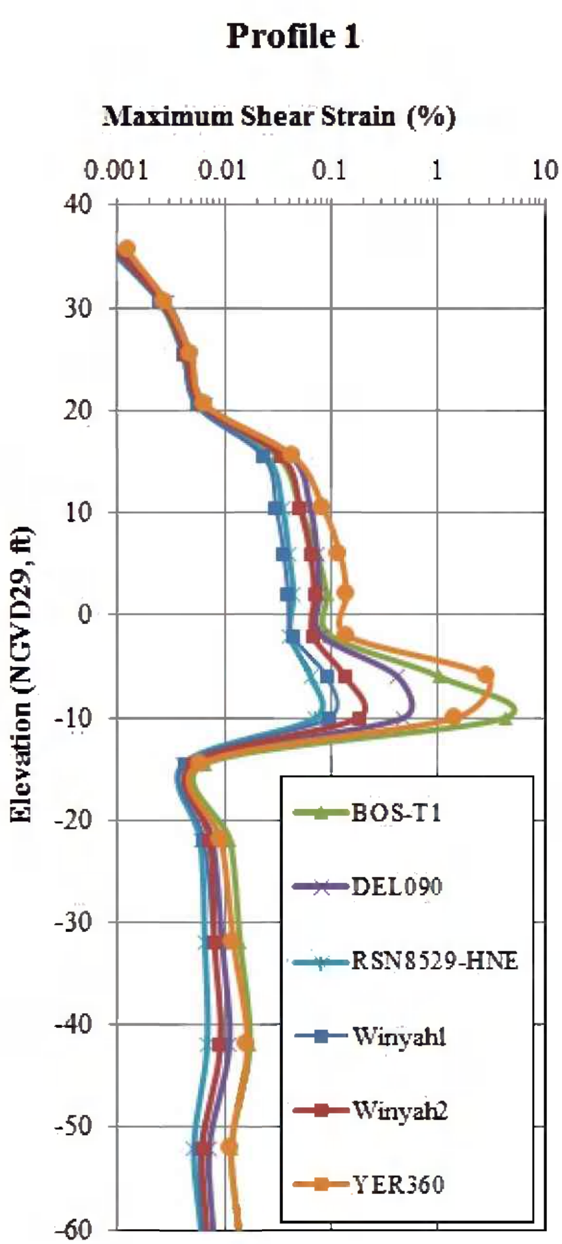


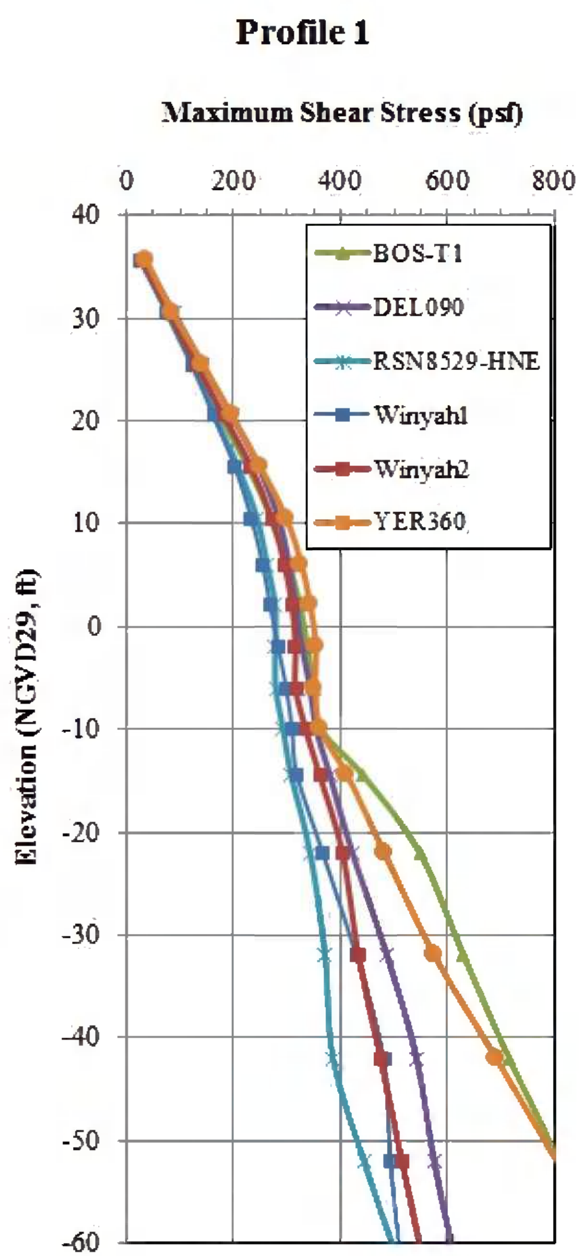
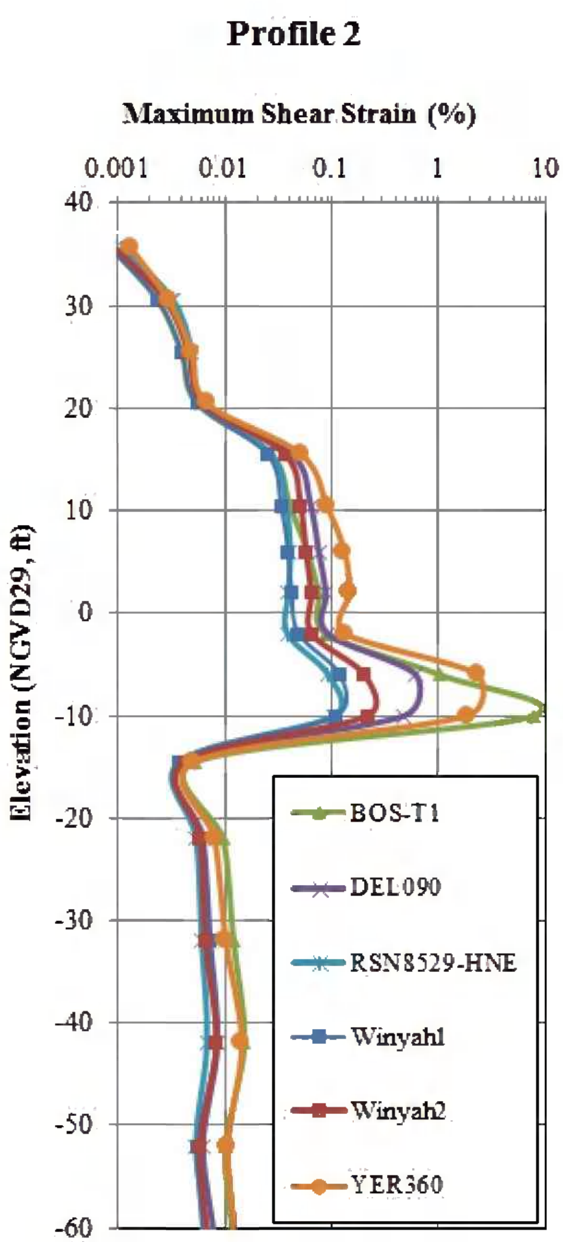


Figure 9a. Site Response Analysis Results for Profile 1



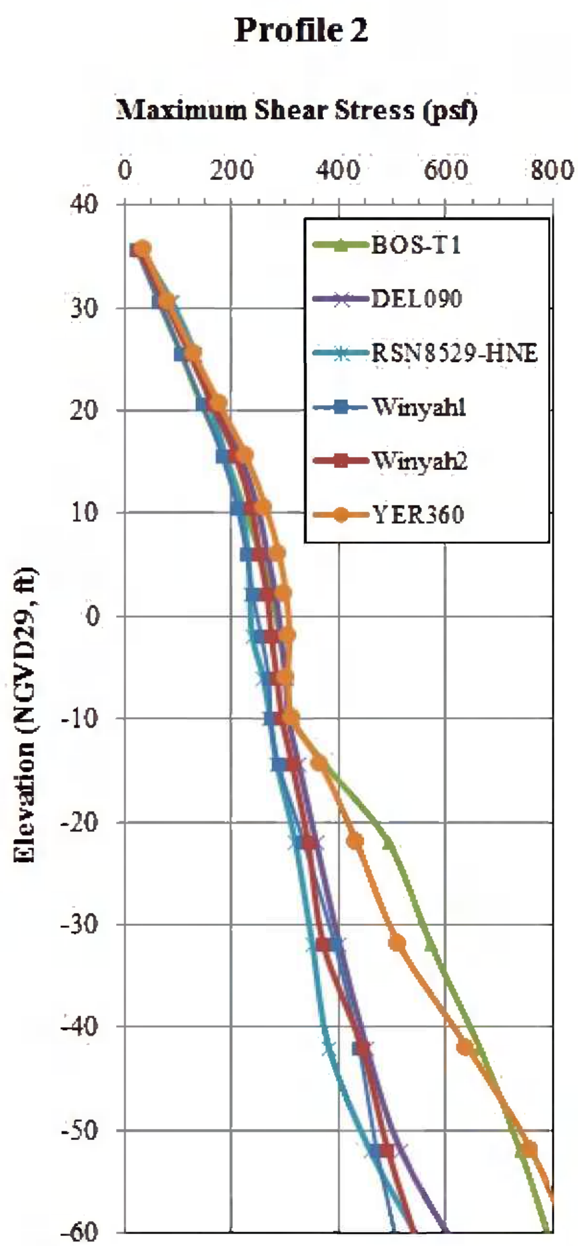


Figure 9b. Site Response Analysis Results for Profile 2

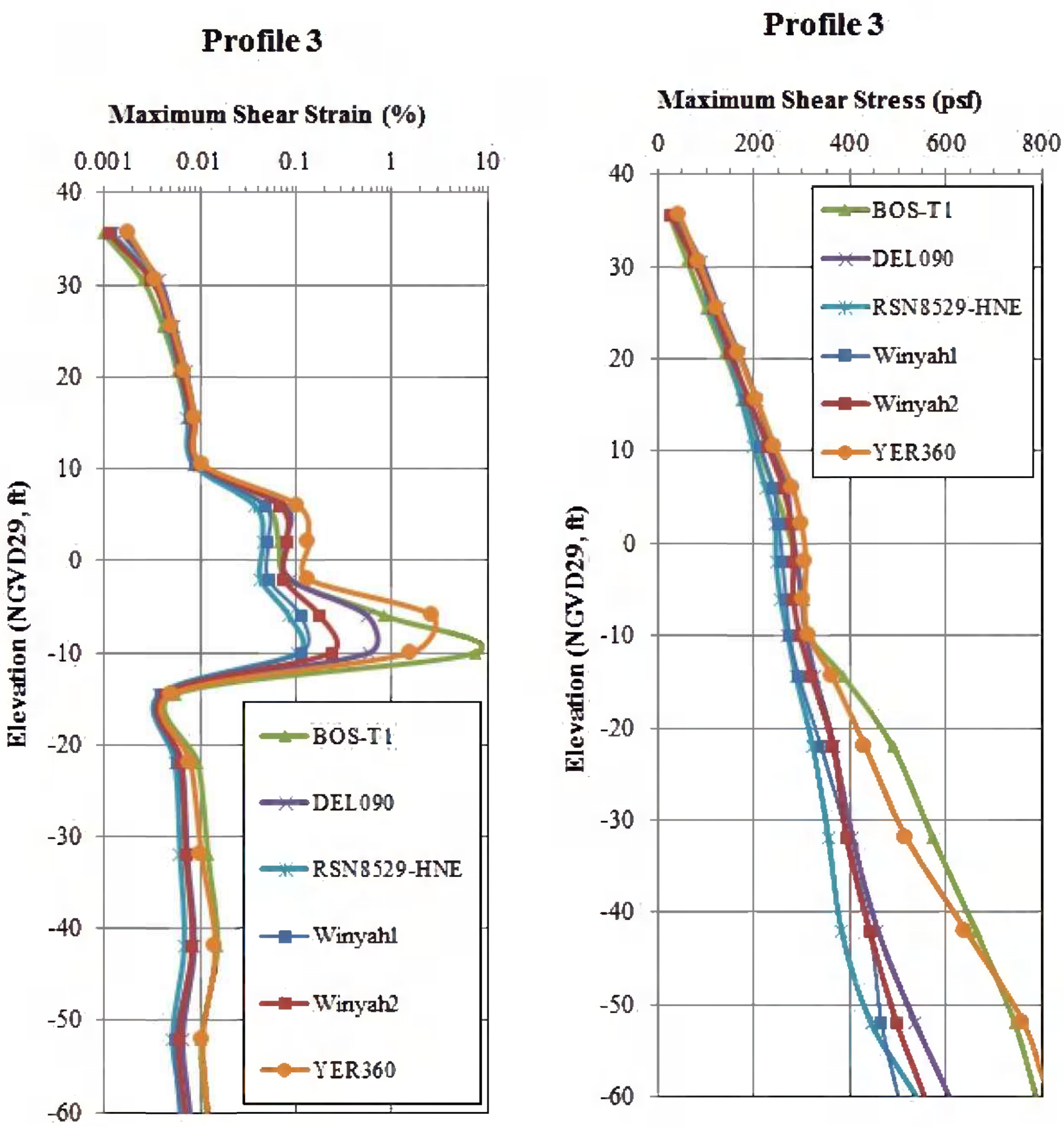


Figure 9c. Site Response Analysis Results for Profile 3

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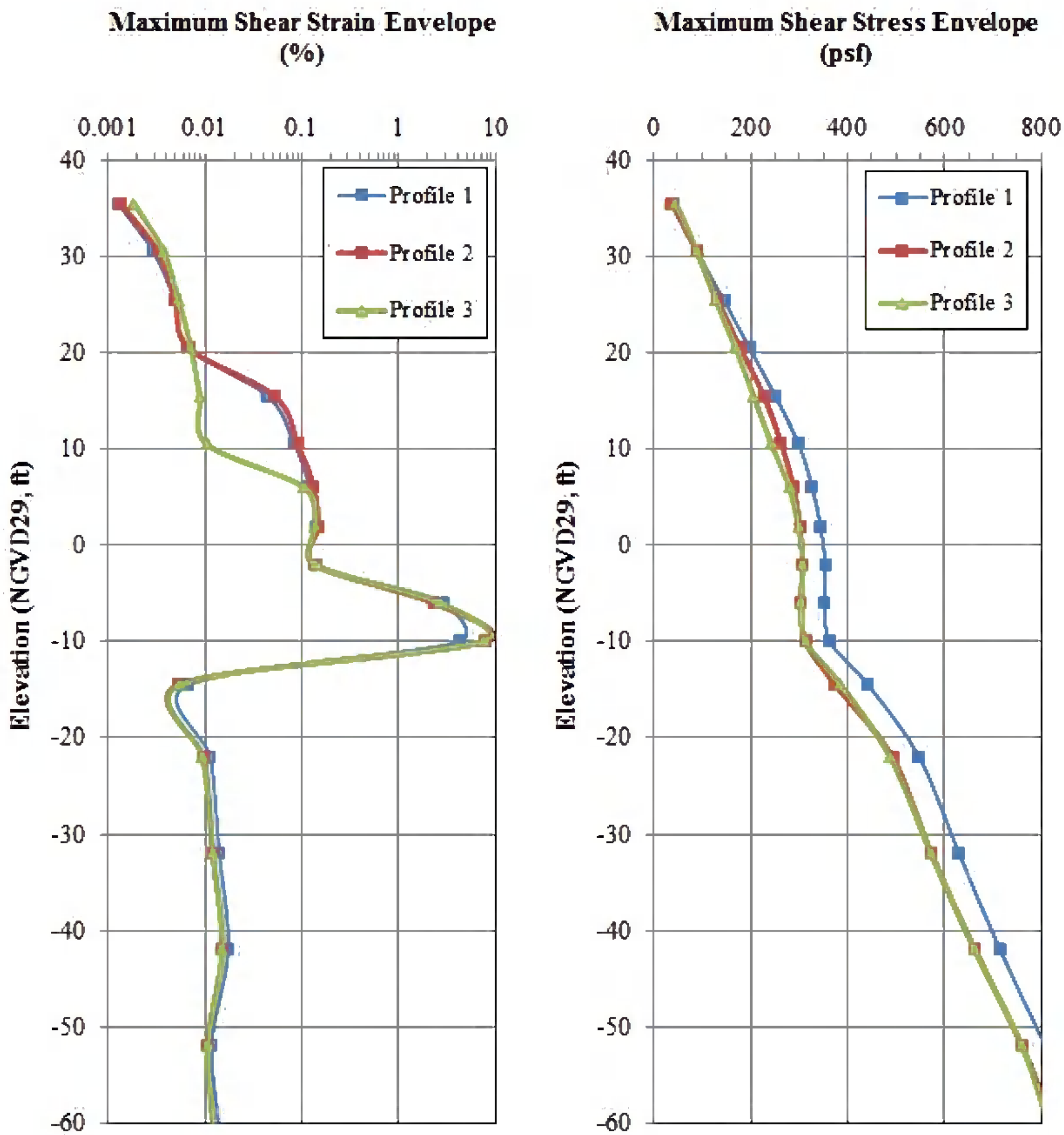

Figure 10. Maximum Shear Strain and Shear Stress Envelopes for Each Profile

Appendix 1

Peak Ground Accelerations from Different Seismic Hazard Maps

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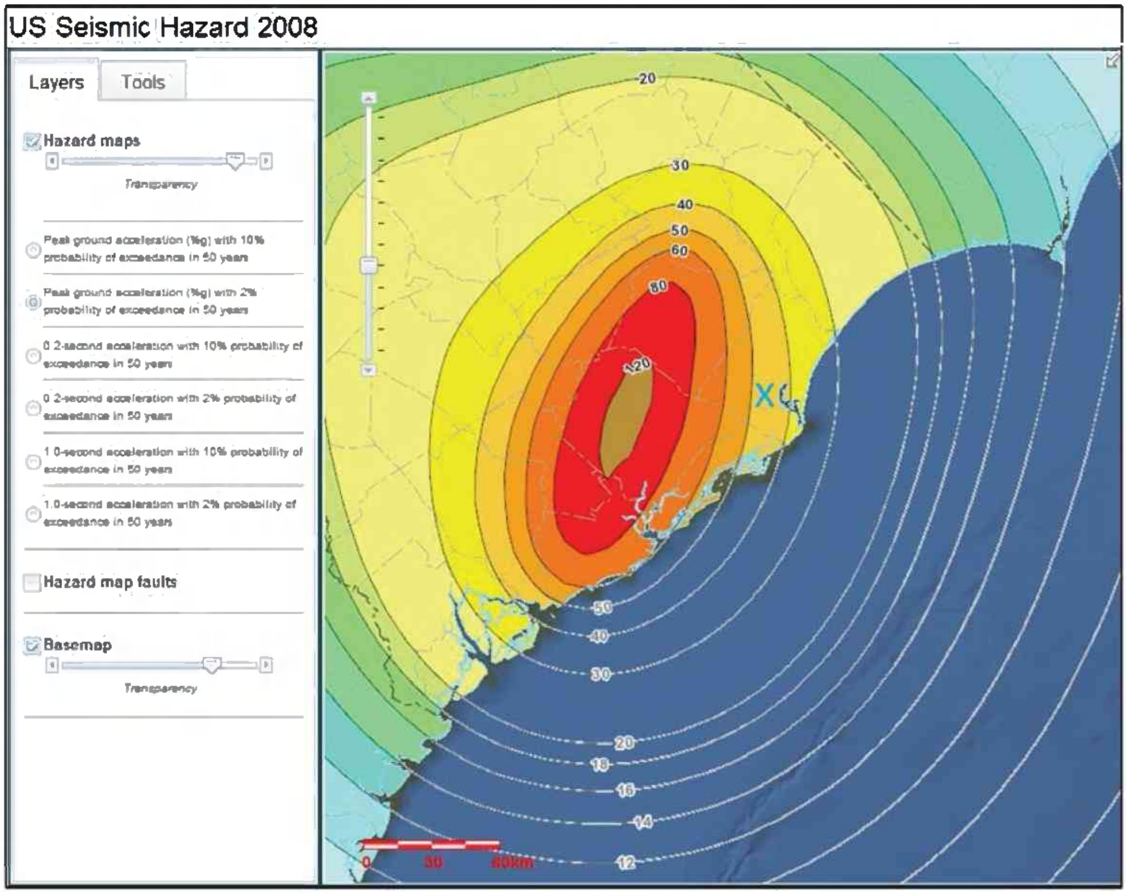

Figure 1-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years (USGS, 2008)

Note:

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

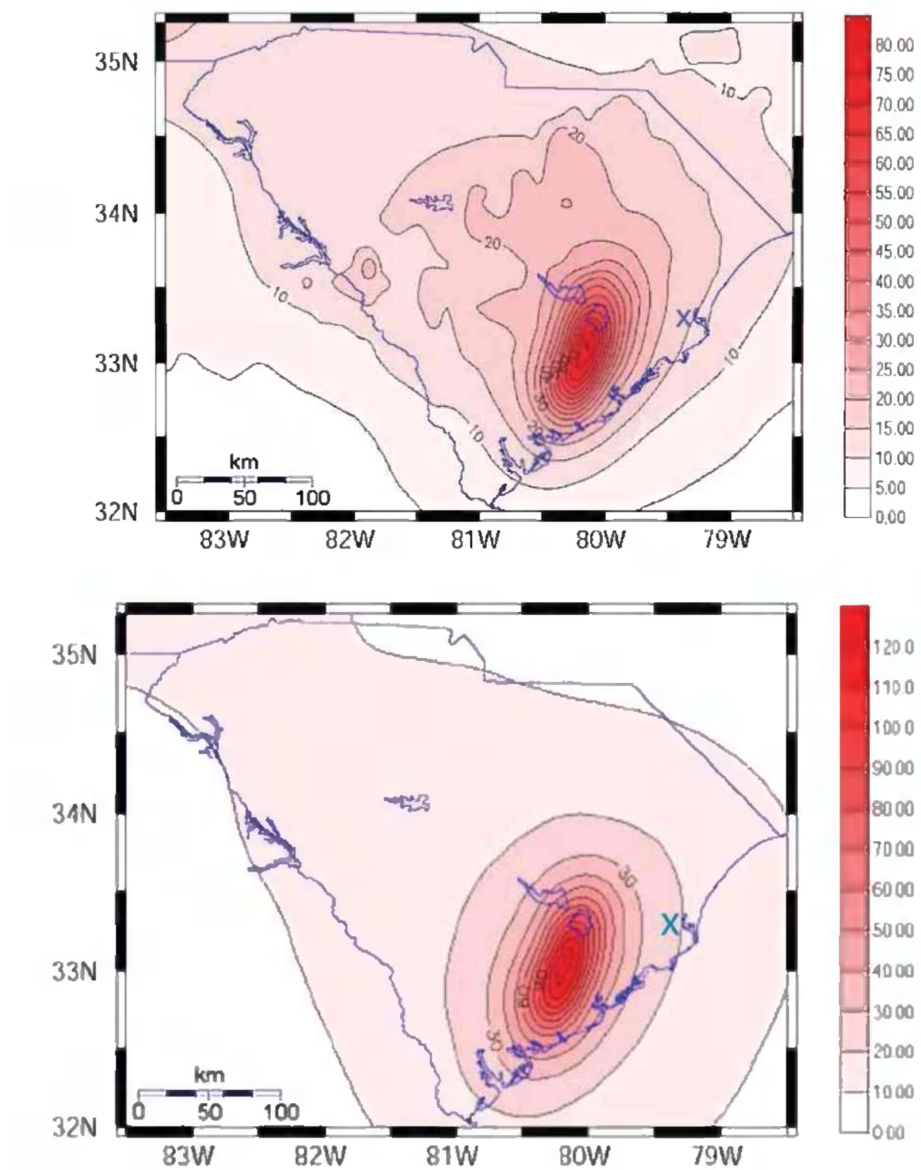

Figure 1-2. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (upper figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Site PGA based on Chapman and Talwani (2006) is approximately 0.16g for “geologically realistic” conditions and 0.21g for hard rock conditions.

Appendix 2

SCDOT Seismic Hazard Maps Used for Development of Target Design Spectra

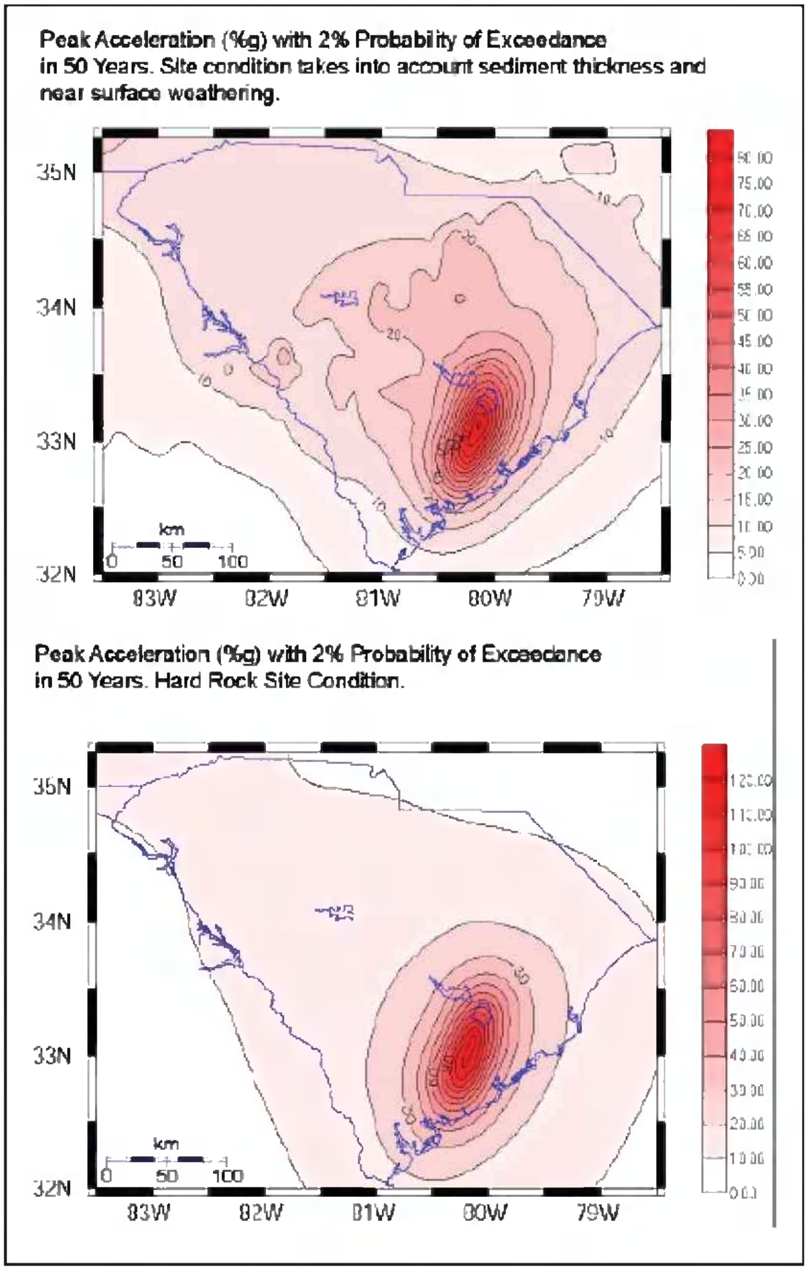

Figure 2-1. PGA (%) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

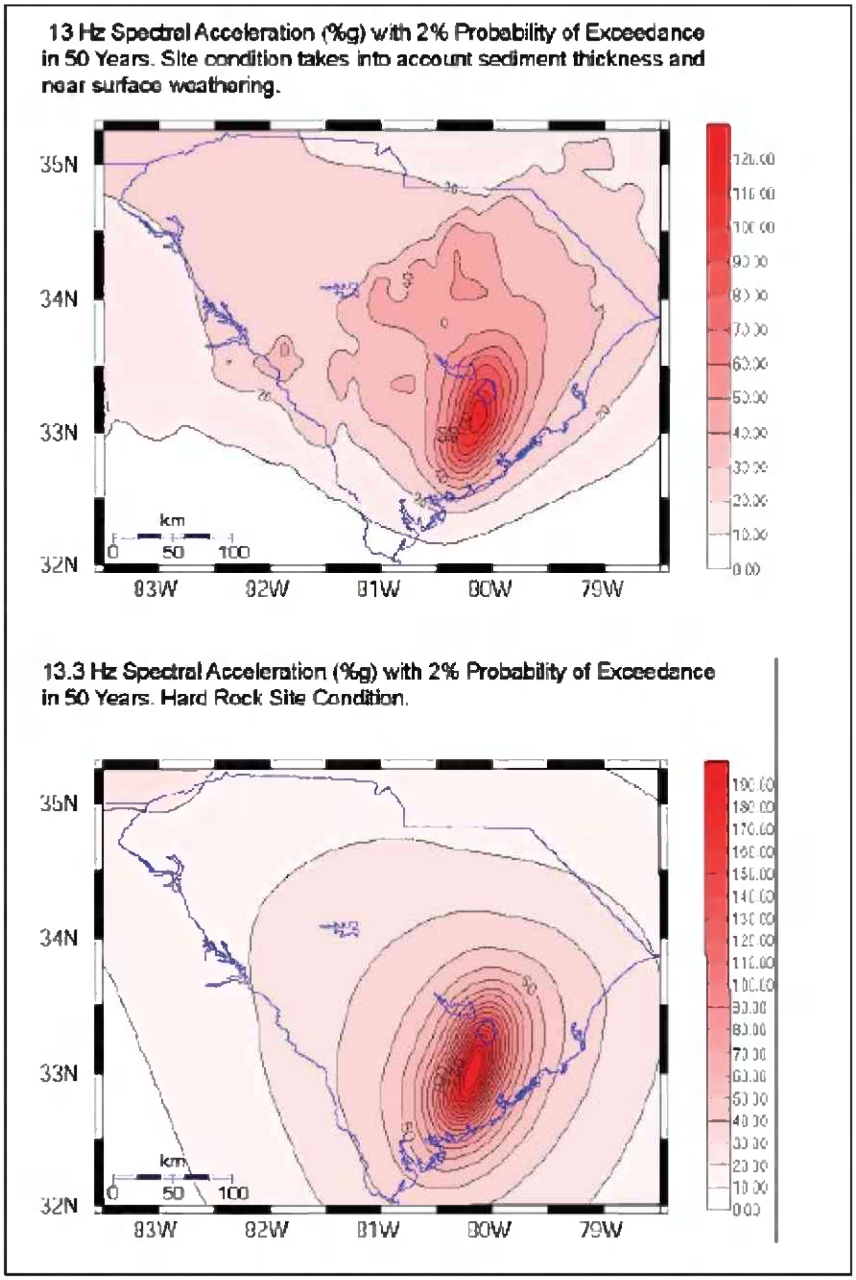

Figure 2-2. Spectral Acceleration (%) for 13 Hz (0.075 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

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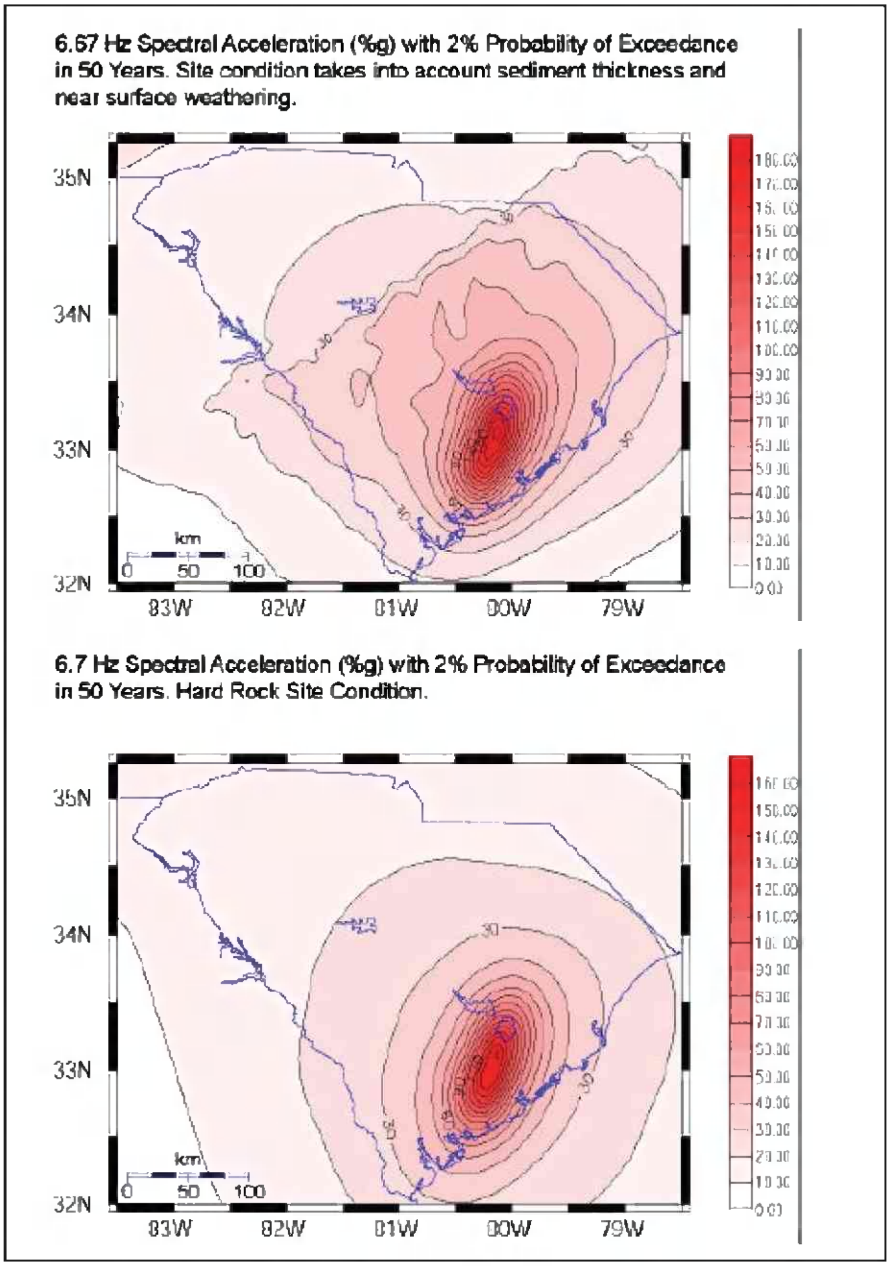

Figure 2-3. Spectral Acceleration (%) for 6.7 Hz (0.15 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

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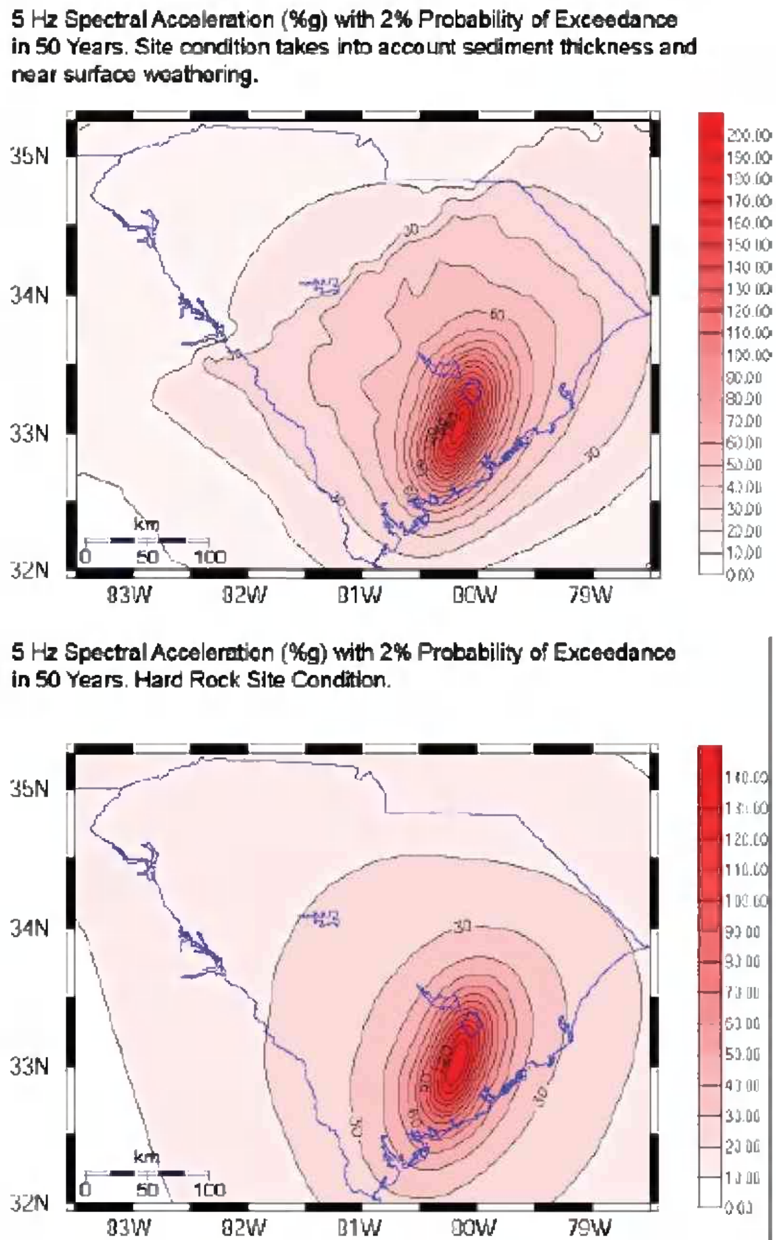

Figure 2-4. Spectral Acceleration (%) for 5 Hz (0.2 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

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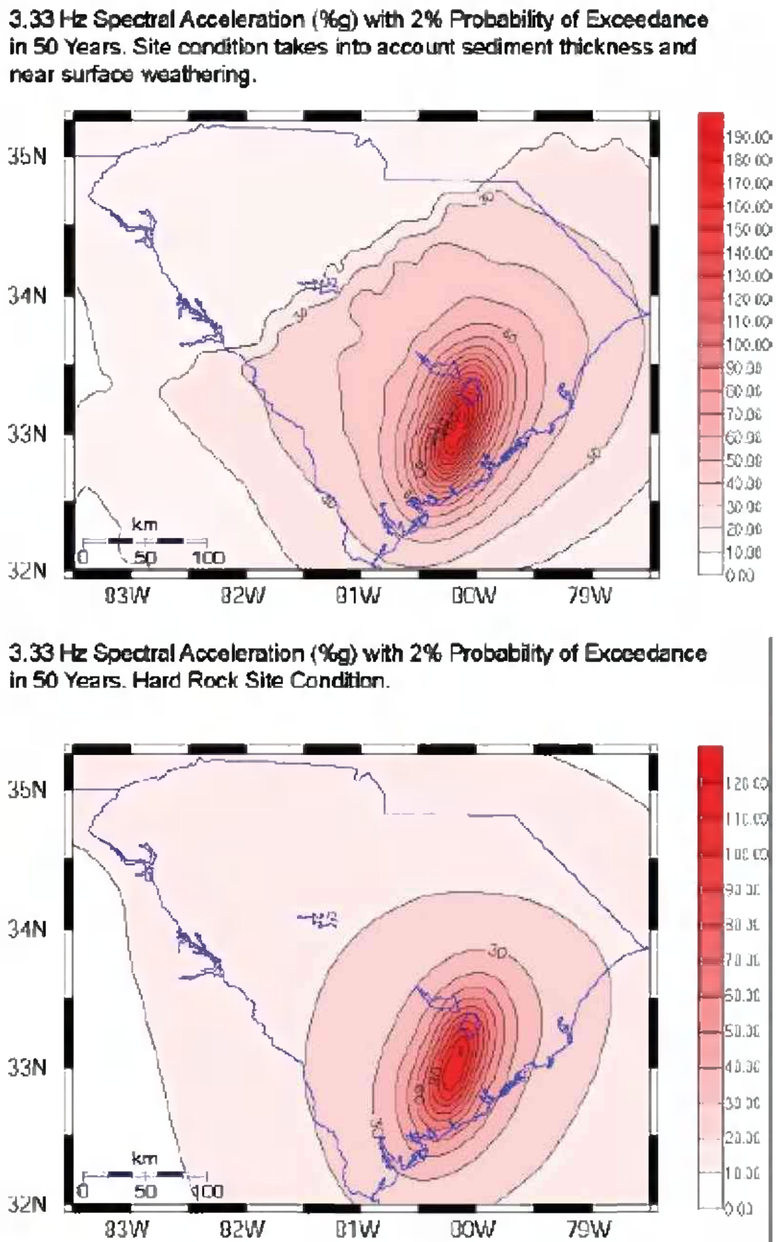

Figure 2-5. Spectral Acceleration (%) for 3.33 Hz (0.3 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

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

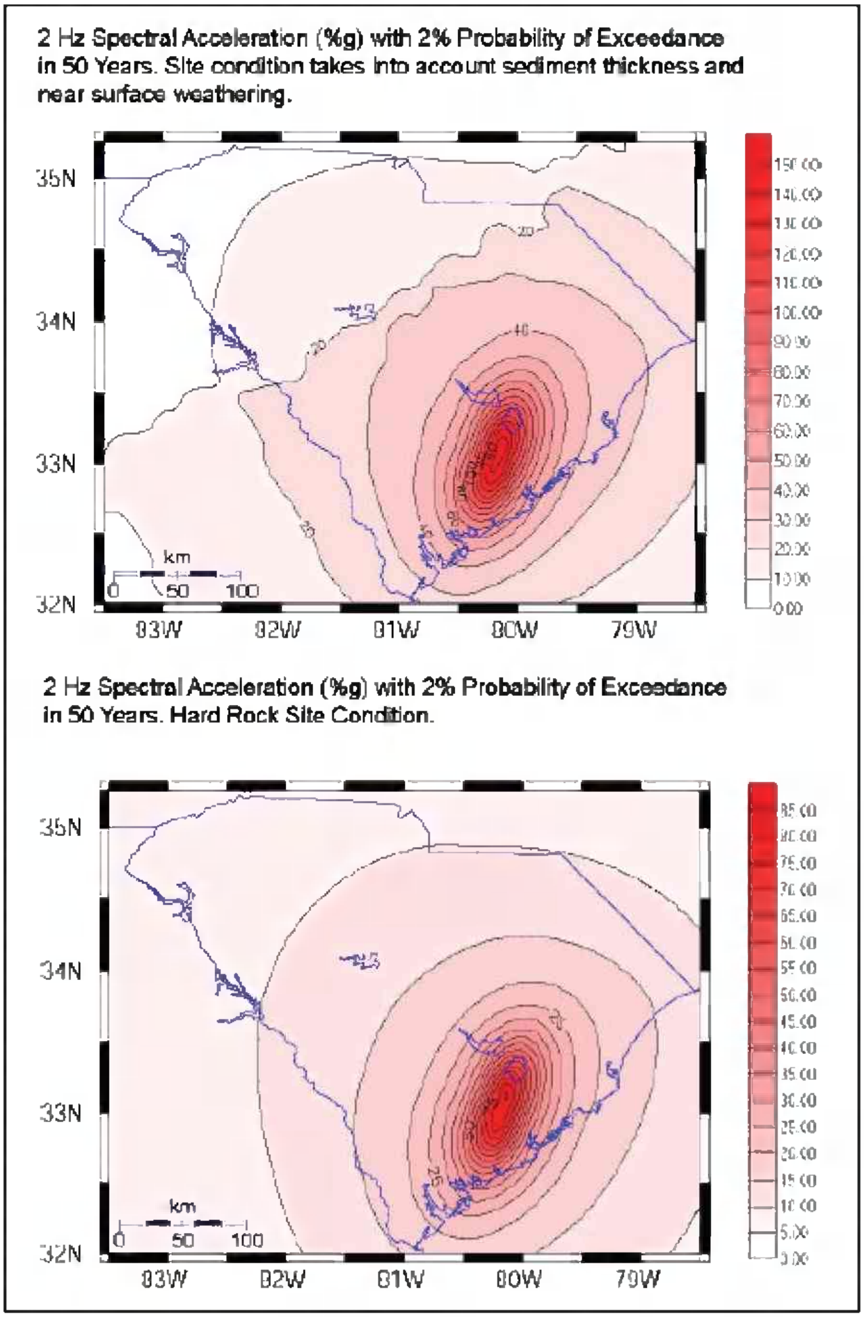

Figure 2-6. Spectral Acceleration (%) for 2 Hz (0.5 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

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

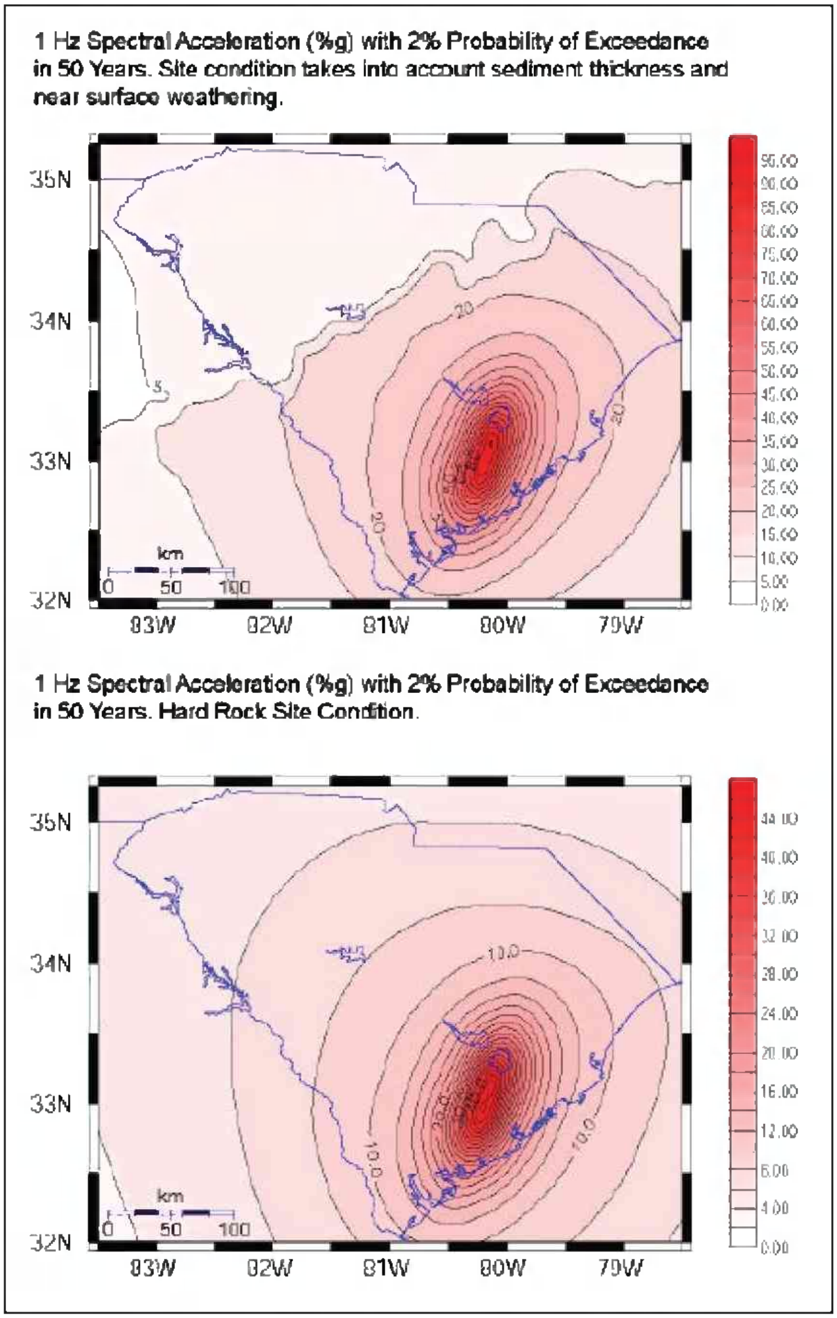

Figure 2-7. Spectral Acceleration (%) for 1 Hz (1 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

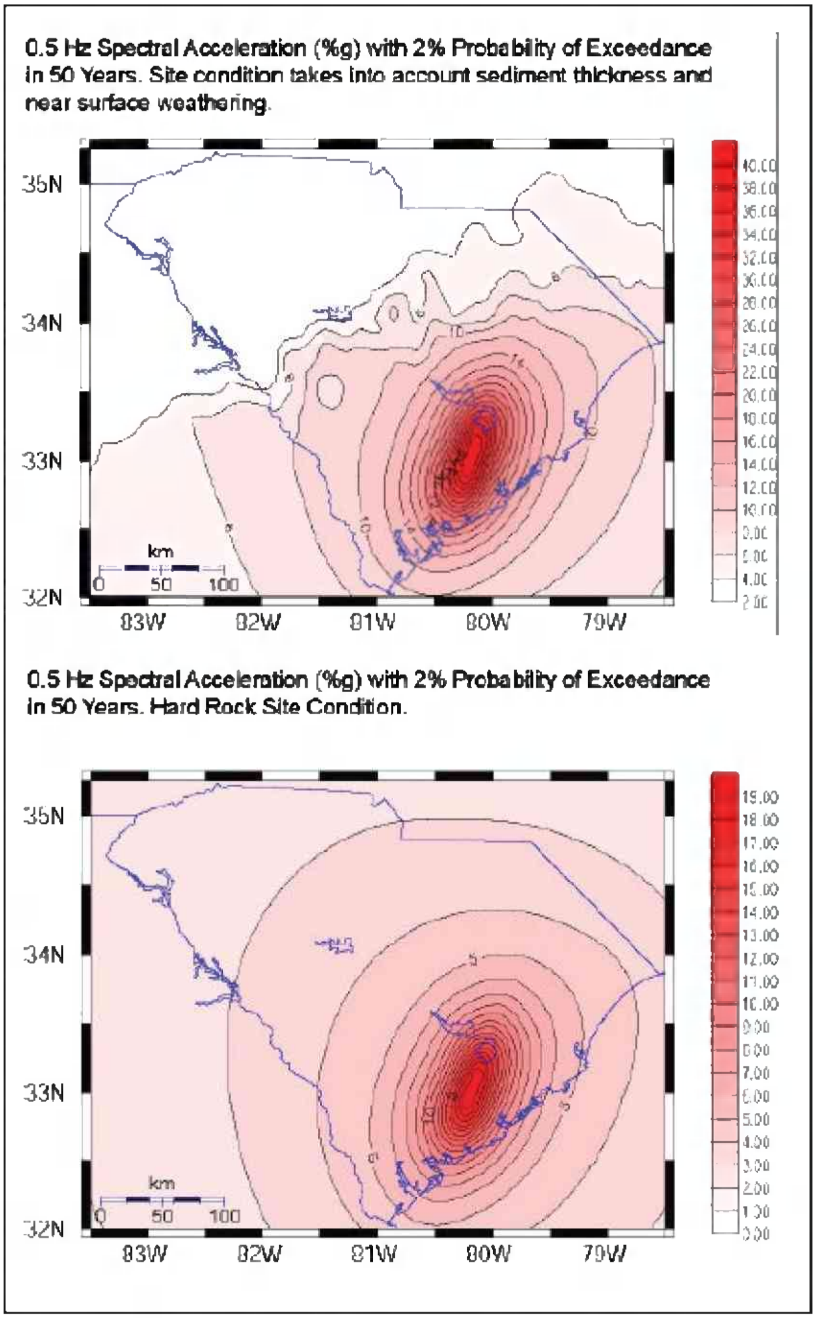

Figure 2-8. Spectral Acceleration (%) for 0.5 Hz (2 s Period) with 2 Percent Probability of Exceedance in 50 Years for Geologically Realistic Conditions (Upper Figure) and Hard Rock Conditions (Lower Figure) (Chapman and Talwani, 2006)

Note:

1. Refer to the figures in Appendix 1 for the site location.

Appendix 3

Selected Time Histories

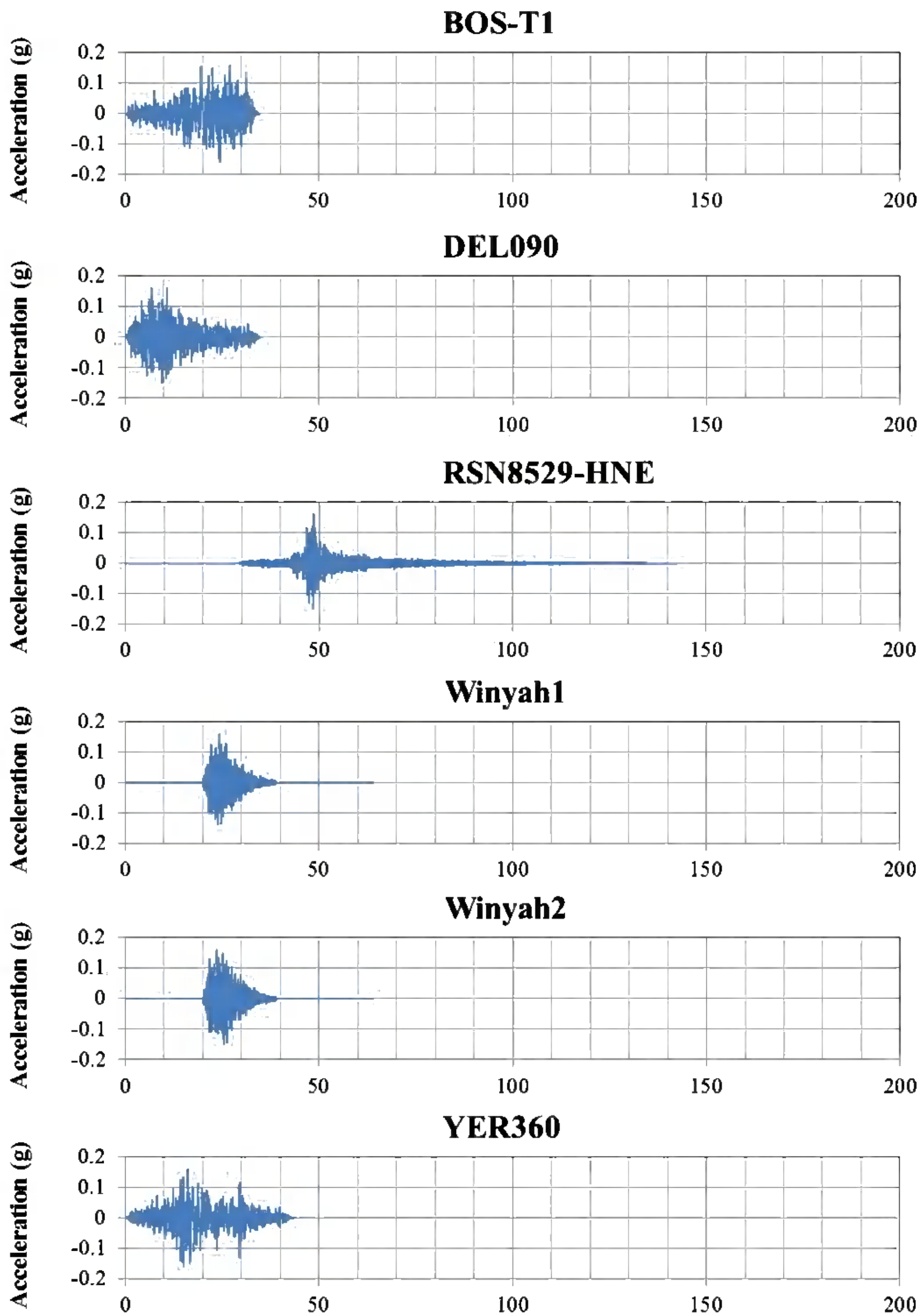

Figure 3-1. Acceleration Time Histories of Selected Earthquake Motions Scaled to a PGA of 0.16g.

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 3

(see Figure 4-1 for description on each step; see Figure 4-2 for the profile)

Step 1 – age of the soil layer: Pleistocene deposit.

Step 2 – soil type: sandy soils with PI=0; groundwater table @ 15 ft bgs.

Step 3 – calculate σ_m'@ mid-depth of the layer (36 ft bgs)

$\sigma_v' = \gamma H - \gamma_w H_w = 125\times36 - 62.4\times21 = 3189.6$ psf

$\sigma_m' = \sigma_v' (1+2K'_o)/3 = 3189.6\times(1+2\times0.47)/3 = 2062.8$ psf

($K_o' = 1 - \sin\phi' = 1 - \sin(32) = 0.47$, see Figure 4-3 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 upper and lower native soils in Profile 3.

Step 5 – select the parameters α, γ_{r1}, k from Figure 4-4.

$\gamma_{r1} = 0.018\%$, $\alpha = 1.00$, $k = 0.454$

Step 6 – compute the reference strain using SCDOT GDM Equation 12-20 (see Figure 4-3 for the equation).

$\gamma_r = \gamma_{r1} (\sigma_m'/P_a)^k = 0.018\times(2062.8/2089)^{0.454} = 0.0179\%$

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 $\gamma = 0.001\%$, $G/G_{max} = 1/[1+(0.001/0.0179)] = 0.947$
If $\gamma = 0.01\%$, $G/G_{max} = 1/[1+(0.01/0.0179)] = 0.642$
If $\gamma = 0.1\%$, $G/G_{max} = 1/[1+(0.1/0.0179)] = 0.152$

Damping Curve for the upper native soil in Profile 3

(see Figure 4-5 for description on each step; see Figure 4-2 for the profile)

Steps 1 through 4 are the same as those for modulus reduction curve development.

Step 5 – select small-strain material damping @ $\sigma_m' = 1$ atm, D_{min1} from Figure 4-6.

$D_{min1} = 0.59\%$

Step 6 – compute the small strain material damping, D_{min}, using SCDOT GDM Equation 12-28 (see Figure 4-7 for the equation).

$D_{min} = D_{min1} (\sigma_m'/P_a)^{-0.5k} = 0.59\times(2062.8/2089)^{-0.5\times0.454} = 0.592\%$

Step 7-9 – instead of taking Steps 7 through 9, use SCDOT GDM Equation 12-29 to compute damping ratio curve (D).

$D = 12.2\ (G/G_{max})^2 - 34.2\ (G/G_{max}) + 22.0 + D_{min}$

If $\gamma = 0.001\%$, $D = 12.2\times(0.947)^2 - 34.2\times(0.947) + 22.0 + 0.592 = 1.14\%$
If $\gamma = 0.01\%$, $D = 12.2\times(0.642)^2 - 34.2\times(0.642) + 22.0 + 0.592 = 5.67\%$
If $\gamma = 0.1\%$, $D = 12.2\times(0.152)^2 - 34.2\times(0.152) + 22.0 + 0.592 = 17.68\%$

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 geologic unit using Equation 12-21
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select the appropriate values for each layer 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 geologic unit (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each layer 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)

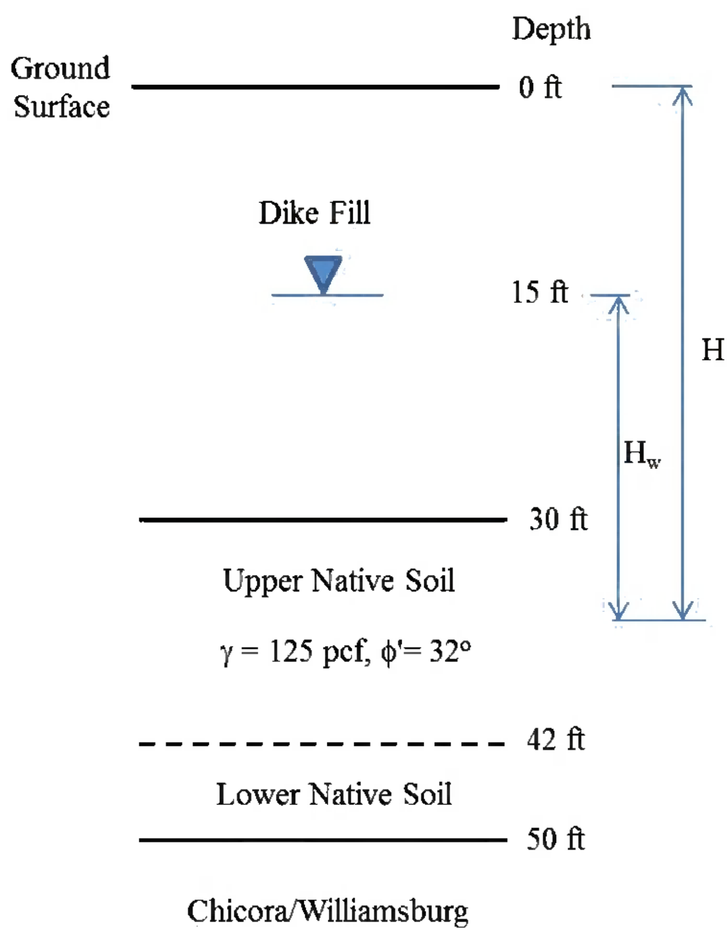

Figure 4-2. Profile 3 for the Example Calculations

$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}$$ 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 ≤ 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)

Written by: **J. McNash / C. Carlson** Date: **10/12/2016** Reviewed by: **W. Shin/G. Rix** Date: **10/12/2016**

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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	60	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	0.362 [2]	—
Tertiary (Stiff Upland Soils)	γ_{r1} (%)	—	—	0.023	0.041 [2]	—	—
	α	—	—	1.00	1.00 [2]	—	—
	k	—	—	0.102	0.045 [2]	—	—
Tertiary (All soils at SRS except Stiff Upland Soils)	γ_{r1} (%)	0.038	0.058	0.079	0.108	0.174 [2]	—
	α	1.00	1.00	1.00	1.00	1.00 [2]	—
	k	0.277	0.240	0.208	0.172	0.106 [2]	—
Tertiary (Tobacco Road, Snapp)	γ_{r1} (%)	0.029	0.056	0.082	0.117	0.205 [1]	—
	α	1.00	1.00	1.00	1.00	1.00 [1]	—
	k	0.220	0.185	0.156	0.124	0.070 [1]	—
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	γ_{r1} (%)	0.047	0.059	0.071	0.088	0.125 [1]	—
	α	1.00	1.00	1.00	1.00	1.00 [1]	—
	k	0.313	0.299	0.285	0.268	0.229 [1]	—
Residual Soil and Saprolite	γ_{r1} (%)	0.040	0.066	0.093 [1]	0.129 [1]	—	—
	α	0.72	0.80	0.89	1.01 [1]	—	—
	k	0.202	0.141	0.099	0.061 [2]	—	—

[1] SRS = Savannah River Site
[2] Tentative Values – Andrus et al. (2003)

Figure 4-4. Recommended Parameters for South Carolina Soils (Table 12-15 of SCDOT, 2010)

Table 12-18, Procedure for Computing Damping Ratio

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select appropriate small-strain material Damping @ σ'_m = 1 atm, D_{min1}, from Table 12-17 for each layer within a geologic unit.
6	Compute the small-strain material Damping, D_{min}, for each layer within a geologic unit using Equation 12-28.
7	Select the appropriate values for each layer of reference strain, γ_{r1}, @ σ'_m = 1atm , curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
8	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit that has a corresponding average σ'_m.
9	Compute the design equivalent viscous damping ratio curves (*D*) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, and small-strain material Damping, D_{min}, for each layer using Equation 12-30. Tabulate values of Soil Damping Ratio, *D*, with corresponding shear strain, γ, for use in a site-specific site response analysis.

Figure 4-5. Procedure for Development of Region-Specific Damping Curve (SCDOT, 2010)

Table 12-17, Recommended Value D_{min1} (%) for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Plasticity Index, *PI* (%)					
	0	15	30	50	100	150
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 [1]
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	---	---	1.14 [1]	1.52 [1]	2.49 [1]	---
Tertiary (Stiff Upland Soils)	---	---	0.98	1.42 [1]	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	0.68	0.94	1.19	1.53	2.37 [1]	---
Tertiary (Tobacco Road, Snapp)	0.68	0.94	1.19	1.53	2.37 [1]	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	0.68	0.94	1.19	1.53	2.37 [1]	---
Residual Soil and Saprolite	0.56 [1]	0.85 [1]	1.14 [1]	1.52 [1]	---	---

[1] Tentative Values – Andrus et al. (2003)

Figure 4-6. Recommended D_{min1} for South Carolina Soils (Table 12-17 of SCDOT, 2010)

$$D_{min} = D_{min1}(\sigma'_m/P_a)^{-0.5k} \qquad \text{Equation 12-28}$$

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)

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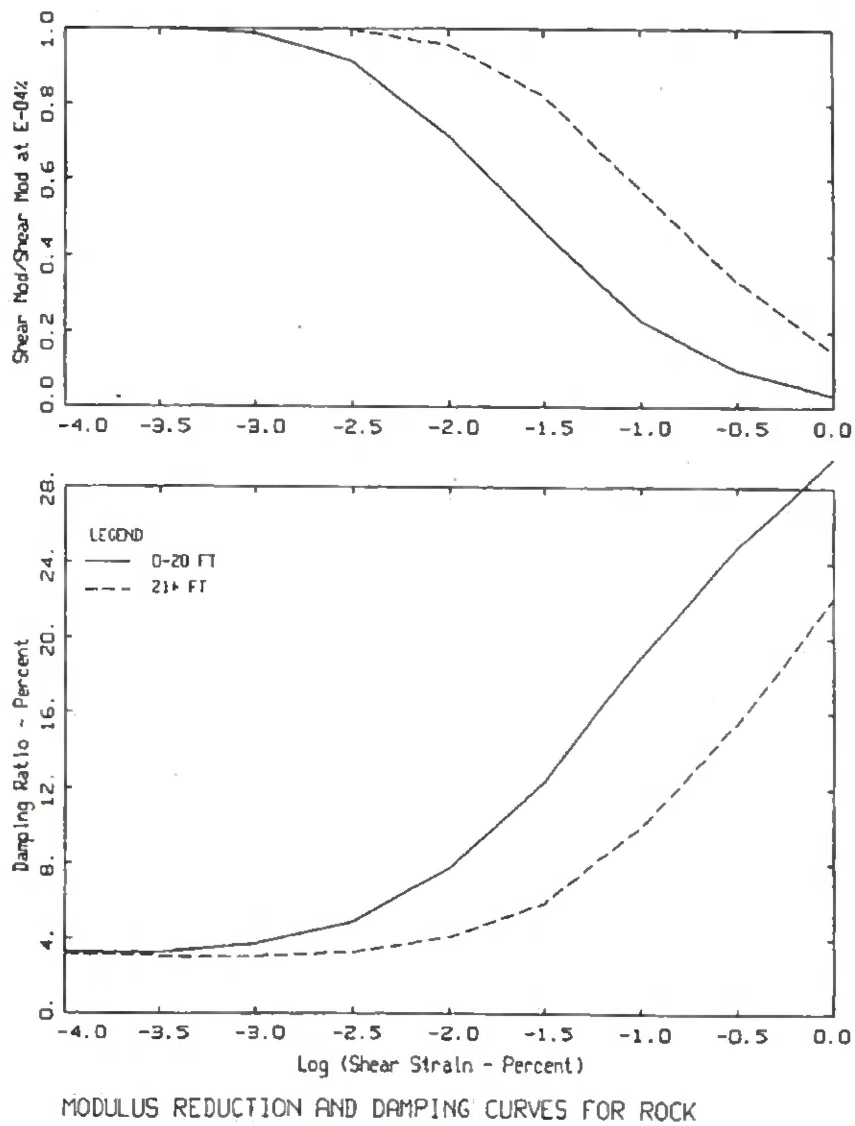

Figure 5b. Generic G/Gmax and hysteretic damping curves for soft rock (Silva et al., 1997).

Figure 4-8. Shear Modulus Reduction and Damping Curves for Chicora/Williamsburg Formation (S&ME, 2001)

Appendix 5

Shear Wave Velocity Profile Selection

Written by: **J. McNash / C. Carlson** Date: **10/12/2016** Reviewed by: **W. Shin/G. Rix** Date: **10/12/2016**

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

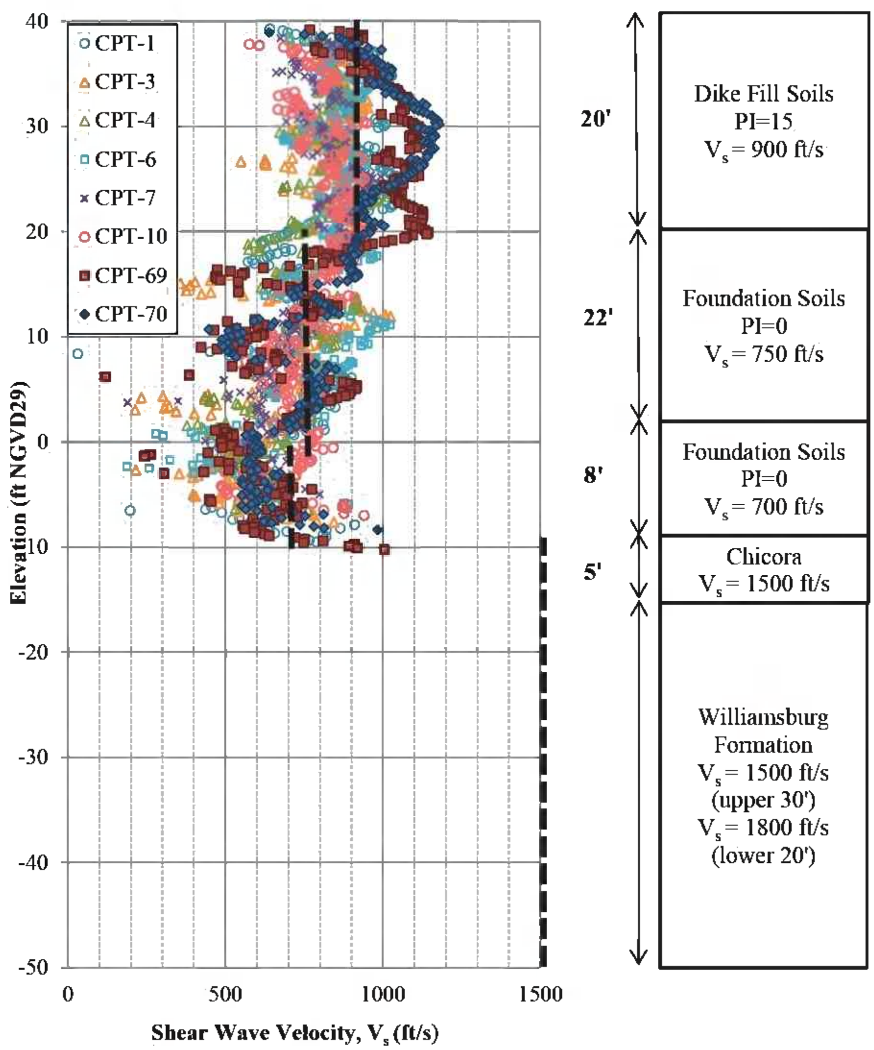

Figure 5-1a. Selected V_s Profile for the Section 1 Dike Model (Profile 1)

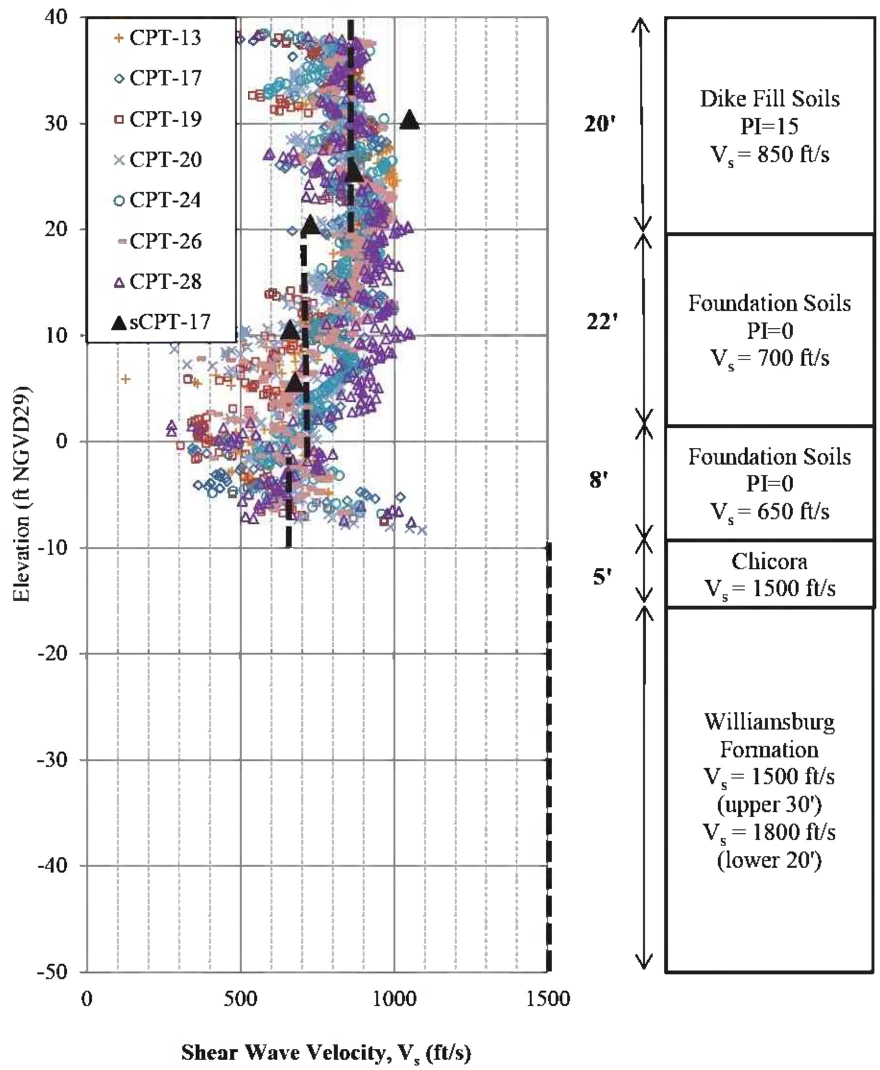

Figure 5-1b. Selected V_s Profile for the Section 2 Dike Model (Profile 2)

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

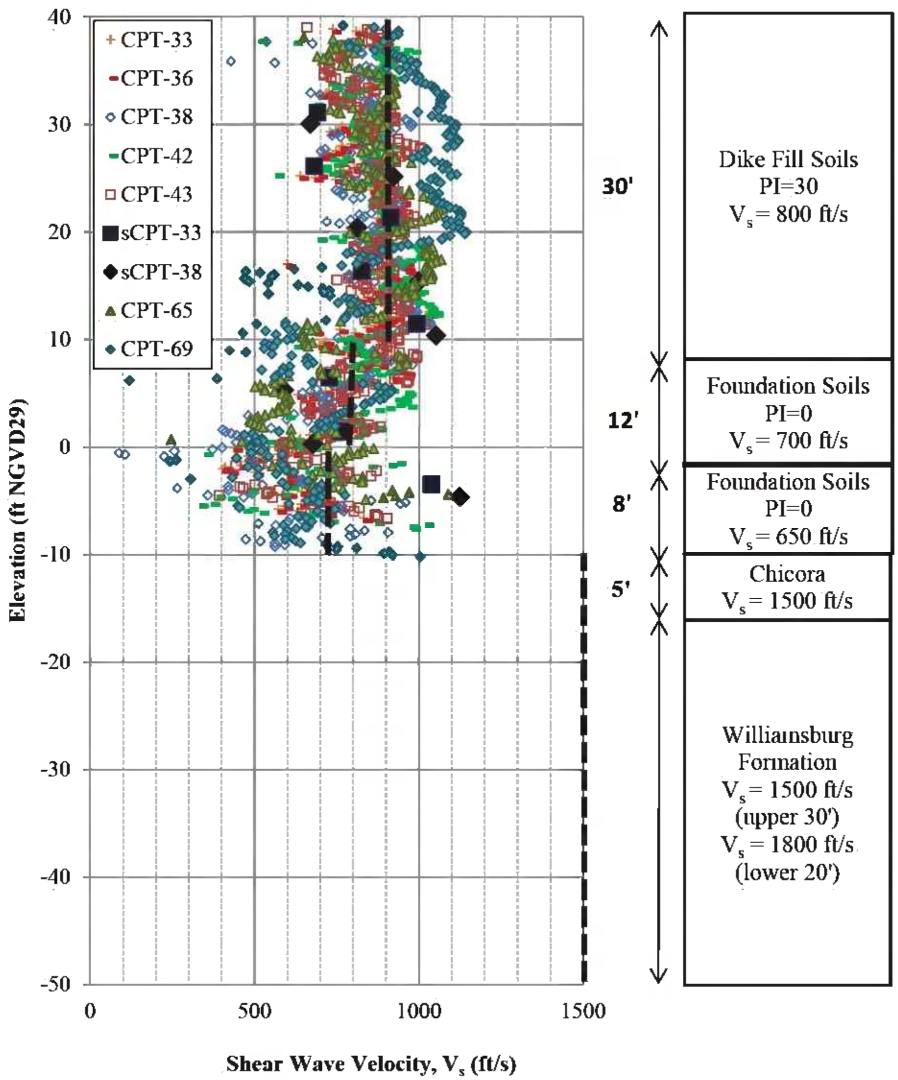

Figure 5-1c. Selected V_s Profile for the Section 3 Dike Model (Profile 3)

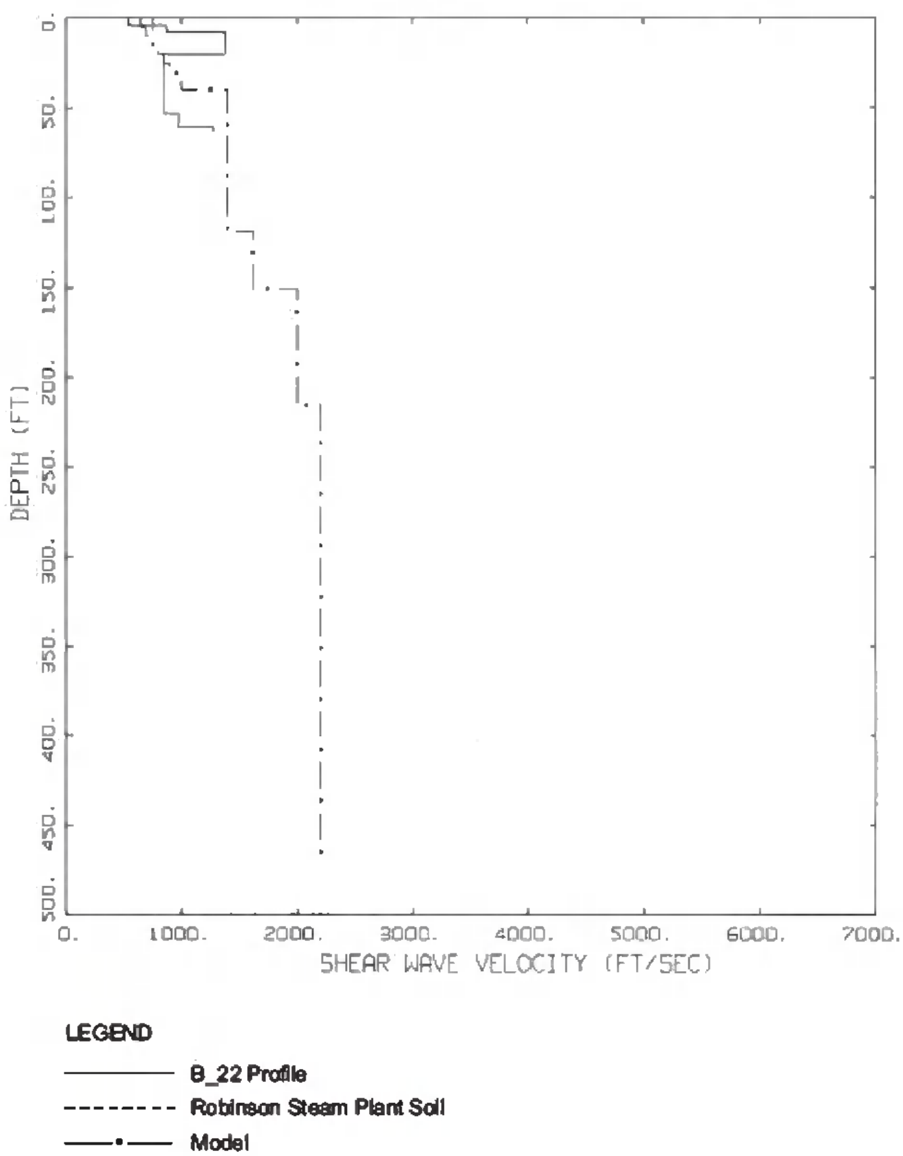

Figure 3-9. Base case shear-wave velocity profile for the Myrtle Beach site response category along with available profiles.

Figure 5-2. Reference V_s Profile for Chicora/Williamsburg Formation (URS, 2001)

Appendix 6

Calculated Acceleration Profiles

Written by: J. McNash / C. Carlson Date: 10/12/2016 Reviewed by: W. Shin/G. Rix Date: 10/12/2016

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

Profile 1

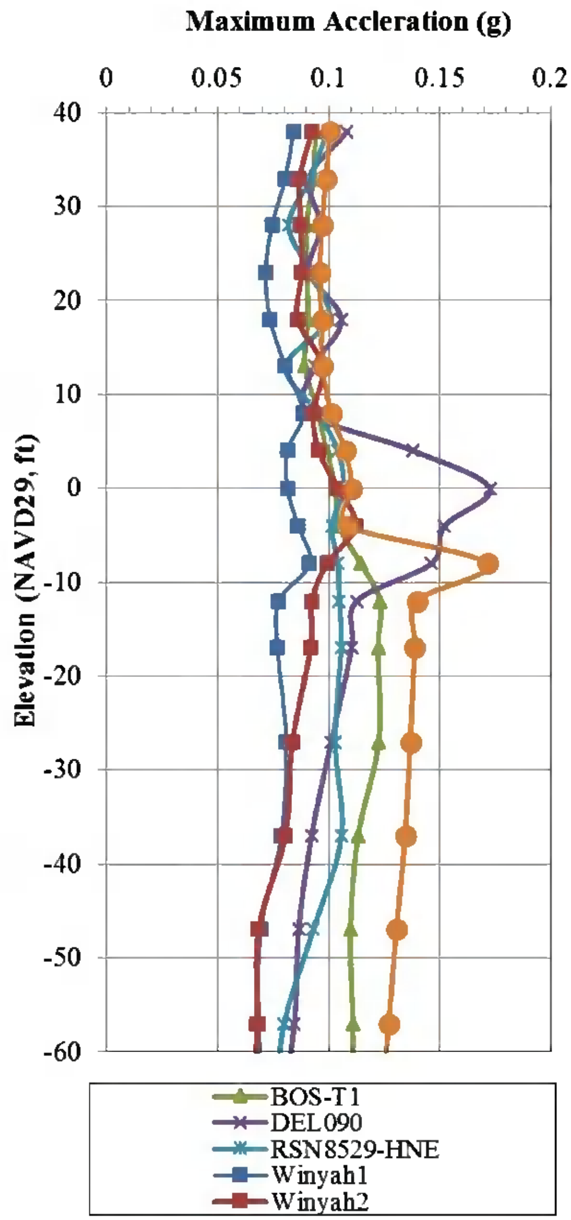

Figure 6-1. Calculated Maximum Acceleration for Profile 1

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.16g.

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

Profile 2

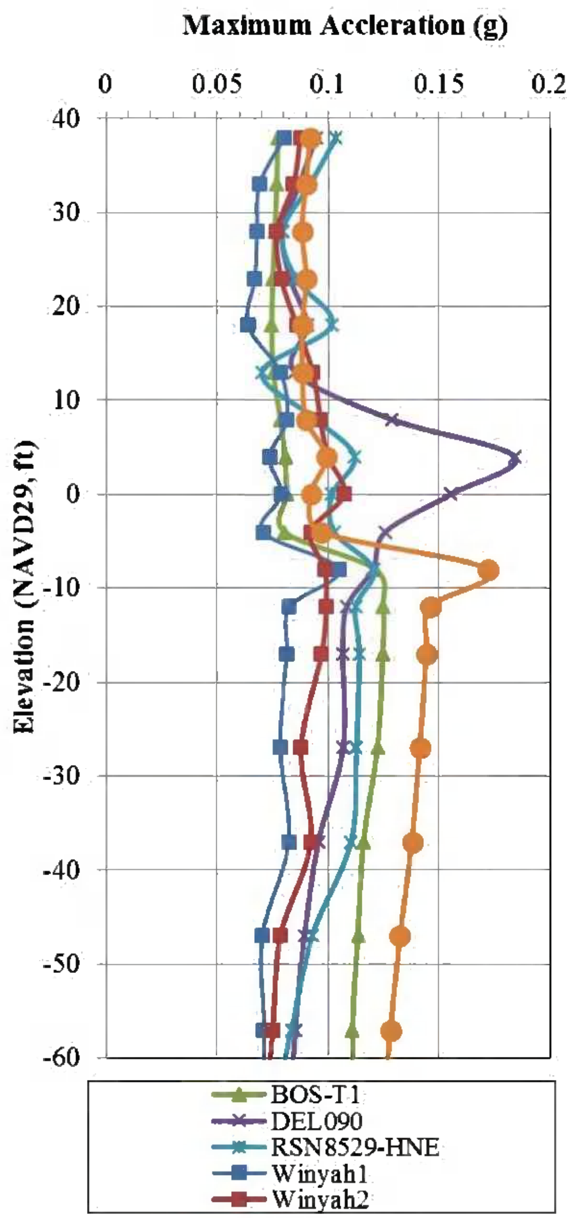

Figure 6-2. Calculated Maximum Acceleration for Profile 2

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.16g.

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

Profile 3

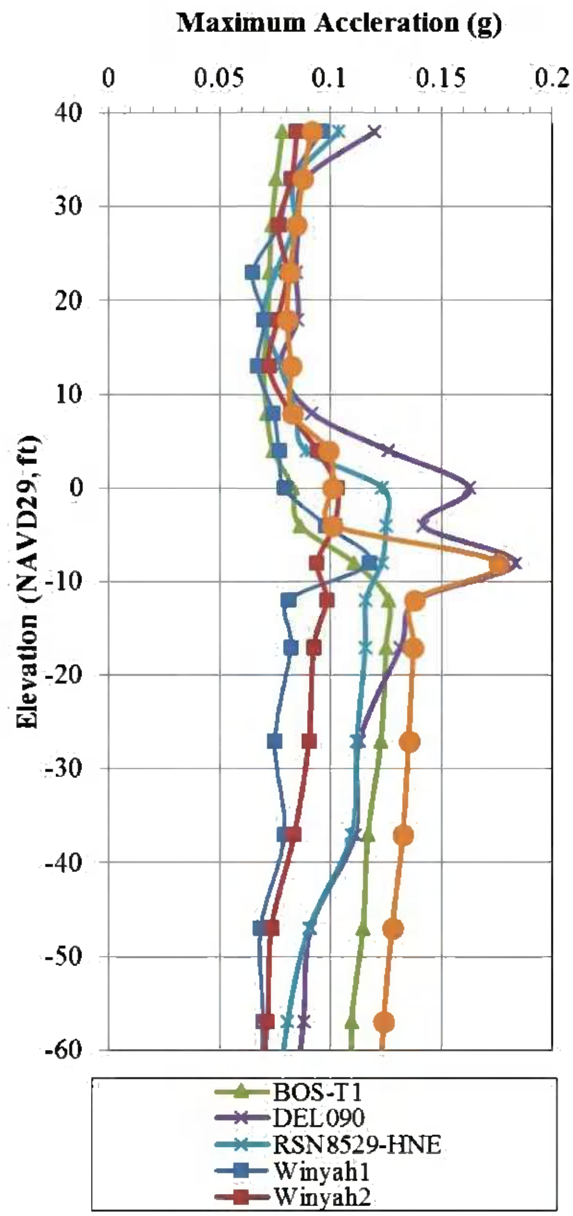


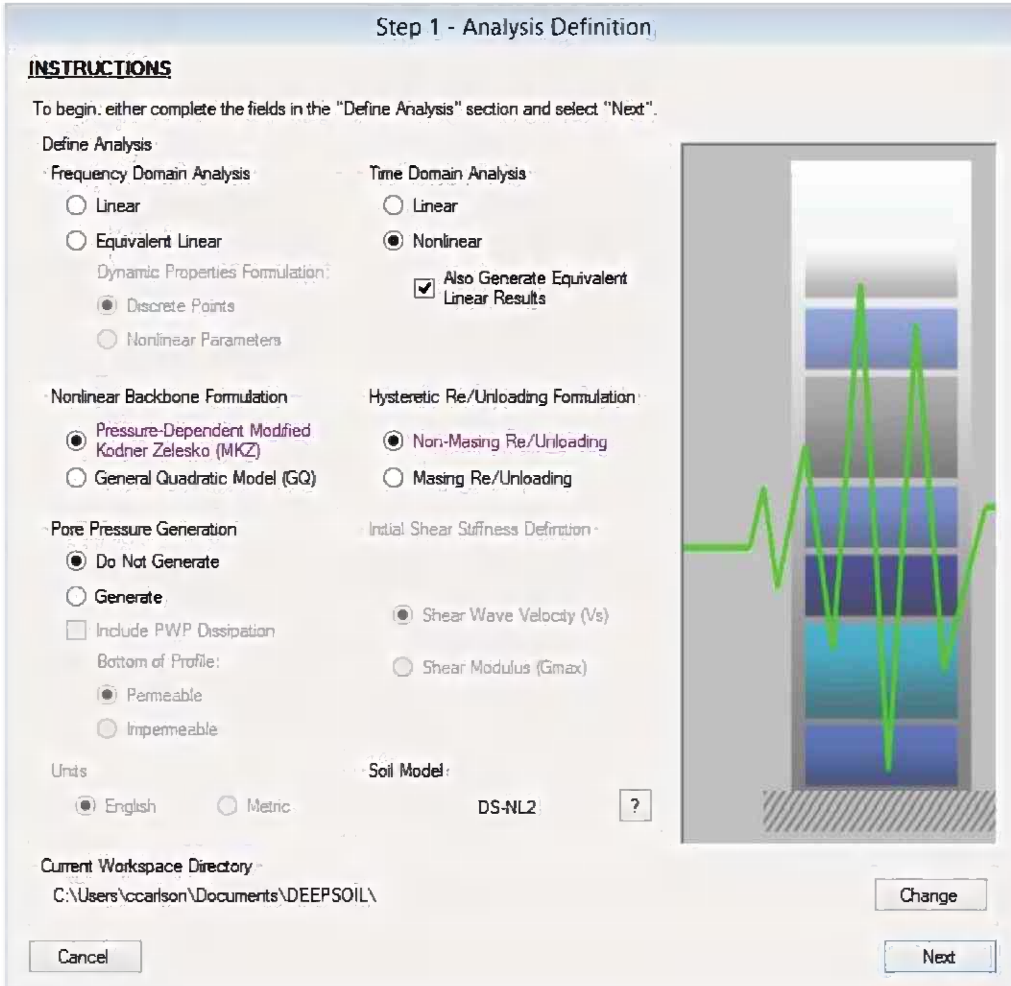
Figure 6-3. Calculated Maximum Acceleration for Profile 3

Notes:

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\
Change
Cancel
Next

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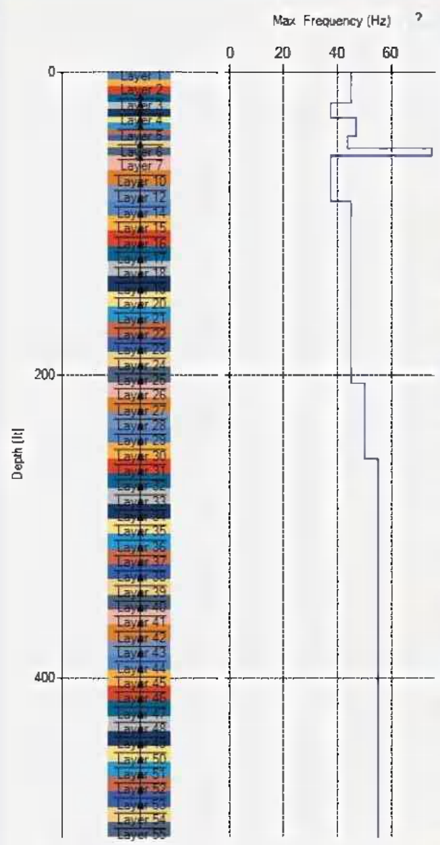
Step 2a - Soil Profile Definition



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	900	1.62324811862373	0.1126	0.18	1.545	0.96	0	0	0.632	0.236	3.25
2	Dike	5	125	900	1.45324974129461	0.1406	0.18	1.545	0.96	0	0	0.63	0.232	3.25
3	Dike	5	125	900	1.38033790786345	0.1574	0.18	1.56	0.96	0	0	0.63	0.232	3.25
	Dike	5	125	900	1.34440279656889	0.1626	0.18	1.53	0.96	0	0	0.626	0.226	3.25
	Foundation Soils	5	125	750	0.618685839156217	0.024	0.18	1.545	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	5	125	750	0.601754375623372	0.0254	0.18	1.545	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	4	125	750	0.588509193544698	0.0248	0.18	1.44	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	4	125	750	0.577935689976007	0.0268	0.18	1.5	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	4	125	750	0.568310874060151	0.028	0.18	1.515	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	4	125	700	0.559545239776579	0.0292	0.18	1.53	1.005	0	0	0.618	0.26	3.25
	Foundation Soils	4	125	700	0.551452632040576	0.0292	0.18	1.485	1.005	0	0	0.618	0.26	3.25
	Chicora	5	130	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.16	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85

Soil Profile Display
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.86 Hz
Natural Period of Profile: 1.17 sec

Soil Profile
Material Properties
Add Layer(s)
Remove Layer

Water Table Location
Top of Layer: 4
No Water Table

Spreadsheet Legend
Below Water Table
Layer Properties
Material Properties

Conversion Tools
Units: English to Metric
Shear: Velocity to Modulus

Back

Next

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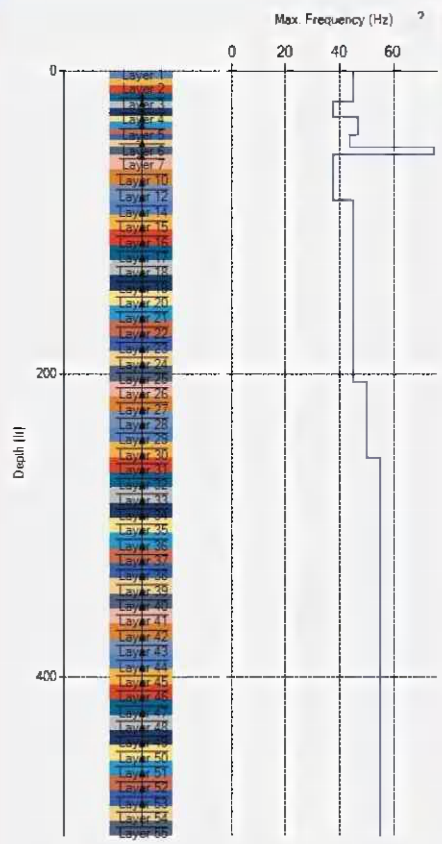
Step 2a - Soil Profile Definition



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
31	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
32	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
33	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
34	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
35	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
36	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
37	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
38	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
39	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
40	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
41	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
42	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
43	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
44	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
45	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
46	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
47	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
48	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
49	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
50	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
51	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
52	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
53	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
54	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
55	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
56	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
57	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65

Soil Profile Display
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.86 Hz
Natural Period of Profile: 1.17 sec

Soil Profile
Material Properties
Add Layer(s)
Remove Layer

Water Table Location
Top of Layer: 4
No Water Table

Spreadsheet Legend
Below Water Table
Layer Properties
Material Properties

Conversion Tools
Units: English to Metric
Shear: Velocity to Modulus

Back

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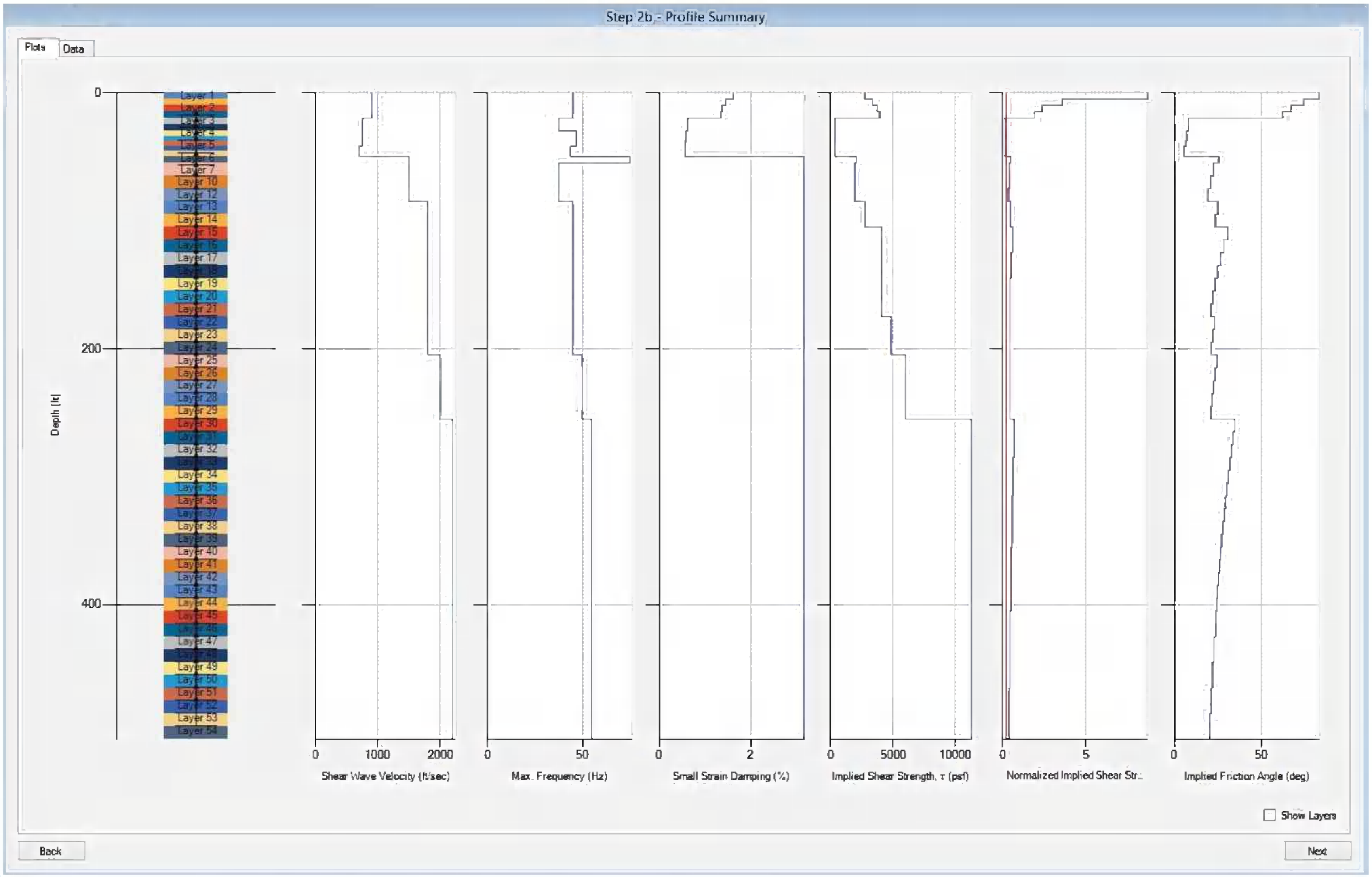
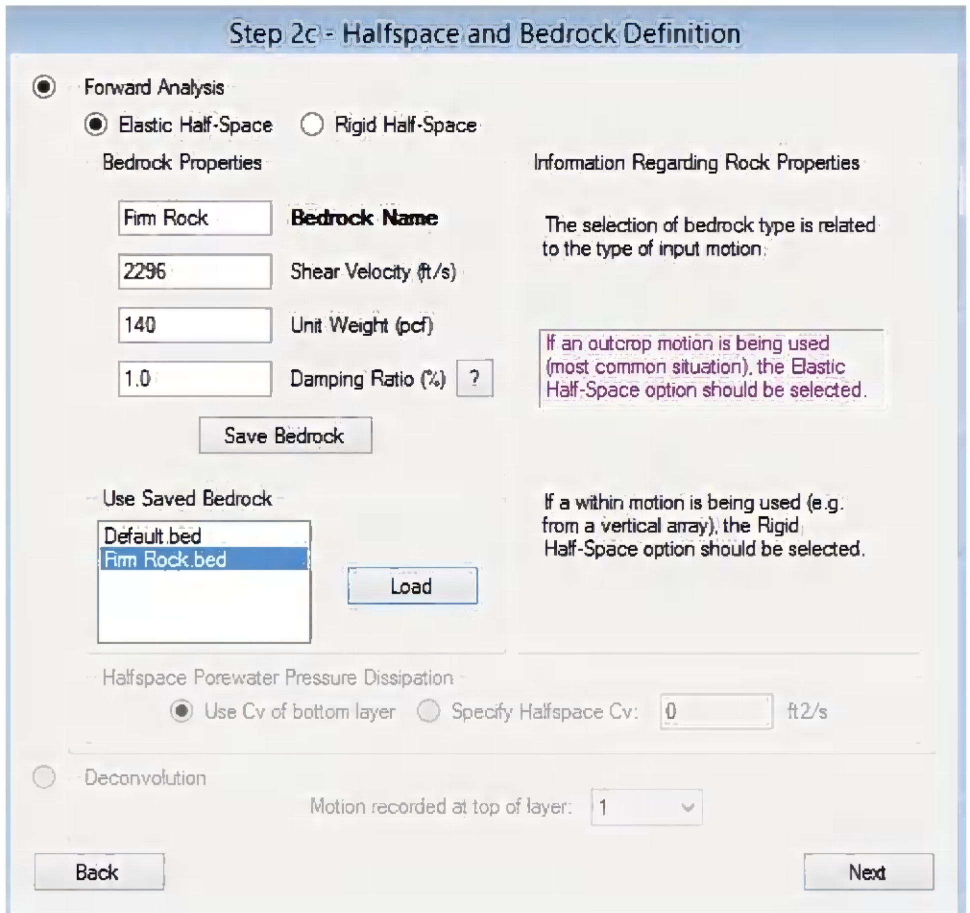
Written by:	**J. McNash / C. Carlson**	Date:	**10/12/2016**	Reviewed by:	**W. Shin/G. Rix**	Date:	**10/12/2016**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GSC5242**	Task No:	**01BT**



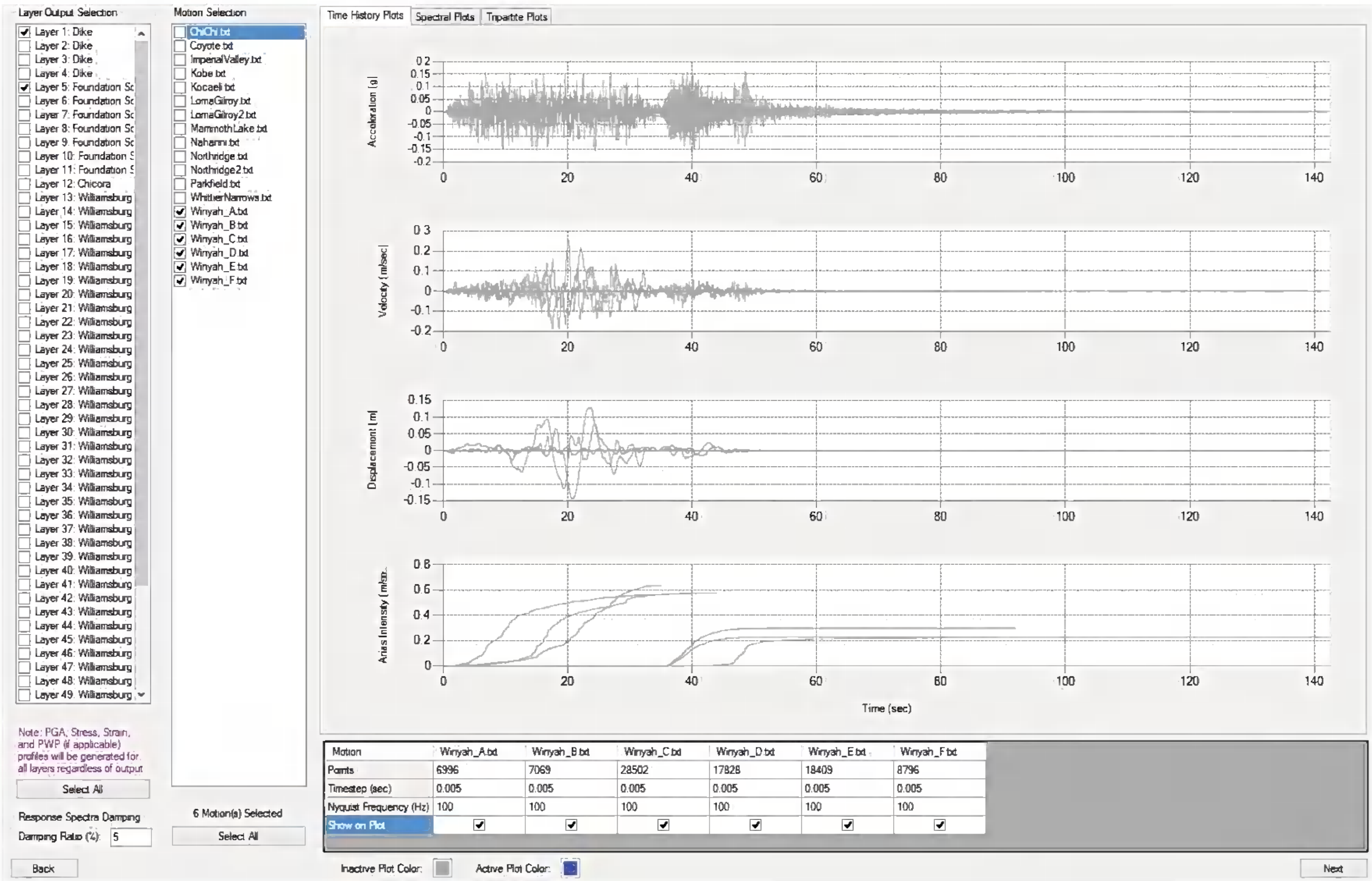
Written by: **J. McNash / C. Carlson** Date: **10/12/2016** Reviewed by: **W. Shin/G. Rix** Date: **10/12/2016**

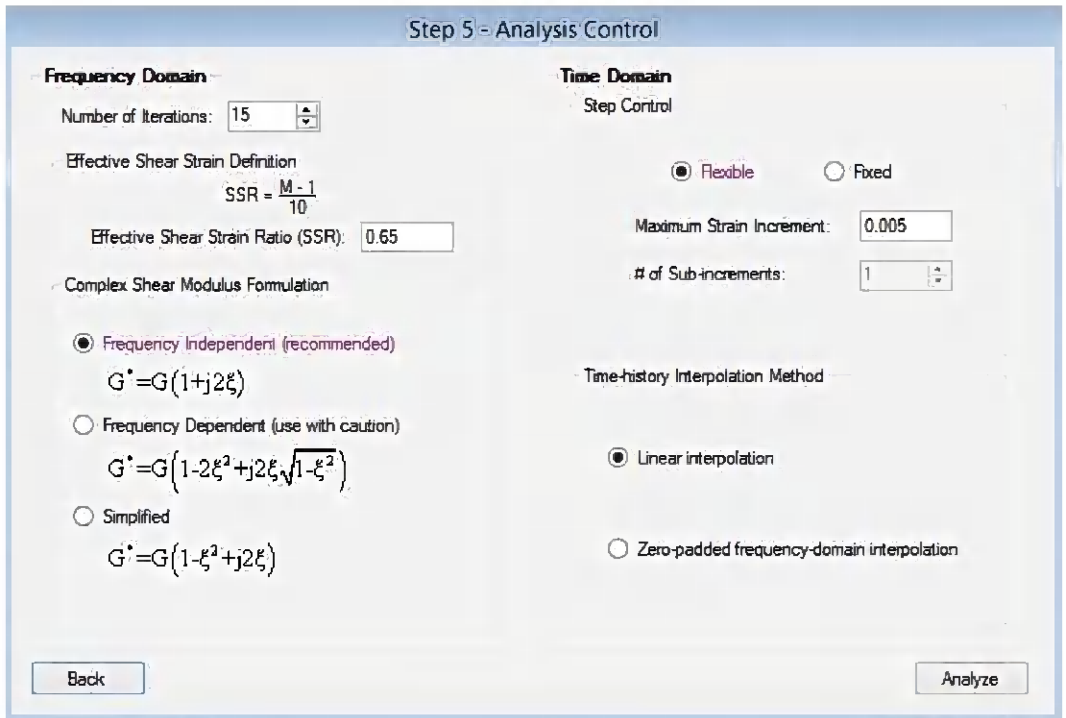
Client: **Santee Cooper** Project: **Winyah Generating Station** Project/ Proposal No.: **GSC5242** Task No: **01BT**



Written by: **J. McNash / C. Carlson** Date: **10/12/2016** Reviewed by: **W. Shin/G. Rix** Date: **10/12/2016**

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

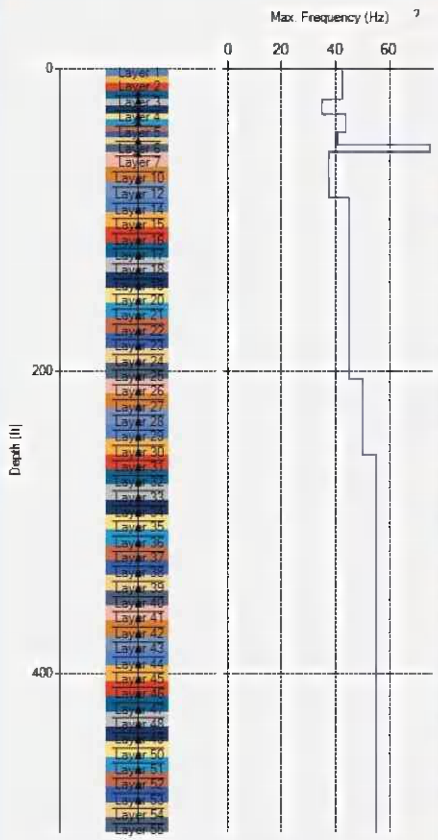



Step 5 - Analysis Control
Frequency Domain
Number of Iterations: 15
Effective Shear Strain Definition
SSR = (M - 1)/10
Effective Shear Strain Ratio (SSR): 0.65
Complex Shear Modulus Formulation
Frequency Independent (recommended)
G* = G(1+j2ξ)
Frequency Dependent (use with caution)
G* = G(1-2ξ²+j2ξ√(1-ξ²))
Simplified
G* = G(1-ξ²+j2ξ)
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
Back
Analyze

Written by: J. McNash / C. Carlson Date: 10/12/2016 Reviewed by: W. Shin/G. Rix Date: 10/12/2016

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

Step 2a - Soil Profile Definition



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
1	Dike	5	125	850	1.62324811862373	0.1126	0.18	1.545	0.96	0	0	0.632	0.236	3.25
2	Dike	5	125	850	1.45324974129461	0.1406	0.18	1.545	0.96	0	0	0.63	0.232	3.25
3	Dike	5	125	850	1.38033790786345	0.1574	0.18	1.56	0.96	0	0	0.63	0.232	3.25
[illegible]	Dike	5	125	850	1.34440279656889	0.1626	0.18	1.53	0.96	0	0	0.626	0.226	3.25
[illegible]	Foundation Soils	5	125	700	0.618685839156217	0.024	0.18	1.545	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	5	125	700	0.601754375623372	0.0254	0.18	1.545	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	700	0.588509193544698	0.0248	0.18	1.44	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	700	0.577935689976007	0.0268	0.18	1.5	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	700	0.568310874060151	0.028	0.18	1.515	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	650	0.559545239776579	0.0292	0.18	1.53	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	650	0.551452632040576	0.0292	0.18	1.485	1.005	0	0	0.618	0.26	3.25
[illegible]	Chicora	5	130	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85

Soil Profile Display
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.84 Hz
Natural Period of Profile: 1.18 sec

Soil Profile
Material Properties
Add Layer(s)
Remove Layer

Water Table Location
Top of Layer: 4
No Water Table

Spreadsheet Legend
Below Water Table
Layer Properties
Material Properties

Conversion Tools
Units: English to Metric
Shear: Velocity to Modulus

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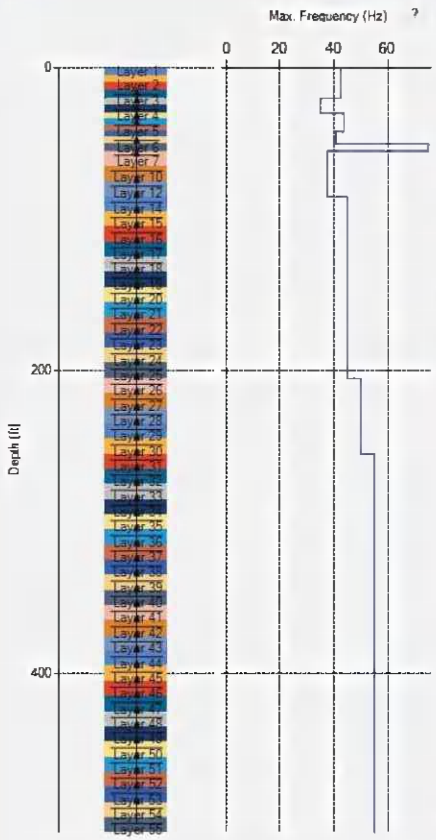
Written by: J. McNash / C. Carlson Date: 10/12/2016 Reviewed by: W. Shin/G. Rix Date: 10/12/2016

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

Step 2a - Soil Profile Definition



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
31	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
32	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
33	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
34	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
35	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
36	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
37	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
38	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
39	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
40	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
41	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
42	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
43	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
44	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
45	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
46	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
47	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
48	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
49	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
50	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
51	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
52	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
53	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
54	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
55	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
56	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
57	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65

Soil Profile Display
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.84 Hz
Natural Period of Profile: 1.18 sec

Soil Profile
Material Properties
Add Layer(s)
Remove Layer

Water Table Location
Top of Layer: 4
No Water Table

Spreadsheet Legend
Below Water Table
Layer Properties
Material Properties

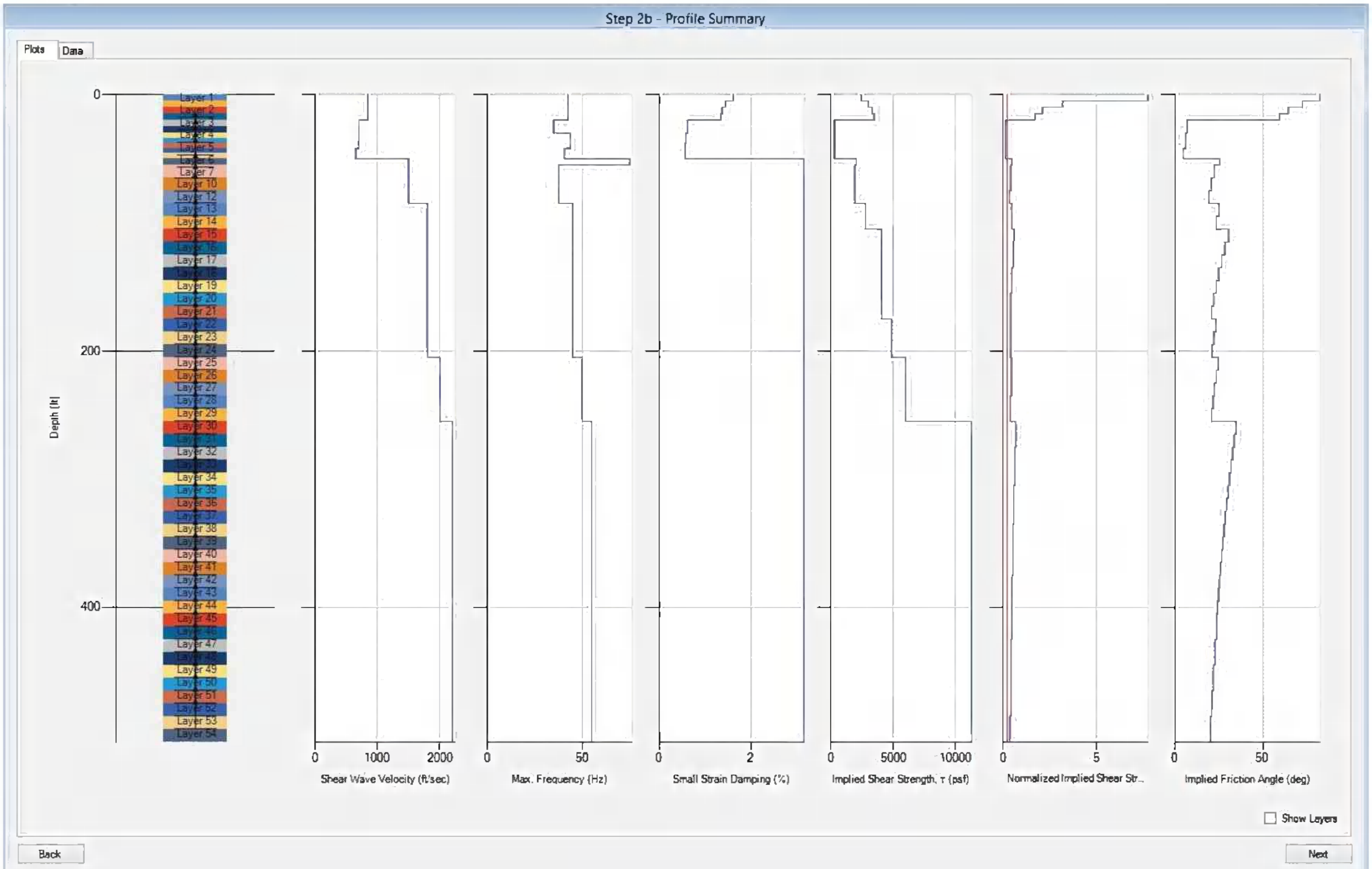
Conversion Tools
Units: English to Metric
Shear: Velocity to Modulus

Back

Next

Written by: J. McNash / C. Carlson Date: 10/12/2016 Reviewed by: W. Shin/G. Rix Date: 10/12/2016

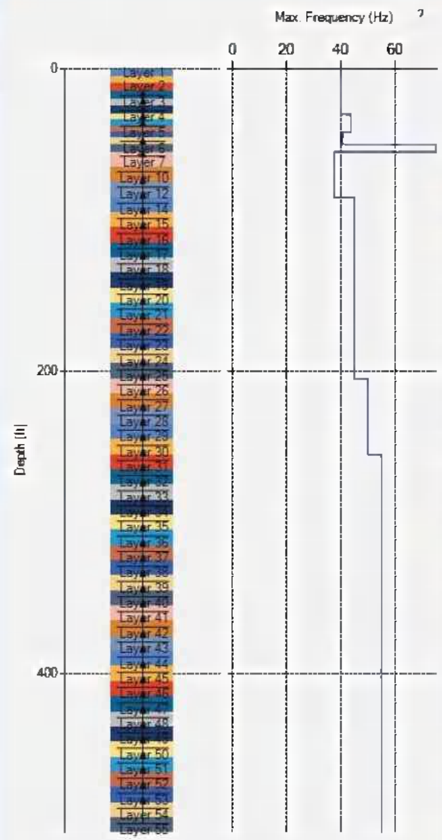
Client: Santee Cooper Project: Winyah Generating Station Project/ Proposal No.: GSC5242 Task No: 01BT



Written by: J. McNash / C. Carlson Date: 10/12/2016 Reviewed by: W. Shin/G. Rix Date: 10/12/2016

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

Step 2a - Soil Profile Definition



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
1	Dike	5	125	800	1.69261114263076	0.1892	0.18	1.53	0.975	0	0	0.626	0.24	3.25
2	Dike	5	125	800	1.5972730421619	0.2126	0.18	1.53	0.975	0	0	0.61	0.218	3.25
3	Dike	5	125	800	1.554653525905	0.2244	0.18	1.53	0.975	0	0	0.61	0.218	3.25
[illegible]	Dike	5	125	800	1.53319584210204	0.233	0.18	1.545	0.975	0	0	0.61	0.218	3.25
[illegible]	Dike	5	125	800	1.52160923352892	0.2366	0.18	1.545	0.975	0	0	0.61	0.218	3.25
[illegible]	Dike	5	125	800	1.51154821449006	0.2374	0.18	1.53	0.975	0	0	0.61	0.218	3.25
[illegible]	Foundation Soils	4	125	700	0.588509193544698	0.0248	0.18	1.44	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	700	0.577935689976007	0.0268	0.18	1.5	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	700	0.568310874060151	0.028	0.18	1.515	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	650	0.559545239776579	0.0292	0.18	1.53	1.005	0	0	0.618	0.26	3.25
[illegible]	Foundation Soils	4	125	650	0.551452632040576	0.0292	0.18	1.485	1.005	0	0	0.618	0.26	3.25
[illegible]	Chicora	5	130	1500	3.166977270163	0.0316	0.18	1.635	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1500	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	105	1800	3.16305340131604	0.032	0.18	1.41	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.15856006382266	0.0402	0.18	1.425	0.975	0	0	0.816	0.31	0.65
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	1800	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
[illegible]	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85

Soil Profile Display

Total Profile Depth (ft): 505.00

Natural Freq. of Profile: 0.85 Hz

Natural Period of Profile: 1.18 sec

Soil Profile

Material Properties

Add Layer(s)

Remove Layer

Water Table Location

Top of Layer: 4

No Water Table

Spreadsheet Legend

Below Water Table

Layer Properties

Material Properties

Conversion Tools

Units: English to Metric

Shear: Velocity to Modulus

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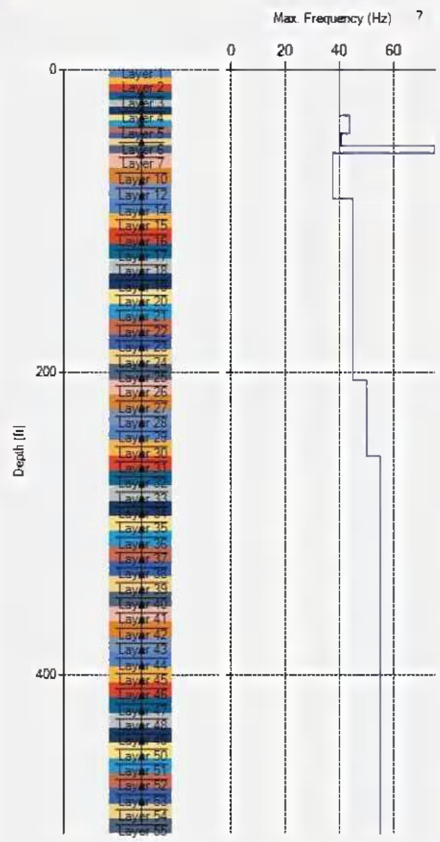
Written by: **J. McNash / C. Carlson** Date: **10/12/2016** Reviewed by: **W. Shin/G. Rix** Date: **10/12/2016**

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

Step 2a - Soil Profile Definition



Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref. Strain (%)	Ref. Stress (MPa)	Beta	s	b	d	P1	P2	P3
28	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
29	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
30	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
31	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
32	Williamsburg Formation	10	125	2000	3.16107320786548	0.0556	0.18	1.515	0.99	0	0	0.768	0.28	0.85
33	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
34	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
35	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
36	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
37	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
38	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
39	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
40	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
41	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
42	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
43	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
44	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
45	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
46	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
47	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
48	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
49	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
50	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
51	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
52	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
53	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
54	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
55	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
56	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65
57	Williamsburg Formation	10	125	2200	3.15051070328587	0.0812000000000001	0.18	1.515	0.975	0	0	0.81	0.308	0.65

Soil Profile Display
Total Profile Depth (ft): 505.00
Natural Freq. of Profile: 0.85 Hz
Natural Period of Profile: 1.18 sec

Soil Profile
Material Properties
Add Layer(s)
Remove Layer

Water Table Location
Top of Layer: 4
No Water Table

Spreadsheet Legend
Below Water Table
Layer Properties
Material Properties

Conversion Tools
Units: English to Metric
Shear: Velocity to Modulus

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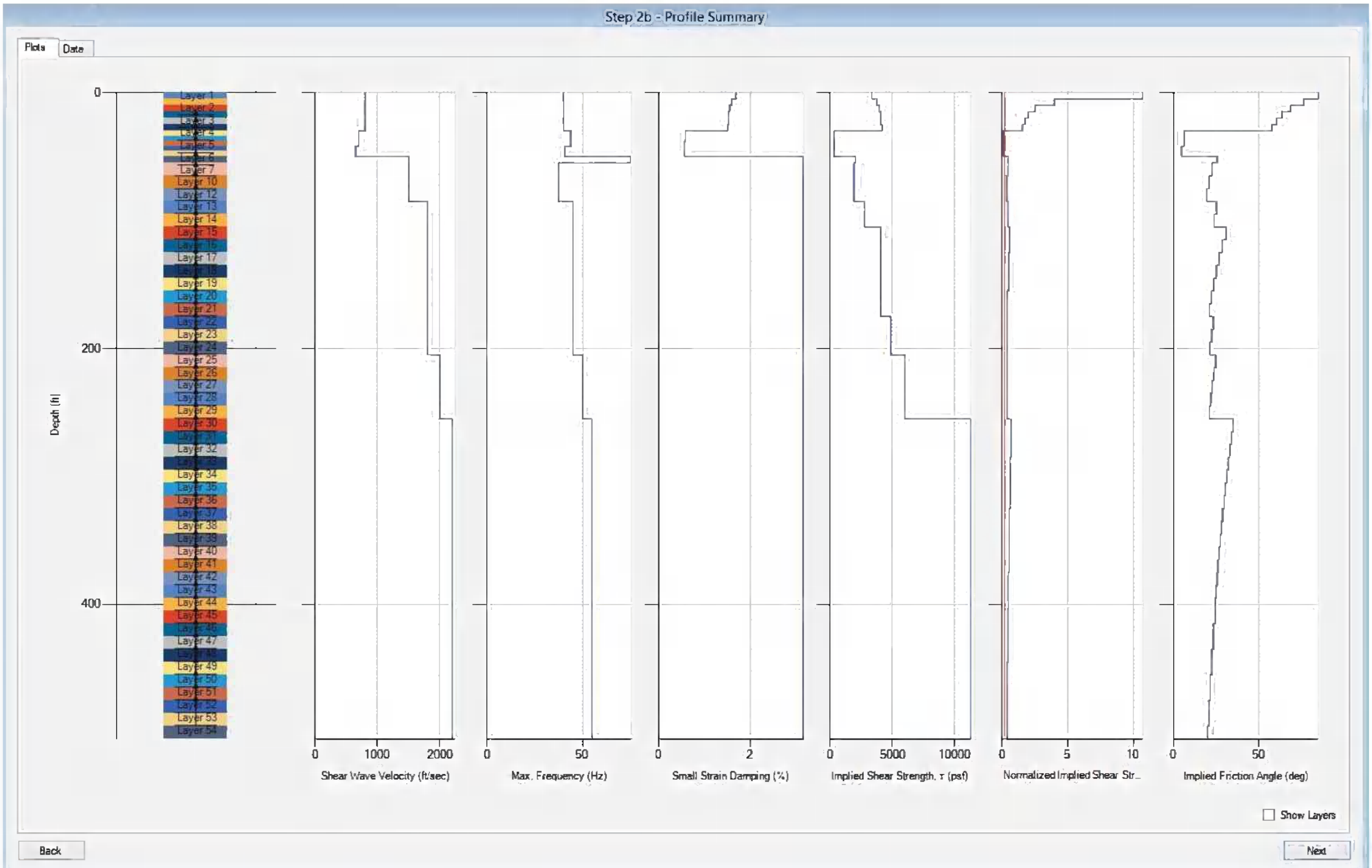
Written by:	J. McNash / C. Carlson	Date:	10/12/2016	Reviewed by:	W. Shin/G. Rix	Date:	10/12/2016
Client:	Santee Cooper	Project:	Winyah Generating Station	Project/ Proposal No.:	GSC5242	Task No:	01BT



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: SLURRY POND**

Calculation Prepared by:	Signature	[signature]	10/11/2016
	Name	James McNash, P.E.	Date
Assumptions & Procedures Checked by: (peer reviewer)	Signature	[signatures]	10/11/2016
	Name	Ming Zhu, Ph.D., P.E. Glenn Rix, Ph.D., P.E.	Date
Computations Checked by:	Signature	[signature]	10/11/2016
	Name	Josh Colley, EI	Date
Computations Back-checked by:	Signature	[signature]	10/11/2016
	Name	James McNash, P.E.	Date
Approved by: (pm or designate)	Signature	[signature]	10/11/2016
	Name	Fabian Benavente, P.E.	Date

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	J. McNash	Date:	10/11/2016	Reviewed by:	M. Zhu/G. Rix/J. Colley	Date:	10/11/2016

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

LIQUEFACTION POTENTIAL ANALYSIS: SLURRY 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 Slurry Pond at Winyah Generating Station (WGS or Site). This calculation package is Attachment 7 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report* (Safety Factor Assessment Report) prepared by Geosyntec Consultants (Geosyntec) to demonstrate compliance with the United States Environmental Protection Agency's (USEPA) Coal Combustion Residuals (CCR) Rule with respect to the periodic safety factor assessment criteria presented in 40 Code of Federal Regulations (CFR) 257.73(e). Ground motions and resulting cyclic shear stresses for the analyzed earthquake are presented in Attachment 6 titled "Seismic Hazard Evaluation and Site Response Analysis: Slurry Pond" (Site Response Package) to the Safety Factor Assessment Report. The liquefaction potential was evaluated for soil borings and cone penetration test (CPT) soundings advanced through the Slurry Pond perimeter dike based on geotechnical information collected during Geosyntec's 2013 geotechnical subsurface investigations and a historical investigation performed in 1999 (PCRA, 1999). 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: Slurry 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. Therefore, if soils at a particular site are deemed not to be susceptible to liquefaction based on the state-of-practice or regulatory guidance, additional analyses such as post-liquefaction slope stability or lateral spreading are not necessary for the anticipated seismic ground motions.

It was assumed that soils classified as Organic Peat, Silt, and Clay, or a combination of these materials, are typically not liquefiable. Additionally, soils that exhibit "clay-like" behavior according to data collected during CPT soundings were also screened as not potentially liquefiable. "Clay-like" behavior was defined as a soil with a Soil Behavior Index (I_c) greater than 2.60. The interpretation of CPT soundings and the computation of I_c are discussed in the Data Package and reiterated below. If a zone of soil that was considered to be non-liquefiable by the above criteria, the soil zone was assigned a factor of safety (FS) against liquefaction triggering of 2.00. The criteria recommended by Bray and Sancio (2006) were applied to evaluate the susceptibility of fine-grained soils to cyclic softening. Nearly all of the tested samples were found to be "Not Susceptible" to cyclic softening by these criteria.

The liquefaction analysis described below was performed based on the simplified procedure recommended by Seed and Idriss (1971) and later updated by Idriss and Boulanger (2008). Analyses were performed on both the CPT soundings and 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 effective overburden stress. The CSR for a soil interval is calculated as follows:

$$CSR_{M,\sigma'_{vo}} = 0.65\,\frac{\tau_{max}}{\sigma'_{vo}} \tag{1}$$

where:

$CSR_{M,\sigma'vo}$	=	Cyclic Stress Ratio due to an earthquake with magnitude, M;
τ_{max}	=	maximum shear stress developed during an earthquake (psf); and
σ'_{vo}	=	effective vertical stress (psf).

The cyclic shear stress represents the loading or demand on a soil unit during an earthquake.

Corrected Normalized CPT Sounding Interpretation

To evaluate the resistance or capacity of the soil against liquefaction, soil data must be interpreted from each boring or sounding. A discussion of the interpretation of the CPT data is provided in the Data Package, and the equations used in the interpretation are reiterated below.

The normalized cone tip resistance, Q, and friction ratio, F, were calculated by:

$$Q = \left(\frac{q_c-\sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \tag{2}$$

and,

$$F = \left(\frac{f_s}{q_c-\sigma_{vo}}\right)\times 100\% \tag{3}$$

where:

q_c	=	measured tip resistance (tsf);
σ_{vo}	=	total vertical stress (tsf);
σ'_{vo}	=	effective vertical stress (tsf);
P_a	=	atmospheric pressure (P_a = 1.058 tsf);
n	=	varies from 0.5 in sands to 1.0 in clays; and
f_s	=	measured sleeve friction (tsf).

It is noted that the tip resistance (q_c) measured in the field must be adjusted for pore pressure effects on the cone tip if the data collection software does not automatically account for the area ratio of the cone. This correction is discussed within the Data Package.

The soil behavior type index, I_c, as derived by Robertson and Wride (1998) is calculated by:

$$I_c = \left[(3.47 - \log(Q))^2 + (\log(F) + 1.22)^2\right]^{0.5} \quad (4)$$

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:

$$q_{c1} = C_N q_c \quad (5)$$

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.

<u>Corrected Normalized SPT Blow Count</u>

Interpretation of soil test borings and SPT blow counts is discussed within the Data Package, but is briefly reiterated below. The corrected normalized SPT blow count, $(N_1)_{60}$, which is applied in computing resistance of a soil against liquefaction, was calculated by the following equation presented by Idriss and Boulanger (2008).

$$(N_1)_{60} = N_{meas} C_E C_B C_S C_R C_N \quad (6)$$

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:

$$C_E = \frac{ER}{60} \quad (7)$$

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:

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{\left(0.784-0.0768\sqrt{(N_1)_{60}}\right)} \quad (8)$$

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:

$$CRR_{M=7.5,\sigma'_{vo}=1\,atm} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right) \quad (9)$$

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:

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \times \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \quad (10)$$

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:

$$CRR_{M=7.5,\,\sigma'_{vo}=1\,atm} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad (11)$$

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:

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \quad (12)$$

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:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right) \leq 1.1 \quad (13)$$

where:

$$C_\sigma = 1 / (37.3 - 8.27(q_{c1N})^{0.264}) \quad \leq 0.3 \text{ for CPT soundings.} \qquad (14)$$

and,

$$C_\sigma = 1 / (18.9 - 2.55((N_1)_{60})^{0.5}) \quad \leq 0.3 \text{ for SPT borings.} \qquad (15)$$

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:

$$CRR_{\sigma'_{vo}} = K_\sigma \times CRR_{\sigma'_{vo}=1\text{ atm}} \qquad (16)$$

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):

$$MSF = 6.9 \times \exp\left(\frac{-M}{4}\right) - 0.058, \text{ and MSF} \leq 1.8 \qquad (17)$$

The MSF was calculated as 1.05 for a magnitude 7.3 earthquake, which was selected based on the historical seismicity and 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:

$$CRR_M = MSF \times CRR_{M=7.5} \qquad (18)$$

Age Correction Factor (K_{DR})

Correlations associated with liquefaction potential analysis were developed based on case histories of the presence or absence of liquefaction in relatively young soil deposits (i.e., Holocene age). As described in the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (2010), the CRR may be adjusted to account for diagenesis and other age-related effects in older soils that have not previously experienced liquefaction. Equation 13-47 within Chapter 13 of the SCDOT Geotechnical Design Manual computes the Age Correction Factor (K_{DR}) based on its age (t in years) as:

$$K_{DR} = 0.17\log_{10}(t) + 0.83 \qquad (19a)$$

Meanwhile, Andrus et al. (2008) presents a similar equation for the K_{DR} as:

$$K_{DR} = 0.19\log_{10}(t) + 0.68 \quad (19b)$$

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 account for this aging effect, and is computed as follows.

$$CRR_{M,K} = K_{DR} \times CRR_{M} \quad (20)$$

Factor of Safety

Finally, the factor of safety against liquefaction (FS_{liq}) triggering for both SPT and CPT analyses was computed by:

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'vo,Kdr}}{CSR_{M,\sigma'vo}} \quad (21)$$

where:

$CRR_{M,\sigma'vo,Kdr}$ = cyclic resistance ratio adjusted for earthquake magnitude, overburden, and aging;

and

$CSR_{M,\sigma'vo}$ = cyclic stress ratio for the same earthquake and overburden stress.

ANALYSIS CASES

As noted previously, liquefaction potential computations were conducted on soil data collected in soil borings and CPT soundings overseen by Geosyntec in 2013 and on historical borings performed by PCRA in 1999. Computations were limited to soil borings and soundings located through the dike centerline. As noted within the Data Package (Attachment 5), S&ME conducted a geotechnical site investigation in 1978 prior to the construction of the perimeter dikes. Within this calculation package, soil borings advanced by S&ME (denoted by the Boring ID scheme "SC-XX") were not evaluated and will be evaluated as a part of the "Unstable Areas" evaluation for the Slurry Pond as these borings were not advanced through the perimeter dike soils. Paul C. Rizzo and Associates (PCRA) performed an investigation in 1999 to evaluate phreatic conditions through the Slurry Pond perimeter dikes. PCRA advanced some borings without sampling and without SPTs to the top of the Chicora stratum. PCRA (1999) boring logs where SPTs were collected were utilized as a part of this liquefaction potential analysis package. A boring location map can be found within Attachment 5 of this Safety Factor Assessment Report.

Three representative soil profiles of shear wave velocity (V_s) were developed and presented in the Data Package from the dike fill soils to the Chicora stratum. These profiles were developed from direct measurements of V_s and by means of a correlation with CPT sounding data. As discussed in the Data Package, these representative V_s profiles were supplemented with historical data to extend the V_s profile into the underlying Chicora and Williamsburg Formation Clay strata during the site response analysis.

For each representative soil profile, a site response analysis, described within the Site Response Package, 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 boring or sounding. The maximum shear stress at each computed depth for each representative profile was tabulated and provided in Table 2. The τ_{max} for measurements in between depth intervals listed within Table 2 were linearly interpolated to calculate τ_{max} at every depth interval.

INPUT PARAMETERS

The following section describes the selection of the input parameters applied for the liquefaction potential analysis.

Calibration of Historical Borings

Liquefaction potential of dike fill soils was evaluated based on data provided by historical boring logs, in addition to borings and soundings overseen by Geosyntec (2013). As stated previously, correlations developed to predict the resistance of soils to liquefaction are based on empirical observations using a standard procedure or method during drilling activities. The historical geotechnical reports did not explicitly provide many details of the methodology during the geotechnical investigation. However, Geosyntec intentionally located a soil test boring and CPT sounding directly adjacent to a historical PCRA boring during the 2013 investigation to "calibrate" the historical boring to Geosyntec's soil borings, which were conducted in accordance with current industry standards. Based on a comparison of data, Geosyntec assumed for the purposes of this analyses that historical PCRA borings were advanced using mud rotary wash methods with bit that deflects the drilling fluid and creates a borehole approximately four inches in diameter. Furthermore, calibration of historical borings demonstrated that an Energy Ratio (ER) between 60 percent and 70 percent for the hammer system during SPT testing corresponds well with Geosyntec data at the same location, as shown in Figure 1. Thus, an ER of 70 percent was selected during this analysis of PCRA borings (denoted by "B-2XX") for the Slurry Pond. The subcontractor during Geosyntec's 2013 investigation, Soil Consultants, Inc. (SCI), reported that the automatic hammer on the utilized drilling rig had an ER of 89 percent, which was independently evaluated within six months of the investigation. Carolina Drilling, Inc. advanced GSB-11A, and who utilized a drilling rig which was found to have an ER of 79.3 percent (Attachment 5).

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 purposes, CPT intervals were assigned a unit weight based on the ranges presented for soils in the region provided within the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (GDM) (SCDOT, 2010) and based on the initial soil behavior index (I_c) as follows:

- Clays and clayey sand mixtures ($I_c > 2.95$): 100 pcf;
- Silt to silty sand mixtures ($2.60 < I_c \leq 2.95$): 100 pcf;
- Silty sands to sand mixtures ($2.05 < I_c \leq 2.60$): 110 pcf;
- Sands ($1.31 < I_c \leq 2.05$): 120 pcf; and
- 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 Slurry 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 2, soils immediately surrounding Slurry 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 on the surface elevation of historical boring conducted prior to dike construction by S&ME (1978). Foundation soils were assumed to begin at the surface elevation of the nearest historical boring adjacent to the investigation point. Boring information and the top of foundation soil elevation (or approximate dike base elevation) are summarized within Table 5 of this Liquefaction Package. An age correction factor of 1.0 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 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 Data Package, fines content data of dike fill and foundation materials is variable across the Site. Physical samples are not collected during CPT soundings, and many historical borings did not extensively test soil index properties. As it is considered impractical to collect index testing (and fines content data) on every soil sample or interval, representative fines content profiles were developed based on index testing data presented within the Data Package and applied to CPTs when a soil boring was not located in the vicinity of a CPT sounding. Index testing for soil borings, when available, were utilized for each individual SPT N-value. The representative fines content profiles are shown in Figures 3a through 3c and summarized in Table 6. The extent of each profile is delineated in Figure 4; while, the assignment of each fines content profile is summarized within Table 5.

Phreatic Surface

The phreatic surface through the perimeter dike centerline at the time of this Liquefaction Package was developed predominantly based on water levels collected from temporary piezometers installed through the Slurry Pond perimeter dikes in 2014, CPT porewater pressure signatures, and CPT porewater pressure dissipation tests. As free water was removed from the Slurry Pond (W-SW-WSP), water elevations in temporary piezometers PPZ-31 and PPZ-32 were also lowered. For the purposes of this calculation package, the phreatic surface at the time of analysis through the dike centerline was conservatively selected as 16.0 ft NGVD29 for borings and soundings located on the western perimeter dikes (i.e., west of CPT-17).

For borings and soundings positioned on the eastern perimeter dikes (i.e., CPT-17 to the divider dike), the phreatic surface at the time of boring and analysis was selected based on soil boring depth to water measurements or CPT porewater pressure signatures. The free water storage capacity of the Slurry Pond is provided in the north to northwest corner of the impoundment, and the phreatic surface within the adjacent perimeter dikes responds relatively quickly to fluctuations in the free water level. Meanwhile, FGD residuals have accumulated in the south and east portions of the Slurry Pond. Temporary piezometers PPZ-33 and PPZ-34, located within the interior of the Slurry Pond, indicate that an elevated phreatic surface exists within the residual material that is slowly lowering as the free water is maintained at an operating pool of 19.6 ft NGVD29 or below. Since the interstitial water has not drained from the mass of FGD residuals, the water levels in the eastern perimeter dikes currently (i.e., time of analysis) were assumed to be equivalent to the phreatic surface at the time of each boring or sounding, which were conducted when the Slurry Pond was operated with a free water elevation of approximately 34.25 ft NGVD29.

Phreatic surface assumptions through the Slurry Pond perimeter dikes at the time of the boring (TOB) and at the time of analysis (TOA) for this calculation package are summarized in Table 5. For soil borings, the depth to water measurements from the nearest hollow stem auger boring was selected for the water elevation at the TOB.

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 (including the historical borings) 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 boring elevations are shown in Figures 5 through 15. Figure 5 shows CPT-1, CPT-3, and B-206, which are located in the southeast corner of the Slurry Pond immediately north of the divider dike. Subsequent figures depict calculation results for soil borings and CPT soundings positioned progressively in a counter-clockwise direction around the surface impoundment. Example calculations are provided within Appendix 1.

The liquefaction potential calculation results can be generally summarized as follows:

- The computed FS_{liq} in the southeast corner of the Slurry Pond ranged between 1.2 and 1.7 (when not greater than 2.0) between elevations 10.0 ft and 20.0 ft NGVD29 and 0.0 to -5.0 ft NGVD29.

- The computed FS_{liq} from the southeastern corner to the north end of the eastern dikes of the Slurry Pond were found to range between 1.05 and 1.60 (when not greater than 2.0) at elevations 0.0 ft and 10.0 ft NGVD29. It is noted that the lower values within this range were computed in the southeastern corner of the Slurry Pond.

- The computed FS_{liq} in the northwestern perimeter dikes were found to range between 1.3 and 1.5 (when not greater than 2.0) at elevations 2.0 to 5.0 ft NGVD29 and -2.0 to -5.0.

It is noted that other zones within the foundation soils were computed with FS_{liq} between 1.2 and 2.0 at elevations other than those listed above. No zones of liquefiable soils (FS_{liq} below 1.0) were indicated by the evaluation results.

CONCLUSIONS

Based the liquefaction potential computations presented within this calculation package, liquefiable soils were not observed in the dike fill soils (i.e., native soils recompacted to form impounding perimeter dikes) or foundation soils beneath the perimeter dikes of the Slurry Pond. Soil borings and CPT soundings advanced at the downstream toe of the Slurry Pond perimeter dikes were not evaluated within this calculation package and will be included during an evaluation of "Unstable Areas" for the Slurry Pond. Since liquefiable zones were not computed for borings and CPT soundings advanced through the perimeter dikes within the dike fill or foundation soils beneath the perimeter dikes, additional post-liquefaction stability and displacement analyses are not warranted for the Slurry Pond dikes at this time.

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TABLES

Table 1. Correction Factors for Interpretation of SPT for Liquefaction Potential Analysis (Idriss and Boulanger, 2008)

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

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Table 2. Summary of Representative Profiles for the Dike Centerline

Profile 1		Profile 2		Profile 3	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	38.9	2.5	37.1	2.5	45.4
7.5	88.7	7.5	88.6	7.5	90.0
12.5	143.8	12.5	131.2	12.5	127.1
17.5	197.1	17.5	178.0	17.5	169.0
22.5	250.7	22.5	226.9	22.5	206.1
27.5	298.8	27.5	263.3	27.5	242.6
32	327.1	32	287.8	32	279.9
36	343.6	36	300.9	36	298.5
40	354.0	40	307.2	40	307.4
44	352.7	44	304.0	44	304.1
48	363.4	48	313.4	48	313.4
52.5	442.2	52.5	375.2	52.5	385.9
60	547.9	60	492.3	60	488.1

Notes:

1. Profiles were developed in the Site Response Package provided as Attachment 6.
2. For calculation points located in between the depth intervals listed above, the average τ_{max} was linearly interpolated for liquefaction potential computations.

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Table 3. Susceptibility of Soil Deposits to Liquefaction during Strong Seismic Shaking (Youd and Perkins, 1978)

Type of deposit	Distribution of cohesionless sediments in deposit	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction			
		< 500 years	Holocene	Pleistocene	Pre-Pleistocene
Continental					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plains	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebkha	Locally variable	High	Moderate	Low	Very low
Coastal zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach—high wave energy	Widespread	Moderate	Low	Very low	Very low
Beach—low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
Artificial fill					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 4. Age Correction Factor (K_{DR}) based on Soil Age

Soil Age, t (years)	K_{DR} [1]	K_{DR} [2]
126	1.19	1.08
546	1.30	1.20
5,038	1.46	1.38
10,000	1.51	1.44
450,000	1.79	1.75

Notes:

1. K_{DR} computed by SCDOT Geotechnical Design Manual (SCDOT, 2010), as provided in Equation 19a.
2. K_{DR} computed by Andrus et al (2008) as provided in Equation 19b.

Table 5. Summary of Borings and Soundings Analyzed for Liquefaction Potential

Borehole ID	Northing	Easting	Elevation	Dike Bottom Elevation	Dike Bottom Basis	GWT EL at TOB	GWT Depth at TOB	GWT EL at TOA	GWT Depth at TOA	T_{cyclic} profile	FC Basis
-	ft	ft	ft	ft	-	ft	ft	ft	ft	-	-
CPT-01	548805.925	2500016.426	39.38	25.2	SC-52	29.4	10.0	29.4	10.0	Profile 1	Profile A
CPT-03	549179.558	2500199.574	38.33	19.0	SC-42	26.3	12.0	26.3	12.0	Profile 1	Profile A
CPT-04	549351.863	2500330.724	38.62	19.0	SC-42	22.6	16.0	22.6	16.0	Profile 1	Profile A
CPT-06	549456.322	2500411.877	38.32	17.5	SC-41	22.3	16.0	22.3	16.0	Profile 1	Profile A
CPT-07	549602.193	2500529.564	38.51	17.5	SC-41	22.5	16.0	22.5	16.0	Profile 1	Profile A
CPT-10	550077.297	2500483.518	37.98	15.3	SC-28	22.0	16.0	22.0	16.0	Profile 1	Profile B
CPT-13	550273.629	2500271.050	38.22	15.3	SC-28	22.2	16.0	21.2	17.0	Profile 2	Profile B
CPT-17	550690.756	2499821.833	38.07	16.4	SC-27	16.0	22.1	22.1	16.0	Profile 2	GSB-5
CPT-19	550993.800	2499493.874	38.51	17.1	SC-26	22.5	16.0	16.0	22.5	Profile 2	Profile B
CPT-20	551089.929	2499388.484	38.57	17.5	SC-20	22.6	16.0	16.0	22.6	Profile 2	Profile C
CPT-24	551266.120	2499180.501	38.61	17.5	SC-20	16.0	22.6	16.0	22.6	Profile 2	Profile C
CPT-26	551311.434	2498800.750	38.16	15.8	SC-21	22.2	16.0	16.0	22.2	Profile 2	Profile C
CPT-28	551131.581	2498327.600	38.52	9.00	SC-22	16.0	22.5	16.0	22.5	Profile 2	Profile C
CPT-33	550968.320	2498133.106	38.98	9.00	SC-22	16.0	23.0	16.0	23.0	Profile 3	Profile C
CPT-36	550667.082	2497917.354	38.71	12.7	SC-32	12.7	26.0	16.0	22.7	Profile 3	Profile C
CPT-38	550185.539	2497835.386	37.82	10.9	SC-33	12.8	25.0	16.0	21.8	Profile 3	Profile C
CPT-42	549898.093	2497798.050	37.81	10.9	SC-33	12.8	25.0	16.0	21.8	Profile 3	Profile C
CPT-116	550265.804	2497845.353	37.95	12.7	SC-32	16.0	22.0	16.0	22.0	Profile 3	Profile C
CPT-116A	NS	NS	37.95	12.7	SC-32	16.0	22.0	16.0	22.0	Profile 3	Profile C
CPT-117	550370.532	2497861.690	37.86	12.7	SC-32	16.0	21.9	16.0	21.9	Profile 3	Profile C
GSB-03	549760.850	2500650.369	38.39	18.2	SC-40	28.4	10.0	28.4	10.0	Profile 1	GSB-03
GSB-04	550370.128	2500167.341	38.66	15.3	SC-28	28.7	10.0	28.4	10.3	Profile 1	GSB-04
GSB-05	550684.521	2499829.144	38.05	16.4	SC-27	22.1	16.0	16.0	22.1	Profile 2	GSB-05
GSB-07	551318.990	2498950.556	38.16	15.8	SC-21	22.2	16.0	16.0	22.2	Profile 2	GSB-07
GSB-08	551066.479	2498252.288	39.19	9.00	SC-22	23.2	16.0	16.0	23.2	Profile 2	GSB-08
GSB-10	550868.544	2498024.243	38.86	12.7	SC-32	22.9	16.0	16.0	22.9	Profile 3	GSB-10
GSB-11	550344.335	2497857.101	37.78	12.7	SC-32	21.8	16.0	16.0	21.8	Profile 3	GSB-11
GSB-11A	550344.335	2497857.101	37.78	12.7	SC-32	13.4	24.4	13.4	24.4	Profile 3	Profile C
B-201	2497800.325	550003.0901	38.1	10.9	SC-33	22.1	16.0	16.0	22.1	Profile 3	B-201
B-202	2499022.861	551293.4313	38.2	15.8	SC-21	22.2	16.0	16.0	22.2	Profile 2	GSB-7
B-203	2498176.332	550991.3928	39.9	9.00	SC-22	23.9	16.0	16.0	23.9	Profile 3	GSB-8
B-205	2498531.792	549368.1928	39.0	10.9	SC-33	23.0	16.0	16.0	23.0	Profile 3	B-205 / Profile C
B-206	2500292.656	549310.6617	38.9	19.0	SC-42	26.9	12.0	26.9	12.0	Profile 1	B-206 / Profile A
B-209	2499818.024	550674.9716	38.8	16.4	SC-27	22.8	16.0	16.0	22.8	Profile 2	GSB-05
B-212	2500549.491	550003.0901	38.6	18.2	SC-40	22.6	16.0	22.6	16.0	Profile 1	B-212 / Profile B
B-214	2499731.727	548665.4911	39.1	25.2	SC-52	29.1	10.0	29.1	10.0	Profile 1	B-214 / Profile A

Notes:

1. GSB-series and CPT-01 through CPT-42 were advanced by Geosyntec in spring 2013; CPT-116, CPT-116A, and CPT-117 were advanced by Geosyntec in fall 2013; and B-series borings were advanced by PCRA in 1999. SC-series borings were advanced by S&ME in 1978 prior to construction of the Slurry Pond.
2. NS - Not Surveyed; ft NGVD29 - feet National Geodetic Vertical Datum of 1929; TOB - Time of Boring; TOA - Time of Analysis; GWT - Groundwater Table; FC - Fines Content.
3. CPT-116A was offset approximately 20 ft north of CPT-116, but was not surveyed. Top of sounding elevation was assumed to be equal of CPT-116.
4. Dike base elevation was estimated based on the elevation of the nearest historical boring (prior to construction) (S&ME, 1978).
5. GSB-11A was performed in Spring 2016 and the GWT at the time of boring was measured within a temporary standpipe as 13.3 ft NGVD29. Since only one grain size distribution test was performed on dike fill materials within GSB-11A, Profile C was selected in lieu of utilizing the boring-specific fines content data.

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Table 6. Summary of Fines Content Profiles

Profile A		Profile B		Profile C	
Elevation (ft)	percent	Elevation (ft)	percent	Elevation (ft)	percent
38	35	38	40	38	40
18	35	15	40	18	40
18	20	15	22	18	10
4	20	10	22	2	10
4	25	10	10	2	16
-15	25	0	10	-15	16
-	-	0	20	-	-
-	-	-15	20	-	-

Notes:

1. Fines content profiles were developed from Geosyntec and historical fines content laboratory testing data. Results were spatially grouped together by area and plotted with respect to elevations to evaluate trends in the subsurface, because physical samples were not available directly adjacent to each CPT sounding. These profiles were applied with CPT soundings.
2. The assignment of each fines content profile is shown on Figure 4.
3. Elevations refer to ft NGVD29.

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FIGURES

Written by:	J. McNash	Date:	10/11/2016	Reviewed by:	M. Zhu/G. Rix/J. Colley	Date:	10/11/2016

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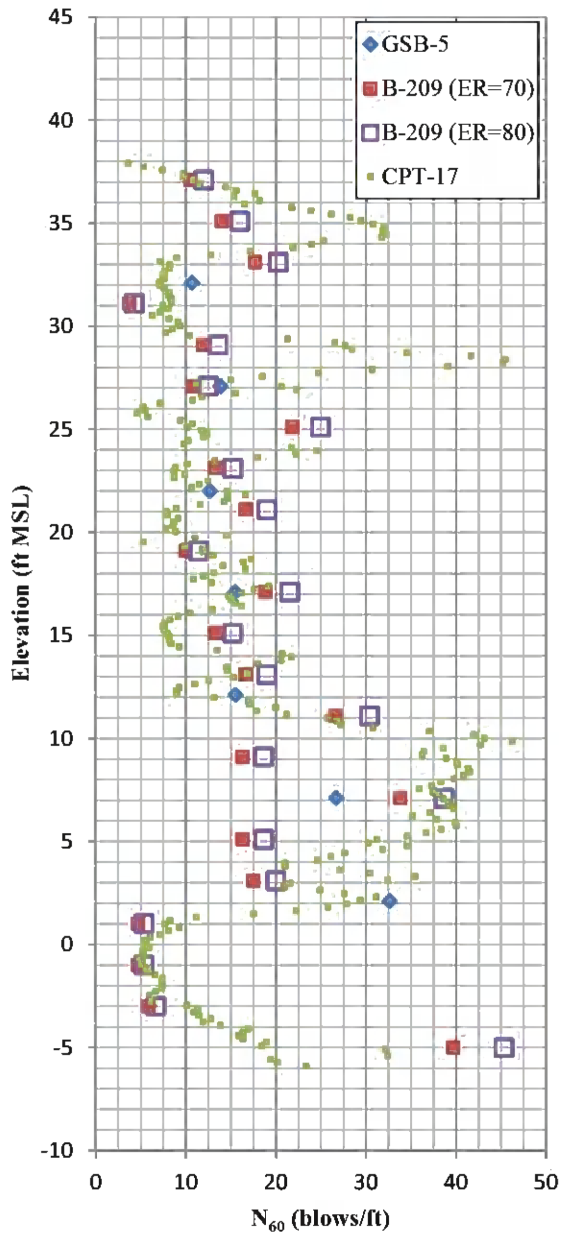

Figure 1. Geosyntec - PCRA Calibration Boring

Note:

1. A computed N_{60} – profile is provided for CPT-17, which is directly adjacent to GSB-5 and B-209.

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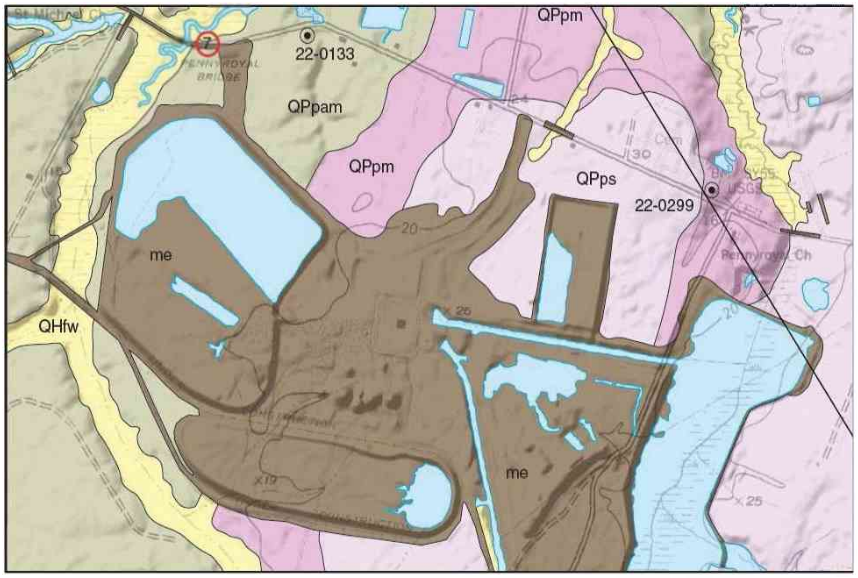

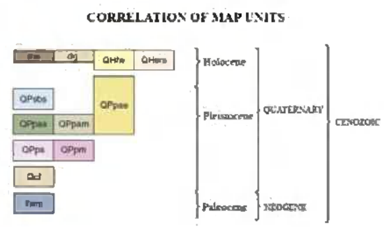

QPpam **Estuarine deposits or marine deposits or both (Pleistocene)** – In its lower part, quartz and phosphatic sand, medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse grained with trace amounts of very fine grained heavy minerals. Lower part 10 to 20 feet thick. 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 2. Geologic Map of Areas Surrounding Slurry Pond
(Map taken from SC Department of Natural Resources: Geological Survey, 2012)

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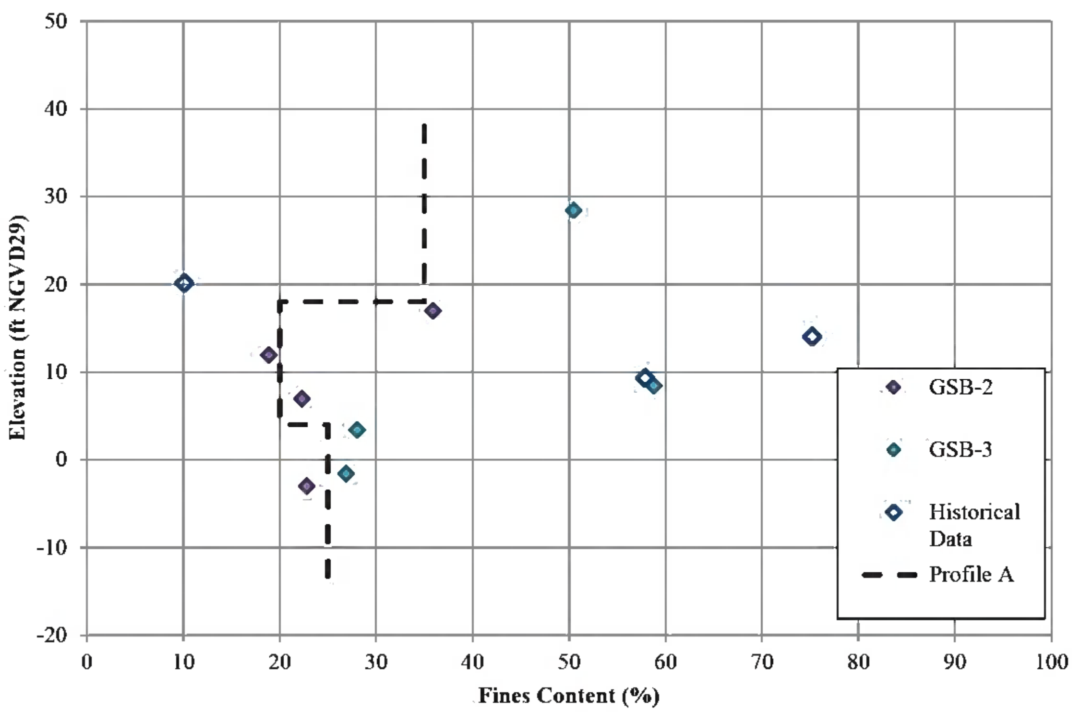

Figure 3a. Representative Fines Content Profile A

Written by: J. McNash Date: 10/11/2016 Reviewed by: M. Zhu/G. Rix/J. Colley Date: 10/11/2016

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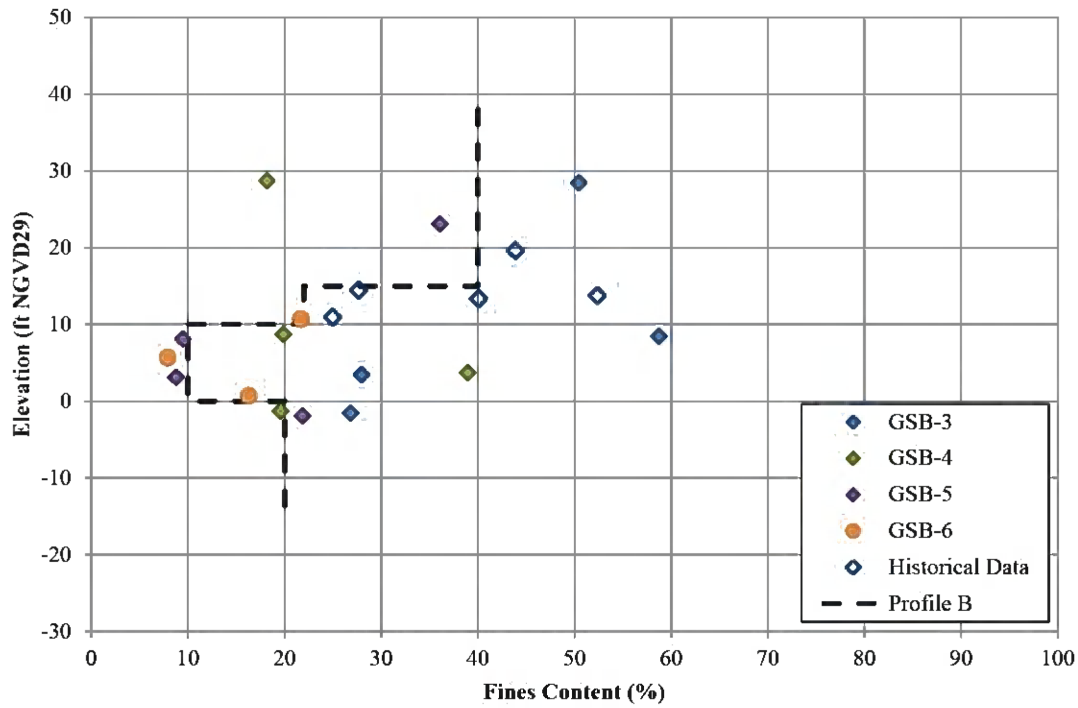

Figure 3b. Representative Fines Content Profile B

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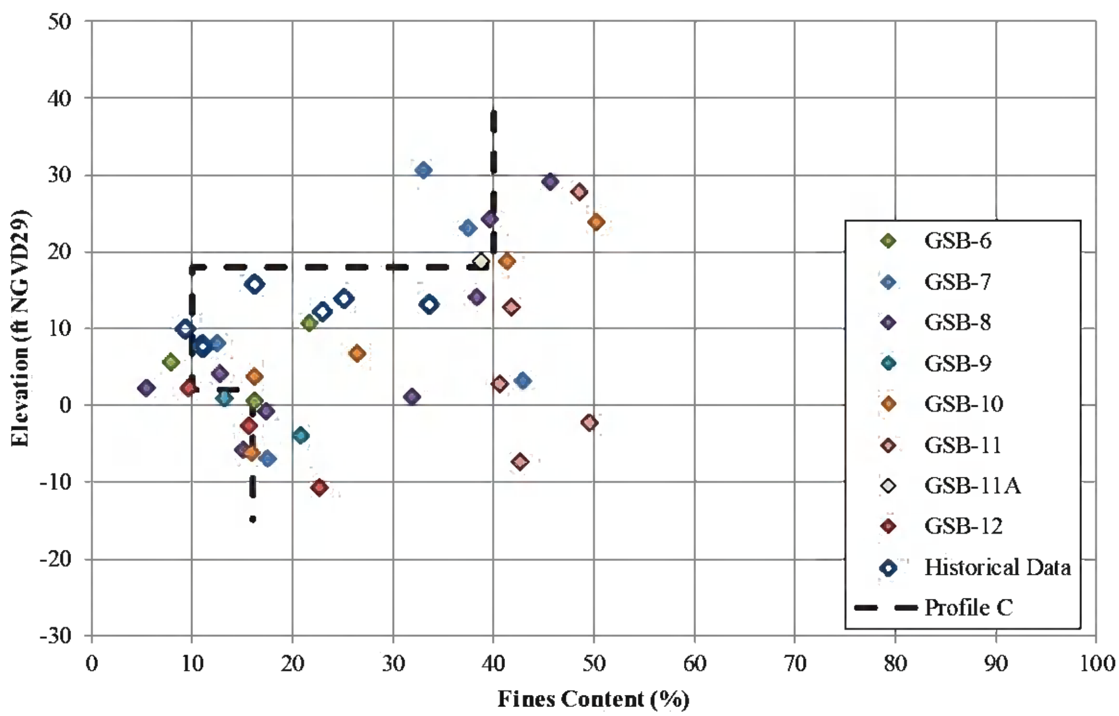

Figure 3c. Representative Fines Content Profile C

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

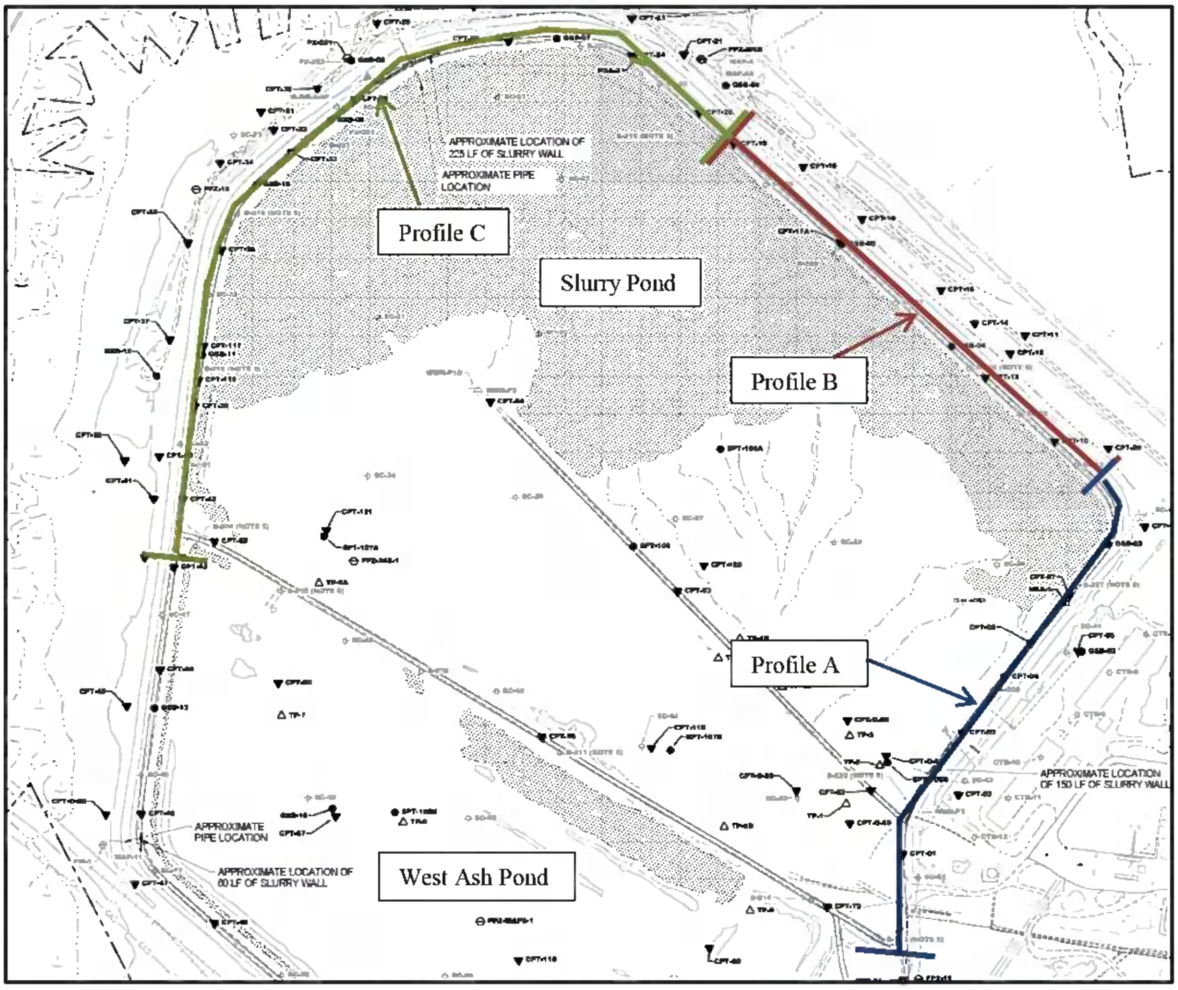

Figure 4. Location and Extent of Fines Content Profiles for Analysis

Note:

1. Representative fines content profiles were developed for the Slurry Pond.

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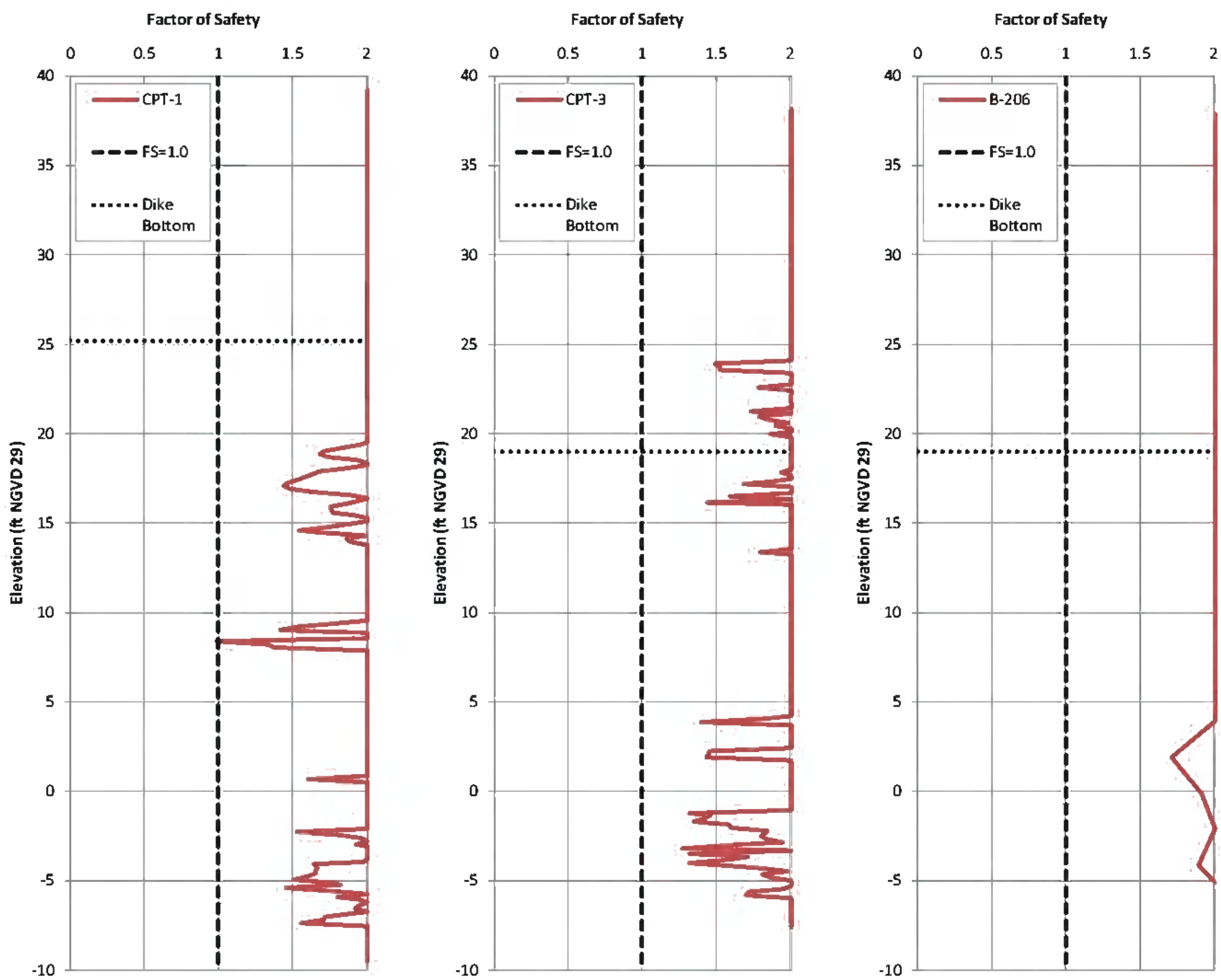

Figure 5. Liquefaction Results for Dike and Foundation Soils for CPT-1, CPT-3, and B-206

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-52 and SC-42 for CPT-1 and CPT-3/B-206, respectively, as provided in Table 5.

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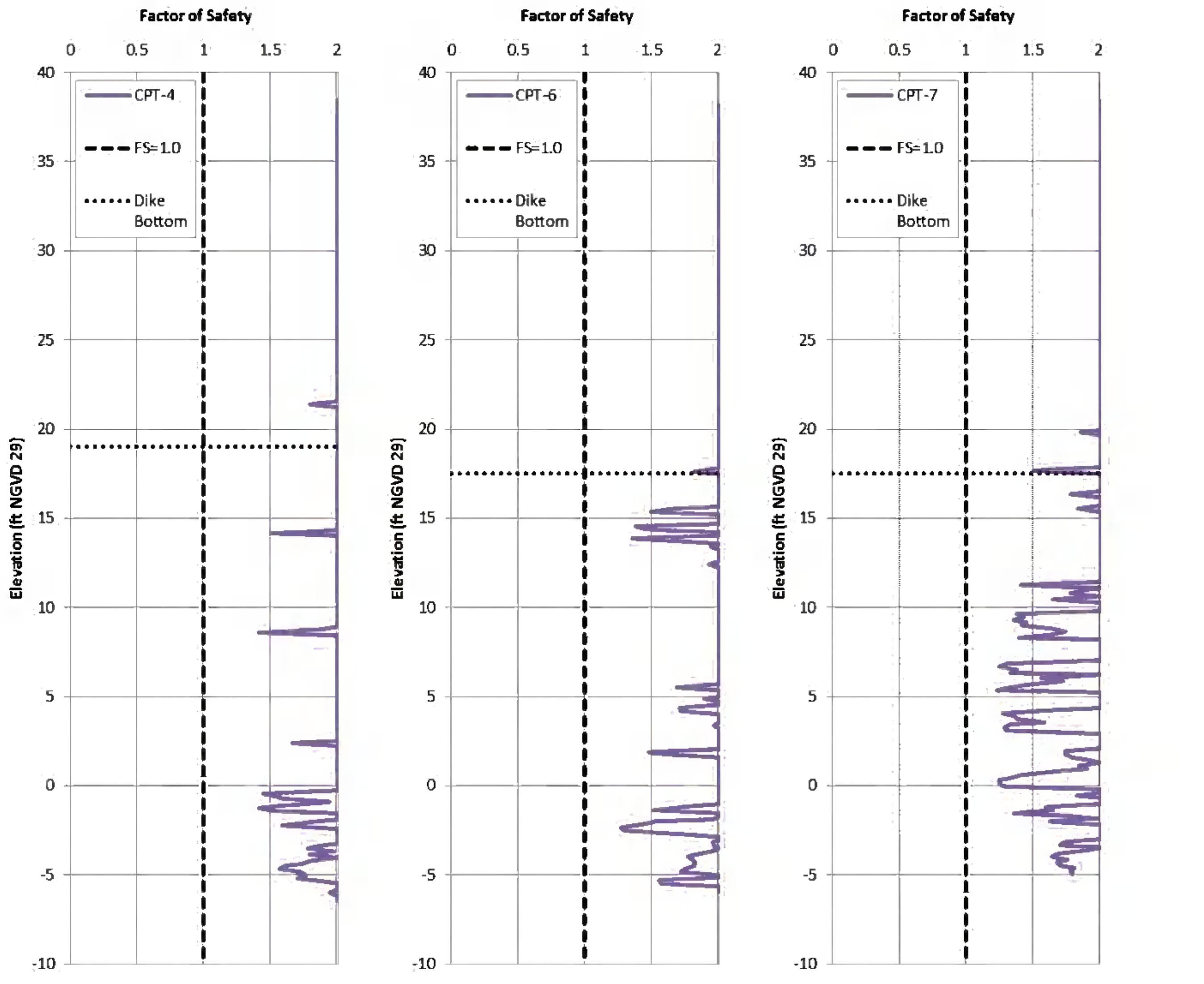

Figure 6. Liquefaction Results for Dike and Foundation Soils for CPT-4, CPT-6, and CPT-7

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-42 and SC-41 for CPT-4 and CPT-6/CPT-7, respectively, as provided in Table 5.

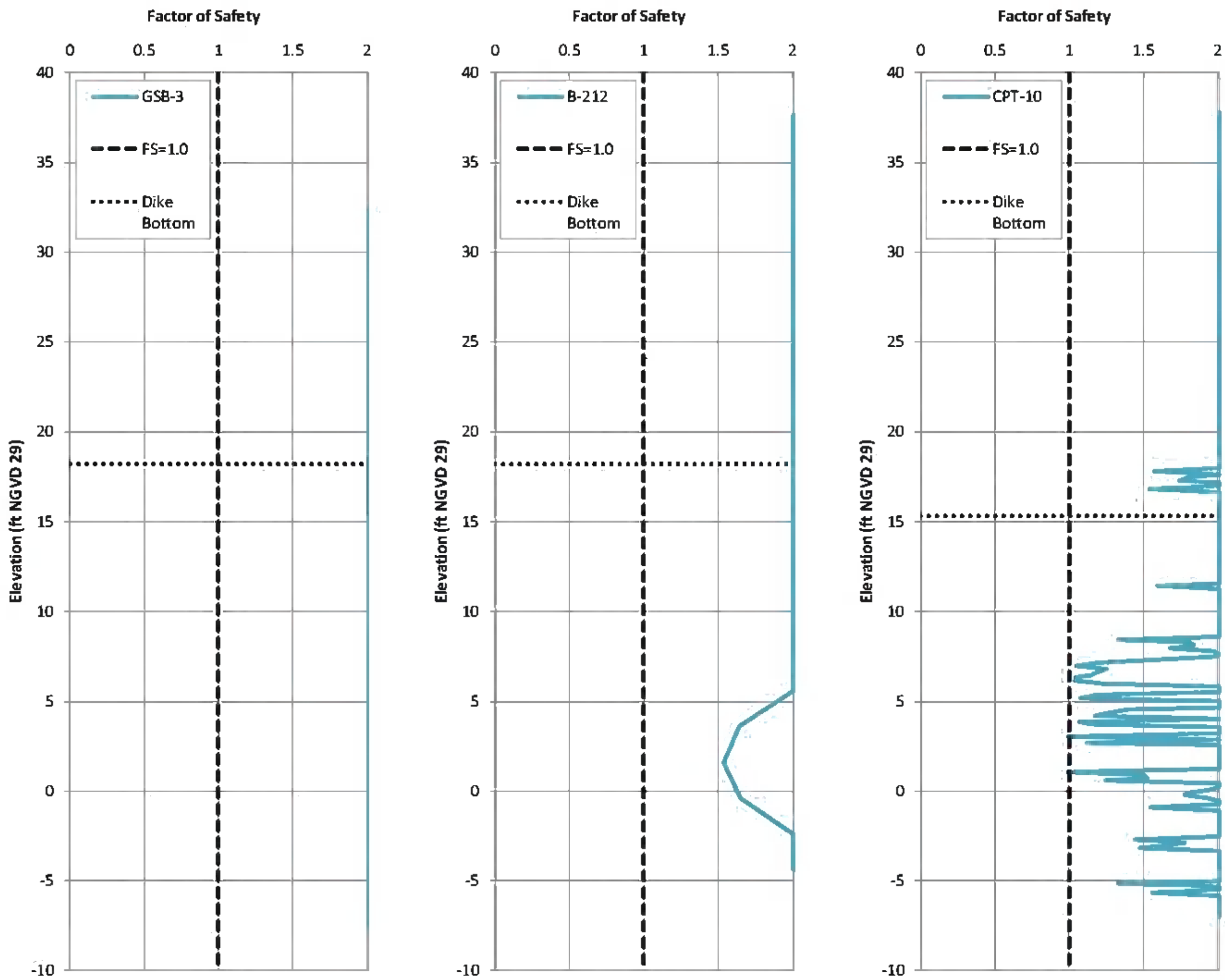

Figure 7. Liquefaction Results for Dike and Foundation Soils for GSB-3, B-212, and CPT-10

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-40 and SC-28 for GSB-3/B-212 and CPT-10, respectively, as provided in Table 5.

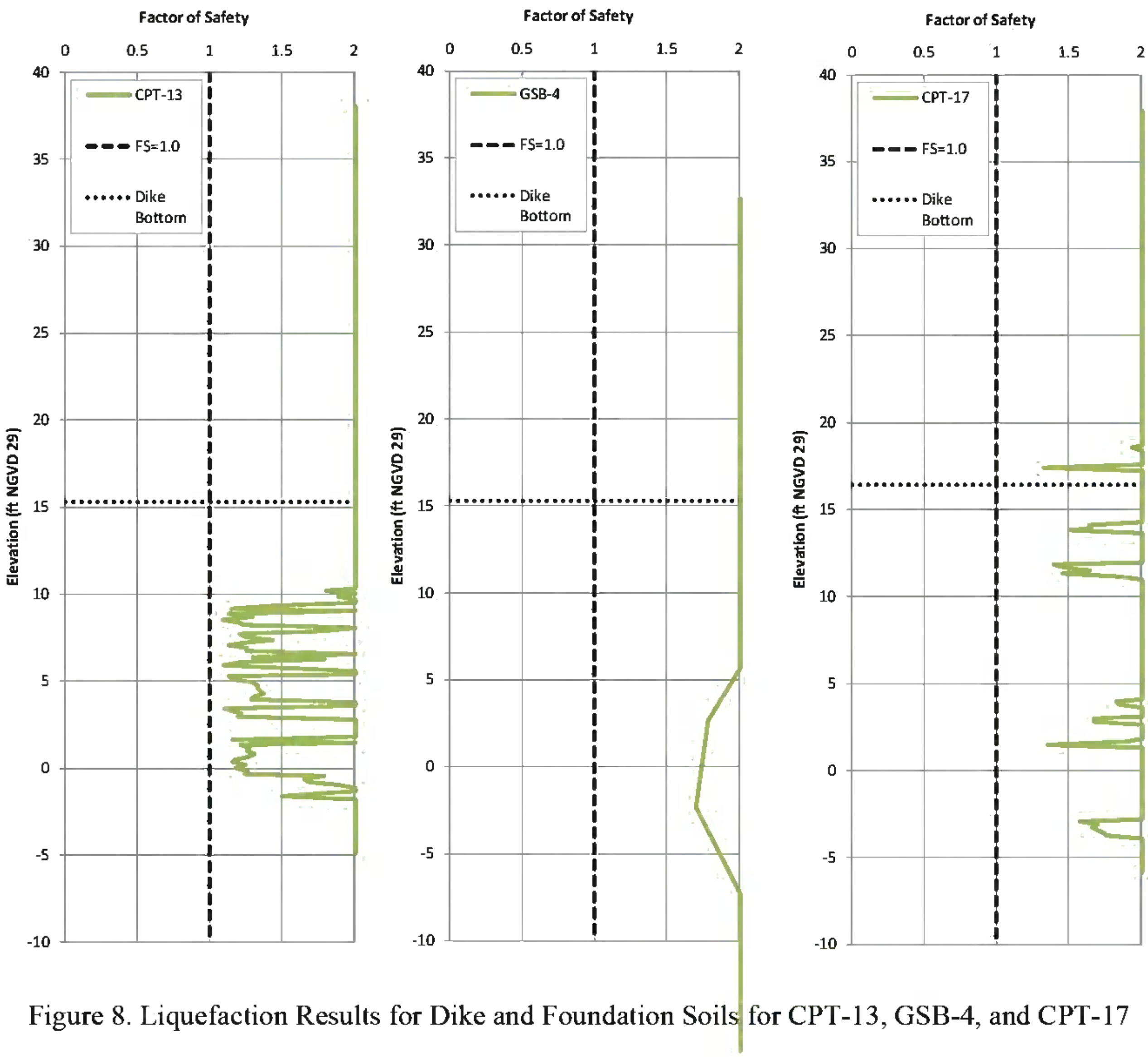

Figure 8. Liquefaction Results for Dike and Foundation Soils for CPT-13, GSB-4, and CPT-17

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-28 and SC-27 for CPT-13/GSB-4 and CPT-17, respectively, as provided in Table 5.

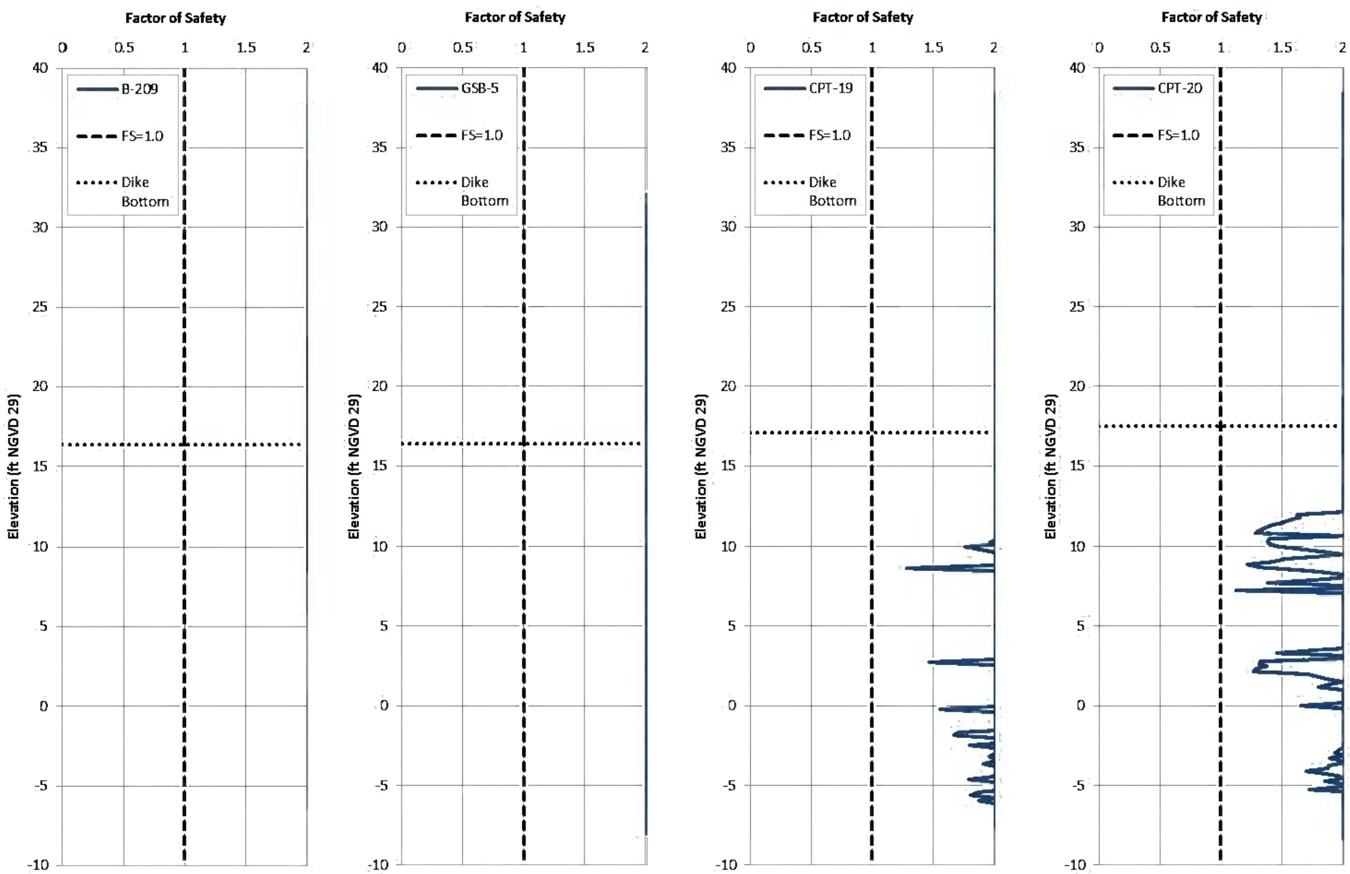

Figure 9. Liquefaction Results for Dike and Foundation Soils for B-209, GSB-5, CPT-19, and CPT-20

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-27, SC-26, and SC-20 for B-209 and GSB-5, CPT-19, and CPT-20, respectively, as provided in Table 5.

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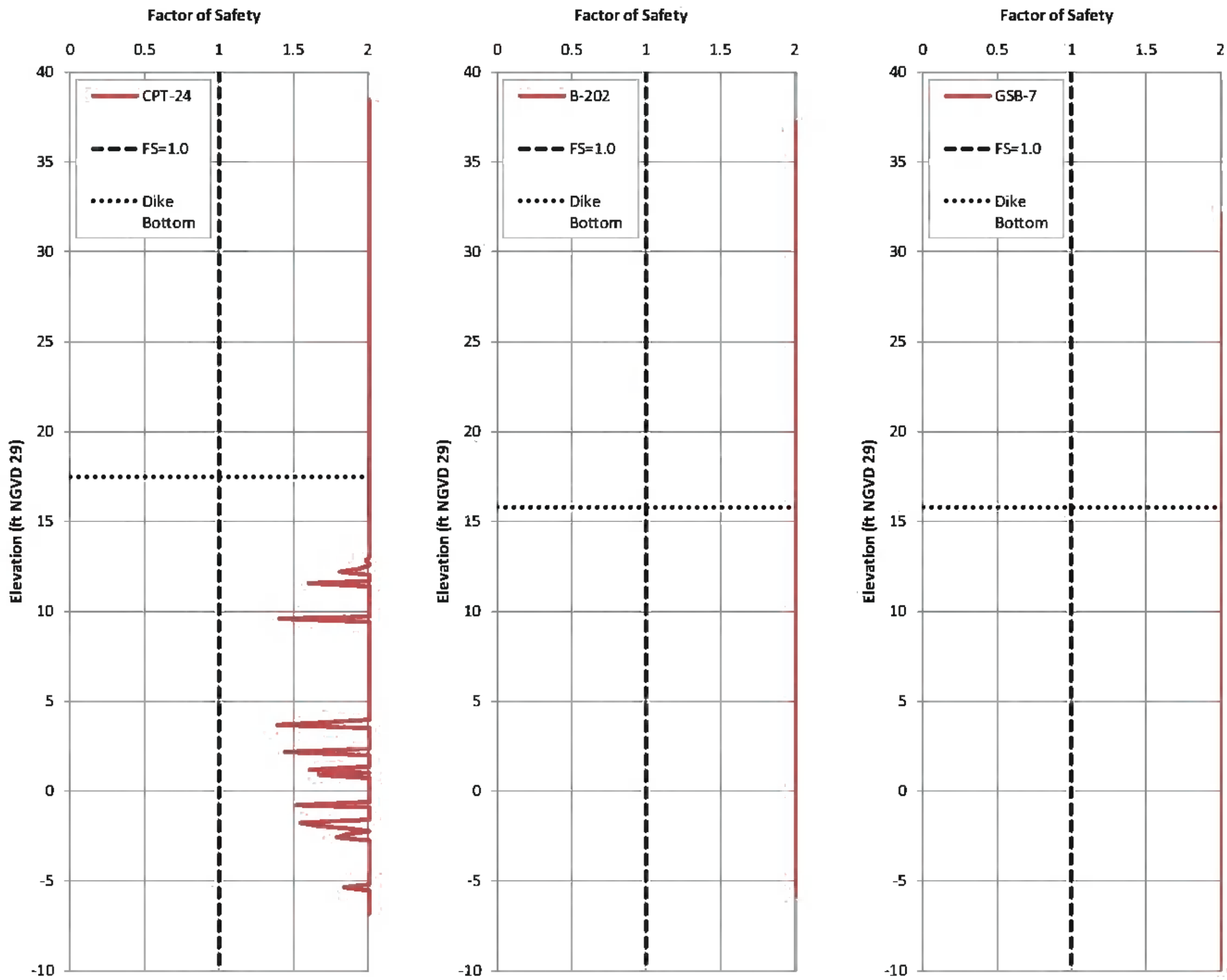

Figure 10. Liquefaction Results for Dike and Foundation Soils for CPT-24, B-202, and GSB-7

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-20 and SC-21 for CPT-24, and B-202/ GSB-7, respectively, as provided in Table 5.

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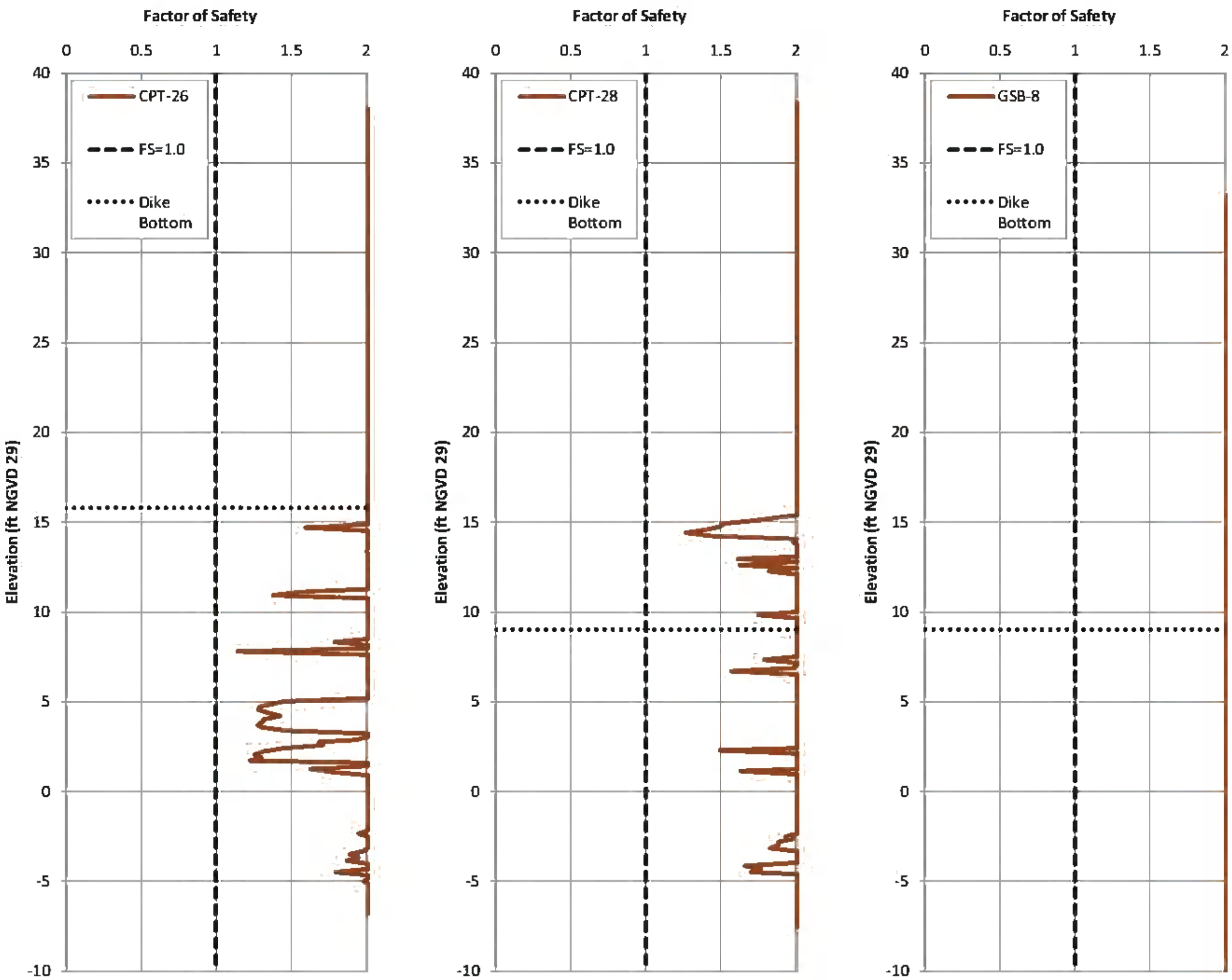

Figure 11. Liquefaction Results for Dike and Foundation Soils for CPT-26, CPT-28, and GSB-8

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-21 and SC-22 for CPT-26 and CPT-28/GSB-8, respectively, as provided in Table 5.

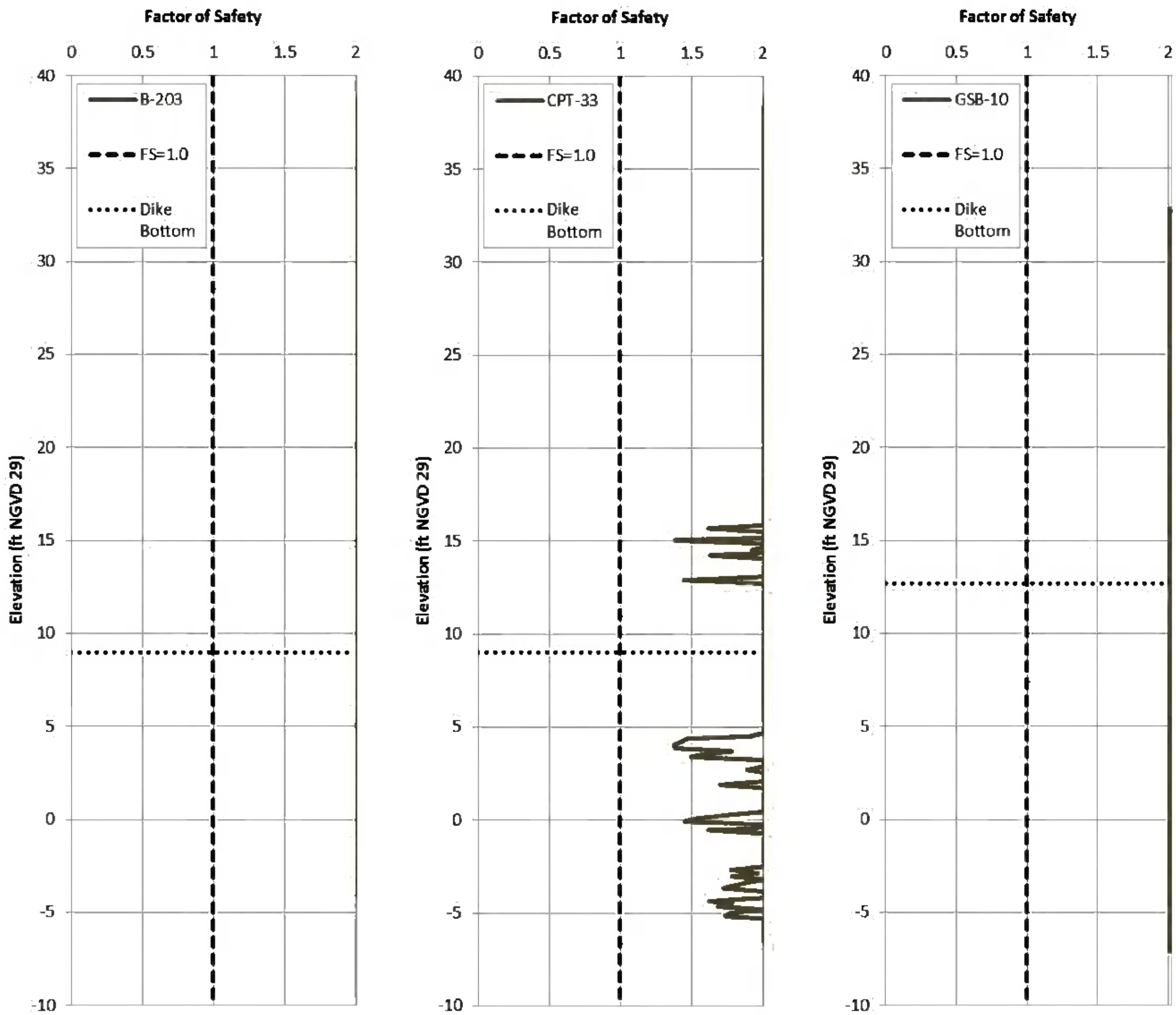

Figure 12. Liquefaction Results for Dike and Foundation Soils for B-203, CPT-33, and GSB-10

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-22 and SC-32 for B-203/CPT-33 and GSB-10, respectively, as provided in Table 5.

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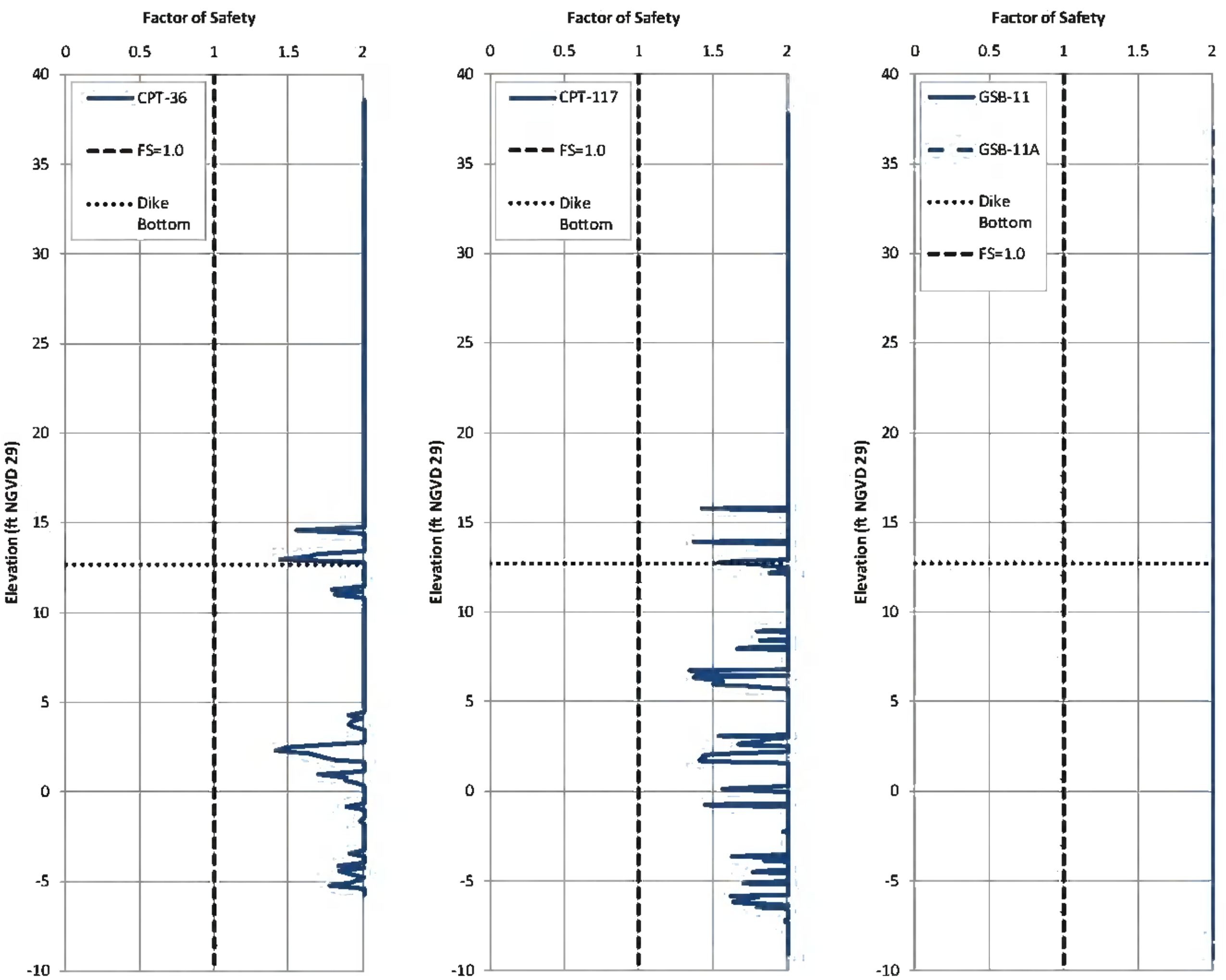

Figure 13. Liquefaction Results for Dike and Foundation Soils for CPT-36, CPT-117, and GSB-11

Notes:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical boring SC-32 for CPT-36, CPT-117, and GSB-11/GSB-11A as provided in Table 5.
2. GSB-11A was plotted with GSB-11 as these two borings were performed at nearly the same location.

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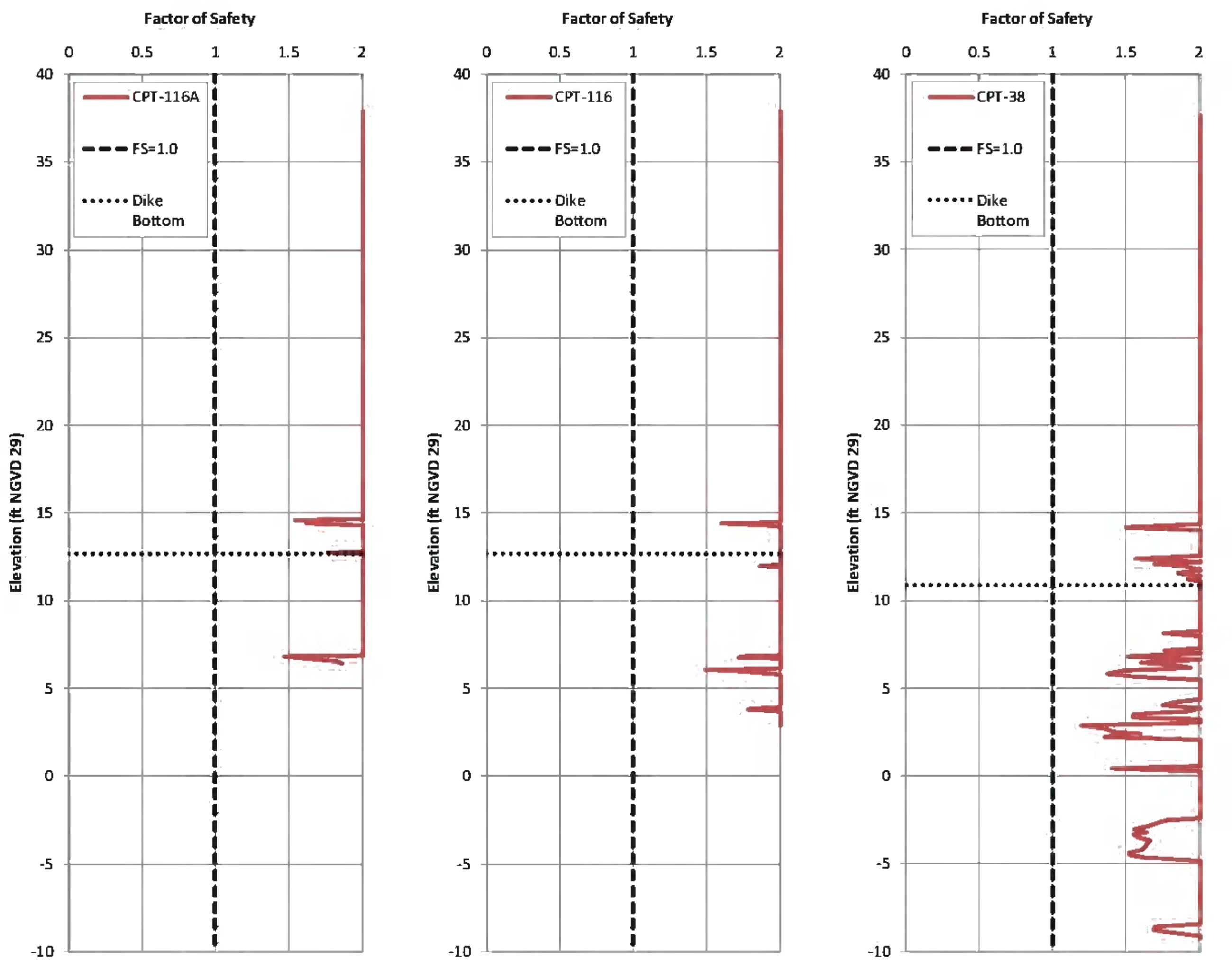

Figure 14. Liquefaction Results for Dike and Foundation Soils for CPT-116A, CPT-116, and CPT-38

Notes:

1. CPT-116A and CPT-116 were not advanced to refusal.
2. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings SC-32 and SC-33 for CPT-116A/CPT-116 and CPT-38, respectively, as provided in Table 5.

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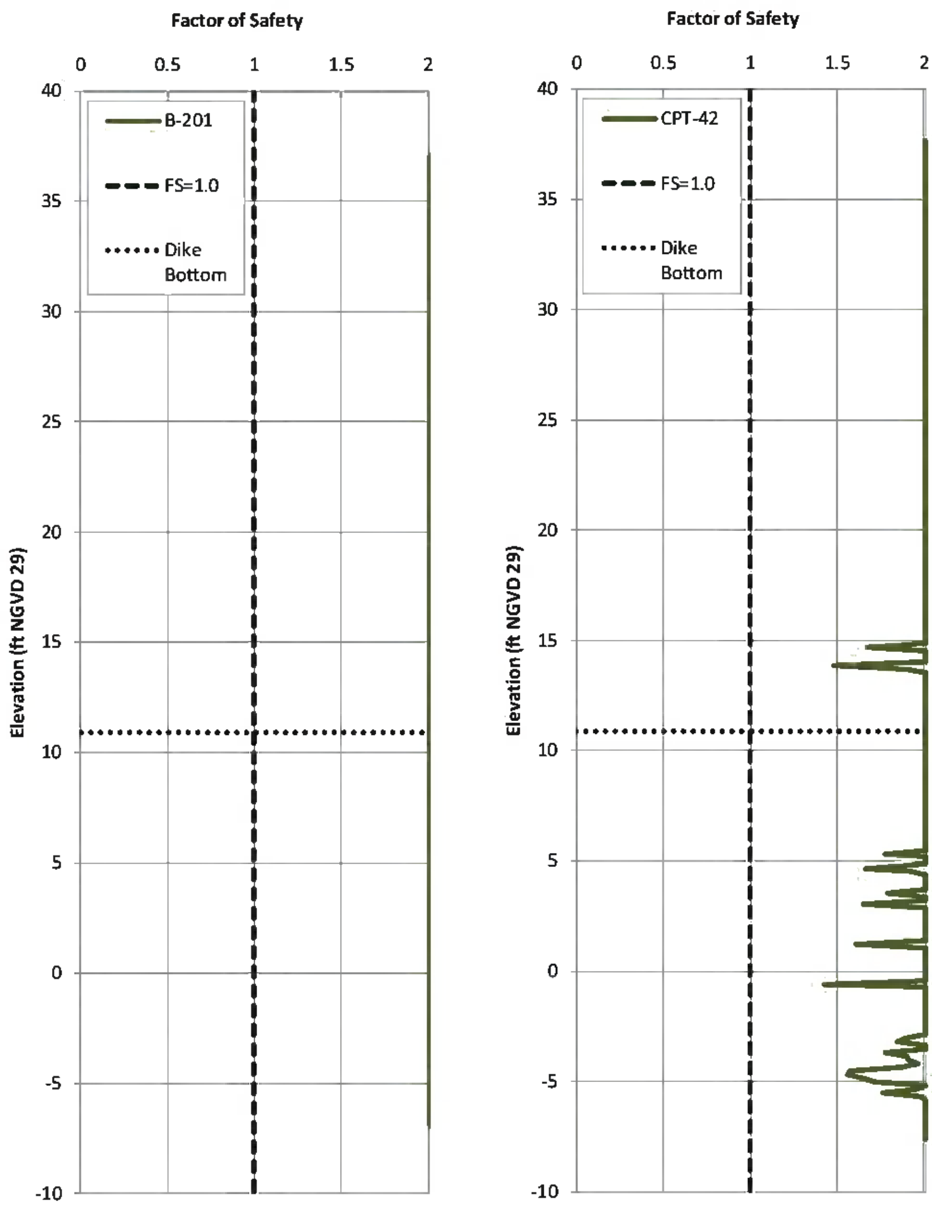

Figure 15. Liquefaction Results for Dike and Foundation Soils for B-201, CPT-42, and CPT-43

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical boring SC-33 for B-201 and CPT-42 as provided in Table 5.

Appendix 1

MathCAD® Example Calculation

CPT - based Liquefaction Analysis $BoringID := "CPT\_122"$

Site Parameters:

Age Correction Factor of Pleistocene Soils: $K_{dr} := 1.3$

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $Prof := "Profile1"$

$$CyclicStress := \begin{cases} READEXCEL("SAP\_Profile\_1.xlsx") & \text{if } Prof = "Profile1" \\ READEXCEL("SAP\_Profile\_2.xlsx") & \text{if } Prof = "Profile2" \end{cases}$$

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:

$Full := READEXCEL(concat(BoringID, ".xlsx"))$

$Data := submatrix(Full, 2, rows(Full) - 1, 0, cols(Full) - 1)$

$depth := Data^{\langle 0 \rangle} \cdot ft$ $qc := Data^{\langle 1 \rangle} \cdot tsf$ $f_s := Data^{\langle 2 \rangle} tsf$ $u_2 := Data^{\langle 3 \rangle} tsf$ $Fines := Data^{\langle 4 \rangle}$

Simple counter used in the Algorithm: $i := 0 .. rows(Data) - 1$

Tip net area ratio: $a := 1$ Correction applied when converting Hogentogler Data (.cpt) to Excel (.xls) format.

Boring Information Inputs:

Boring Elevation: $Elevation := 38.82ft$ NGVD 29

Groundwater Depth at Time of Boring (TOB): $GWT_b := 8.46ft$ bgs

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $Elev_h := 17.6ft$ NGVD 29

Sounding elevation profile: $Elev := Elevation - depth$

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	$\gamma_1 := 115pcf$
2. Silty Sand	$\gamma_2 := 105pcf$
3. Sandy silt and silt	$\gamma_3 := 100pcf$
4. Silty clay/Clayey silt	$\gamma_4 := 90pcf$
5. Clay	$\gamma_5 := 90pcf$
Water	$\gamma_{water} := 62.4pcf$

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:

$$s01(x) := linterp\left(S01^{\langle 0\rangle}, S01^{\langle 1\rangle}, x\right) \qquad s02(x) := linterp\left(S02^{\langle 0\rangle}, S02^{\langle 1\rangle}, x\right)$$

$$s03(x) := linterp\left(S03^{\langle 0\rangle}, S03^{\langle 1\rangle}, x\right) \qquad s04(x) := linterp\left(S04^{\langle 0\rangle}, S04^{\langle 1\rangle}, x\right)$$

Rough estimate (initial guess) of unit weight based on Robertson 1983 soil classification:

$$\text{class}_{1983} := \left| \begin{array}{l} \text{for } i \in 0 .. \text{rows}(qt) - 1 \\ \quad \left| \begin{array}{l} \text{class}_i \leftarrow 5 \\ \text{class}_i \leftarrow 4 \;\text{ if } \dfrac{qt_i}{100 \cdot kPa} \geq s04(Rf_i) \\ \text{class}_i \leftarrow 3 \;\text{ if } \dfrac{qt_i}{100 \cdot kPa} \geq s03(Rf_i) \\ \text{class}_i \leftarrow 2 \;\text{ if } \dfrac{qt_i}{100 \cdot kPa} \geq s02(Rf_i) \\ \text{class}_i \leftarrow 1 \;\text{ if } \dfrac{qt_i}{100 \cdot kPa} \geq s01(Rf_i) \end{array} \right. \\ \text{class} \end{array} \right.$$

$$\gamma 1 := \left| \begin{array}{l} \text{for } i \in 0 .. \text{rows}(qt) - 1 \\ \quad \text{for } m \in 1 .. 5 \\ \quad\quad \gamma 1_i \leftarrow \gamma_m \;\text{ if } \text{class}_{1983_i} = m \\ \gamma 1 \end{array} \right.$$

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:

$$u_{0_i} := \left| \begin{array}{l} (\text{depth}_i - GWT_b) \cdot \gamma_{water} \;\text{ if } \text{depth}_i > GWT_b \\ 0 \;\text{ otherwise} \end{array} \right.$$

Calculating Total and Effective Overburden Pressure

$$\sigma_{v0_i} := \left| \begin{array}{l} (\text{depth}_i - \text{depth}_{i-1}) \cdot \left(\dfrac{\gamma 1_i + \gamma 1_{i-1}}{2} \right) + \sigma_{v0_{i-1}} \;\text{ if } i > 0 \\ \text{depth}_i \cdot \gamma 1_0 \;\text{ otherwise} \end{array} \right.$$

$$\sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}} \cdot 100$$

Calculation of Robertson (1990) plot parameters

Unit weight values to be assigned to Robertson (1990) classification:

Unit weight adjusted to according to specific site conditions:

1. Sensitive, fine grained $\gamma_1 := 85\text{pcf}$
2. Organic Soils-peat to Clay $\gamma_2 := 100\text{pcf}$
3. Clay mixtures $\gamma_3 := 100\text{pcf}$
4. Silt mixtures $\gamma_4 := 100\text{pcf}$
5. Sand mixtures $\gamma_5 := 110\text{pcf}$
6. Sands $\gamma_6 := 120\text{pcf}$
7. Gravelly sand to sand $\gamma_7 := 125\text{pcf}$
8. Very stiff sand to clayey sand $\gamma_8 := 105\text{pcf}$
9. Very stiff fine grained $\gamma_9 := 105\text{pcf}$

Refined soil classification and assigning unit weights

Compute Soil Behavior Index (I_c) corresponding to initial unit weight classification:

$$I_{c_i} := \left[\left(3.47 - \log\left(Q_{t_i}\right)\right)^2 + \left(\log\left(F_{r_i}\right) + 1.22\right)^2\right]^{.5}$$

Soil classification routine for Robertson (2010) (updated from Robertson, 1990) plot:

$$\text{class}_{2010} := \left| \begin{array}{l} \text{for } i \in 0 .. \text{rows}(Q_t) - 1 \\ \quad \left| \begin{array}{l} \text{class}_i \leftarrow 2 \\ \text{class}_i \leftarrow 3 \text{ if } 2.95 < I_{c_i} \le 3.6 \\ \text{class}_i \leftarrow 4 \text{ if } 2.60 < I_{c_i} \le 2.95 \\ \text{class}_i \leftarrow 5 \text{ if } 2.05 < I_{c_i} \le 2.60 \\ \text{class}_i \leftarrow 6 \text{ if } 1.31 < I_{c_i} \le 2.05 \\ \text{class}_i \leftarrow 7 \text{ if } I_{c_i} \le 1.31 \end{array} \right. \\ \text{class} \end{array} \right.$$

Assigning unit weight based on soil classification:

$$\gamma_{fin} := \left| \begin{array}{l} \text{for } i \in 0 .. \text{rows}(Q_t) - 1 \\ \quad \text{for } m \in 1 .. 9 \\ \qquad \gamma 2_i \leftarrow \gamma_m \text{ if } \text{class}_{2010_i} = m \\ \gamma 2 \end{array} \right.$$

▲ Refined soil classification and assigning unit weights

▼ Applying Robertson (2010) values for remaining calculations:

$$\gamma := \gamma_{fin} \qquad class := class_{2010}$$

Final Static Pore Pressure Calculation for CPT interpretation:

$$u_{0_i} := \begin{cases} (depth_i - GWT_b)\cdot\gamma_{water} & \text{if } depth_i > GWT_b \\ 0 & \text{otherwise} \end{cases}$$

Total and Effective Overburden Pressure Final Calculation for CPT interpretaiton:

$$\sigma_{v0_i} := \begin{cases} (depth_i - depth_{i-1})\cdot\left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} & \text{if } i > 0 \\ depth_0\cdot\gamma_0 & \text{otherwise} \end{cases} \qquad \sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$Q_{t_i} := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0_i}}{qt_i - \sigma_{v0_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0_i}}\cdot 100$$

$$Q_i := \frac{qt_i - \sigma_{v0_i}}{\sigma_{v0eff_i}}$$

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:

$$q_{c1\_it} := \left| \begin{array}{l} c \leftarrow 0 \\ \text{"initial CN"} \\ \text{for } i \in 0..\text{rows}(qt)-1 \\ \quad C_{N_i} \leftarrow 1.7 \\ \text{for } i \in 0..\text{rows}(qt)-1 \\ \quad \left| \begin{array}{l} \text{while } c < 500 \\ \quad \left| \begin{array}{l} q_{c1_i} \leftarrow C_{N_i} \cdot qt_i \\ q_{c1N_i} \leftarrow \dfrac{q_{c1_i}}{1\text{atm}} \\ C_{N_i} \leftarrow \min\left[1.7, \left(\dfrac{1\text{atm}}{\sigma_{v0eff_i}}\right)^{1.338-0.249\cdot\left(\max\left(21,\min\left(q_{c1N_i},254\right)\right)\right)^{0.264}}\right] \\ c \leftarrow c+1 \end{array}\right. \\ c \leftarrow 0 \end{array}\right. \\ \left(\dfrac{q_{c1}}{psf} \quad q_{c1N}\right) \end{array}\right.$$

$$q_{c1} := \left(q_{c1\_it}{}^{\langle 0 \rangle}\right)_0 psf \qquad q_{c1N} := \left(q_{c1\_it}{}^{\langle 1 \rangle}\right)_0$$

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):

$i := 0..\text{rows}(qc) - 1$

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: $q_{c1Ncs_i} := q_{c1N_i} + \Delta q_{c1N_i}$

$$CRR_i := \begin{cases} \exp\left[\frac{q_{c1Ncs_i}}{540} + \left(\frac{q_{c1Ncs_i}}{67}\right)^2 - \left(\frac{q_{c1Ncs_i}}{80}\right)^3 + \left(\frac{q_{c1Ncs_i}}{114}\right)^4 - 3\right] & \text{if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} < 211 \\ 2.0 & \text{if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} > 211 \\ 2.0 & \text{otherwise} \end{cases}$$

Overburden Correction Factor (Kσ) for Sands:

$$C_{\sigma_i} := \min\left[\frac{1}{37.3 - 8.27 \cdot \left(\min\left(q_{c1N_i}, 211\right)\right)^{0.264}}, 0.3\right]$$

$$K_{\sigma_i} := \begin{cases} \min\left(1 - C_{\sigma_i} \cdot \ln\left(\frac{\sigma_{v0eff_i}}{1tsf}\right), 1.1\right) & \text{if } I_{c_i} \le 2.60 \\ 1.0 & \text{otherwise} \end{cases}$$

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.

$$CRR_{final_i} := \begin{cases} CRR2_i \cdot K_{dr} & \text{if } I_{c_i} \le 2.60 \wedge Elev_i < Elev_h \\ CRR2_i & \text{otherwise} \end{cases}$$

▲ Compute CRR with Overburden, MSF, and Kdr Corrections

▼ Compute CSR and FS

Compute the CSR for the Soil Profile

$$\tau_{cyc} := \text{submatrix}(CyclicStress, 1, \text{rows}(CyclicStress) - 1, 1, 1)\,psf$$

$$d_{cyc} := \text{submatrix}(\text{CyclicStress}, 1, \text{rows}(\text{CyclicStress}) - 1, 0, 0)\ \text{ft}$$

$$\tau_{max} := \text{linterp}(d_{cyc}, \tau_{cyc}, \text{depth})$$

$$CSR_i := \frac{0.65\ \tau_{max_i}}{\sigma_{v0eff_i}}$$

Compute Factor of Safety

$$FS_i := \begin{cases} 2.00 & \text{if } \text{depth}_i < GWT_b \\ \min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.00\right) & \text{otherwise} \end{cases}$$

Compute CSR and FS

Export Results:

Headers := augment("Depth" , "Elevation" , "qc1N" , "SBT Index" , "FScyclic")

Units := augment("ft" , "ft NGVD29" , "-" , "-" , "-")

$$\text{Export} := \text{augment}\left(\frac{\text{depth}}{\text{ft}}, \frac{\text{Elev}}{\text{ft}}, q_{c1N}, I_c, FS\right)$$

Export2 := stack(Headers, Units, Export)

FileName := concat(BoringID, "_Results" , ".xlsx")

Export3 := WRITEEXCEL(Export2, FileName)

SPT - based Liquefaction Analysis $BoringID := "SPT-109"$

Site Parameters:

Age Correction Factor: $K_{dr} := 1.3$ (Geosyntec, 2013)

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $Prof := "Profile1"$

$$CyclicStress := \begin{cases} READEXCEL("SAP\_Profile\_1.xlsx") & \text{if } Prof = "Profile1" \\ READEXCEL("SAP\_Profile\_2.xlsx") & \text{if } Prof = "Profile2" \end{cases}$$

SPT-Specific data:

Import the SPT-Specific Data in the form of Depth, Blow Count, Visual Classification ("Sand-Like"/"Clay-Like"), fines content, and USCS Classification. Upper two rows contain the headers and units for each field:

$Full := READEXCEL(concat(BoringID, ".xlsx"))$

$Data := submatrix(Full, 2, rows(Full) - 1, 0, cols(Full) - 1)$

$depth := Data^{\langle 0 \rangle} \cdot ft$ $N_{blows} := Data^{\langle 1 \rangle}$ $Class := Data^{\langle 2 \rangle}$ $Fines := Data^{\langle 3 \rangle}$ $USCS := Data^{\langle 4 \rangle}$

Boring Information:

Boring Elevation: $Elevation := 37.39ft$ NGVD29

Groundwater Depth: $GWT := 5.1ft$ bgs

Boring Diameter: $Diameter := 4$ inches

Holocene Elevation: $Elev_h := 16.50ft$ NGVD29

Energy Calibration: $ER := 88$ % (SCI, 2014)

Sampling Method: $C_S := 1.0$

$RodDepth := depth + 5ft$ (Assume 5 ft of rod stick up during SPT test)

Miscelaneous Constants:

Defining external units: $tsf := \frac{tonf}{ft^2}$ $kPa := \frac{1}{95.760518} tsf$

Compute Calibration Factors and N60

Compute Calibration Factors

$$C_E := \frac{ER}{60}$$

$$C_B := \begin{cases} 1.0 & \text{if Diameter} \le 4.0 \\ 1.05 & \text{if } 4.0 < \text{Diameter} < 6.0 \\ 1.15 & \text{otherwise} \end{cases}$$

$$C_R := \left| \begin{array}{l} \text{for } i \in 0..\text{rows(depth)} - 1 \\ \quad \left| \begin{array}{l} rod_i \leftarrow 0.75 \ \text{ if RodDepth}_i \le 13ft \\ rod_i \leftarrow 0.85 \ \text{ if } 13ft < \text{RodDepth}_i \le 20ft \\ rod_i \leftarrow 0.95 \ \text{ if } 20ft < \text{RodDepth}_i \le 33ft \\ rod_i \leftarrow 1 \ \text{ otherwise} \end{array} \right. \\ rod \end{array} \right.$$

Compute N_{60}:

$$N_{60} := \left| \begin{array}{l} \text{for } i \in 0..\text{rows(depth)} - 1 \\ \quad x_i \leftarrow C_B \cdot C_E \cdot C_S \cdot N_{blows_i} \cdot C_{R_i} \\ x \end{array} \right.$$

Compute Calibration Factors and N60

Calculation of CN and Effective Overburden Stress

Compute C_N:

Develop Representative Unit Weight Profile

Unit weight values to be assigned based on density and material class:

Adjust according to specific site conditions

1. Coal Combustion Residuals $\gamma_1 := 100pcf$
2. Loose Sands (N_{blows} <10) $\gamma_2 := 105pcf$
3. Medium Dense Sands (10<N_{blows} <30) $\gamma_3 := 115pcf$
4. Dense Sands $\gamma_4 := 120pcf$
5. Soft Clays $\gamma_5 := 100pcf$
6. Chicora Member $\gamma_6 := 130pcf$
7. Williamsburg Formation Clay $\gamma_7 := 105pcf$

Relate depth to elevation to screen unit weights for Williamsburg Formation Clay

$Elev := (Elevation - depth)$ $\qquad$ $WMElev := -8ft$ $\qquad$ (Approx. Top of Williamsburg Formation Clay)

$$Class_2 := \left| \begin{array}{l} \text{for } i \in 0..rows(depth)-1 \\ \quad \left| \begin{array}{l} y_i \leftarrow 7 \\ y_i \leftarrow 1 \text{ if } Class_i = \text{"SILT"} \\ y_i \leftarrow 2 \text{ if } Class_i = \text{"SAND"} \wedge N_{blows_i} \le 10 \\ y_i \leftarrow 3 \text{ if } Class_i = \text{"SAND"} \wedge 10 < N_{blows_i} < 30 \\ y_i \leftarrow 4 \text{ if } Class_i = \text{"SAND"} \wedge 30 < N_{blows_i} \\ y_i \leftarrow 5 \text{ if } Class_i = \text{"CLAY"} \wedge Elev_i > WMElev \\ y_i \leftarrow 6 \text{ if } Class_i = \text{"CHICORA"} \end{array} \right. \\ y \end{array} \right.$$

Assign unit weight based on soil classification:

$$\gamma_{fin} := \left| \begin{array}{l} \text{for } i \in 0..rows(depth)-1 \\ \quad \text{for } m \in 1..7 \\ \qquad \gamma 2_i \leftarrow \gamma_m \text{ if } Class_{2_i} = m \\ \gamma 2 \end{array} \right.$$

$\gamma := \gamma_{fin}$ $\qquad$ $\gamma_{water} := 62.4pcf$

$i := 0..rows(depth) - 1$

Final Static Pore Pressure Calculation:

$$u_{0_i} := \left| \begin{array}{l} (depth_i - GWT)\cdot\gamma_{water} \text{ if } depth_i > GWT \\ 0 \text{ otherwise} \end{array} \right.$$

Total and Effective Overburden Pressure Final Calculation:

$$\sigma_{v0_i} := \left| \begin{array}{l} (depth_i - depth_{i-1})\cdot\left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} \text{ if } i > 0 \\ depth_0\cdot\gamma_0 \text{ otherwise} \end{array} \right.$$

$$\sigma_{v0eff} := \sigma_{v0} - u_0$$

Calculation of C_{NL} (For Liquefaction) [SCDOT, 2010 - pg. 13-48] Calulation limitted to a maximum N-value = 46 blows/ft)

```
C_NLit := | c ← 0
          | "initial CN"
          | for i ∈ 0 .. rows(depth) − 1
          |   C_N_i ← 1.7
          | for i ∈ 0 .. rows(depth) − 1
          |   | while c < 600
          |   |   | N160L_i ← C_N_i · N60_i
          |   |   | C_N_i ← min[1.7, (1atm / σv0eff_i)^(0.784 − 0.0768·√(min(46, N160L_i)))]
          |   |   | c ← c + 1
          |   | c ← 0
          | (C_N  N160L)
```

$$C_{NL} := \left(C_{NLit}^{\langle 0 \rangle}\right)_0 \qquad N_{160_i} := C_{NL_i} \cdot N_{60_i}$$

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]

```
ΔN160L := | for i ∈ 0 .. rows(depth) − 1
          |   x_i ← min[5.5, exp[1.63 + [9.7 / (Fines_i + 0.01)] − [15.7 / (Fines_i + 0.01)]^2]]
          | 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

$$FS_i := \begin{cases} 2.01 & \text{if } Class_i = \text{"CHICORA"} \\ 2.01 & \text{if } Class_i = \text{"CLAY"} \\ 2.01 & \text{if } depth_i < GWT \\ \min\left(\frac{CRR_{final_i}}{CSR_i}, 2.01\right) & \text{otherwise} \end{cases}$$

-Assume Chicora statum does NOT Liquefy

Compute CSR and FS

Evaluate the Soil Strength Loss (SSL) due to Cyclic Liquefaction or Cyclic Softening:

Export Results:

Headers := augment("Depth" , "Elevation" , "N160" , "Class" , "FScyclic")

Units := augment("ft" , "ft NGVD29" , "-" , "-" , "-")

$$Export := \text{augment}\left(\frac{depth}{ft}, \frac{Elev}{ft}, N_{160}, Class, FS\right)$$

Export2 := stack(Headers, Units, Export)

FileName := concat(BoringID, "_Results" , ".xlsx")

Export3 := WRITEEXCEL(Export2, FileName)

Export Liquefaction Results

ATTACHMENT 8

Safety Factor Assessment

Geosyntec consultants

CALCULATION PACKAGE COVER SHEET

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

TITLE OF PACKAGE: **SAFETY FACTOR ASSESSMENT: SLURRY POND**

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

SAFETY FACTOR ASSESSMENT: SLURRY POND

INTRODUCTION

This calculation package was prepared as Attachment 8 to the *2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond* (Safety Factor Assessment Report) and presents the slope stability analyses for the Slurry Pond 3&4 (Slurry Pond) perimeter dikes at Winyah Generating Station (WGS). The Slurry Pond is a 106-acre surface impoundment, which manages coal combustion residuals (CCR) in the form of flue gas desulfurization (FGD) residuals generated from the wet scrubber system. 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 Slurry 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 Slurry Pond perimeter dikes achieve the safety factor (also referred to as "factor of safety") criteria of §257.73(e)(1) of the CCR Rule. §257.73(e)(1) requires that:

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

It is noted that the liquefaction potential analysis results presented in Attachment 7: *Liquefaction Potential Analysis: Slurry Pond* of the Safety Factor Assessment Report did not indicate that the Slurry Pond dike fill or foundation soils immediately beneath the perimeter dikes were susceptible to liquefaction. Therefore, the liquefaction factor of safety (FS) for the Slurry Pond perimeter dikes utilizing post-liquefaction residual shear strengths was not evaluated as a part of this safety factor assessment.

METHODOLOGY

Static Slope Stability

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

Both the rotational mode (i.e., the circular slip surface mode) and the non-rotational mode (i.e., the block slip surface mode) were considered during these analyses, and the slip mode 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 includes the slope geometry, the subsurface soil stratigraphy, the phreatic surface elevation, the external loading conditions, and the 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: Slurry Pond* (Site Response Package) of this Safety Factor Assessment Report.

2. Compute the seismic horizontal force coefficient (k_h) using the ratio of the critical acceleration (N) to the peak value of earthquake acceleration (A) based 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 ($F = k_h \times W$), for each slice based on slice weight (W), and evaluate the resulting FS. If the calculated FS meets or exceeds the target FS (i.e., $FS \geq 1.0$), the slope is considered to be stable and to meet the requirements of the CCR Rule.

It is noted that during pseudo-static slope stability analyses, undrained shear strengths should be reduced by 20 percent to account for potential strength degradation during cyclic loading.

CROSS SECTION GEOMETRY

The following section describes the development for the: (i) external geometry, (ii) subsurface stratigraphy; and (iii) water levels for the cross sections evaluated as a part of this safety factor assessment.

External Geometry

The heights of the Slurry Pond perimeter dikes are approximately 30 feet (ft) to the north and east, 26 ft to the west, and 15 ft to the southeast. The upstream and downstream side slopes range from 2 horizontal to 1 vertical (2H:1V) to 3H:1V; while, the dike crest is typically 12- to 15-ft wide (Thomas and Hutton, 2012). To the northeast, east, and southeast of the perimeter dikes, a drainage channel has been excavated as part of WGS's stormwater management plan.

Five cross sections were developed and evaluated during this safety factor assessment. The external geometry of each cross section was developed based on the topographic survey prepared by Thomas and Hutton (2012), the original design contours, and the design of the western rim ditch. 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 using AutoCAD® and exported directly into the SLIDE® program. The location and extent of each analyzed cross section are depicted in Figure 1.

It is noted that Cross Section B was selected to demonstrate that perimeter dikes adjacent to soil boring GSB-11/GSB-11A meet the required safety factors listed in the CCR Rule, since weight-of-hammer material was observed within the perimeter dikes during GSB-11 advanced in 2013. GSB-11A was performed in 2016 to collect additional data and to perform laboratory shear strength tests on collected soil samples of dike fill materials. The weight-of-hammer material was not observed during GSB-11A.

Subsurface Stratigraphy

The subsurface stratigraphy for each cross section was developed based on soil borings, tests pits, and cone penetration tests conducted as a part of prior investigations (S&ME, 1978; PCRA, 1999; Geosyntec, 2013) and one soil boring (GSB-11A) performed in 2016 (Attachment 5). Generally, the subsurface in the depth of interest for slope stability analyses consists of the following strata: Dike Fill, Foundation Soils, Chicora Member, and Williamsburg Formation Clay. Further discussion on the development of subsurface conditions can be found in Attachment 5: *Subsurface Stratigraphy and Material Properties: Slurry Pond (*Data Package) of the Safety Factor Assessment Report.

Water Levels

The CCR Rule requires the evaluation of safety factors considering static slope stability analyses under both the long-term "Maximum Normal Storage Pool" and the "Maximum Surcharge Pool" loading conditions. As described within the Hydrologic and Hydraulic (H&H) analysis for the Slurry Pond provided in Attachment 1 of the Safety Factor Assessment Report, the water level within the Slurry Pond is maintained at an elevation of 19.6 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a Floating Pump Station installed to lower the free water within the surface impoundment. The Floating Pump Station routes process water and stormwater from the Slurry Pond ultimately to the Discharge Canal and WGS's Cooling Pond. The operating level of 19.6 ft NGVD29 was selected as the "Maximum Normal Storage Pool" within the Slurry Pond. Since the Slurry Pond is considered a "High Hazard Potential" surface impoundment, the Probable Maximum Flood (PMF) was selected as the Inflow Design Flood (IDF) as required by §257.73(d)(1)(B). The maximum surface water elevation within the Slurry Pond was computed as 35.4 ft NGVD29 by the H&H analysis (Attachment 1), which was selected as the "Maximum Surcharge Pool" for the safety factor assessment.

The phreatic surface through the perimeter dikes to the downstream toe at the time of this safety factor assessment was developed based on water levels collected from temporary piezometers installed through the Slurry Pond perimeter dike centerline in 2014 (Figure 1), with the consideration of depth to water measurements collected during the subsurface investigation in 2013. Water elevations measured from PPZ-31 and PPZ-32, which were located in the western and northwestern corners of the perimeter dike centerline, were found to range between 12.0 and 16.0 ft NGVD29 during the previous operating levels of the Slurry Pond. As free water was removed from the Slurry Pond (W-SW-WSP), water level elevations measured in temporary piezometers PPZ-31 and PPZ-32 lowered over time. Based on the water level measurements with these piezometers during lowering of the Slurry Pond free water, the phreatic surface through the perimeter dike centerline was selected be 12.6 ft NGVD29 for "Maximum Normal Storage Pool" conditions and 16.0 ft NGVD29 for "Maximum Surcharge Pool" conditions for Cross Sections A and B. In March 2016, Santee Cooper installed a temporary standpipe adjacent to GSB-11A. After 96-hrs, the depth to water within the temporary standpipe was measured as 24.4 ft bgs (or at an elevation of 13.4 ft NGVD29) before the instrument was pulled and abandoned.

Because Cross Sections C and D are located in the eastern corner of the Slurry Pond and are distanced from temporary piezometers PPZ-31 and PPZ-32, the depths to water level estimated from CPT porewater pressure measurements and from mud rotary and hollow stem auger boreholes were utilized to estimate the phreatic surface elevation through the Slurry Pond perimeter dikes. During the "Maximum Normal Storage Pool" conditions, measurements from PPZ-33 were selected to model the phreatic surface within the FGD residuals as 29.5 ft NGVD29; while, CPT soundings were used to select the phreatic surface through the dike fill material. During the "Maximum Surcharge Pool" conditions, a phreatic surface elevation of 28.0 ft

NGVD29 through the perimeter dike centerline was selected for Cross Sections C and D. Note that in these areas a rim ditch has been constructed adjacent to the upstream perimeter dike slope since the most recent survey was collected. This rim ditch allows free water to drain to the north corner of the surface impoundment. The depth of this rim ditch was not measured during the most recent dike inspection and thus was not included within this analysis; however, it is anticipated that this rim ditch will facilitate the lowering of the phreatic surface within the FGD residuals and the perimeter dikes.

In both the "Maximum Normal Storage Pool" and "Maximum Surcharge Pool" conditions, the phreatic surface through the Slurry Pond perimeter dikes was assumed to reach steady-state conditions. During this safety factor assessment, static slope stability was evaluated using the "Maximum Normal Storage Pool" conditions and "Maximum Surcharge Pool" conditions; while, the seismic safety factor was evaluated using the "Maximum Normal Storage Pool" conditions as described above.

Cross Section Models

The 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) based on the in-situ and laboratory data collected in the vicinity of the Slurry Pond. Table 1 provides the summary of the material properties selected for the slope stability analyses. It was assumed that seismic waves generated during a potential seismic event would load clayey foundation soils rapidly enough to induce an undrained loading condition within the clayey soils. In accordance with recommendations made within Hynes-Griffin and Franklin (1984), the selected undrained shear strength values were reduced by 20% for the seismic safety factor case to account for potential cyclic degradation during an earthquake at the Site. It is noted that sandy foundation materials where the computed factor of safety against liquefaction (FS_{Liq}) is less than 1.4 were modeled using an R_u coefficient (R_u=0.5) to account for excess porewater pressure generation during a seismic event (Marcuson et al, 1990).

Seismic Loading and Allowable Displacement

An evaluation of the seismic hazard for WGS and the site response analysis for the Slurry Pond perimeter dikes is presented in the *Seismic Hazard and Site Response Package: Slurry Pond* (Attachment 6) of this Safety Factor Assessment Report. Within Attachment 6, six ground motions for WGS were evaluated for three representative dike soil profiles for the Slurry 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 the seismic stability procedures by Bray et al. (1995). Preliminary pseudo-static analyses of the perimeter dikes structures of the Slurry Pond indicated that the critical depth of the anticipated slip surface is approximately 20 ft below the dike crest. Thus, the maximum MHEA at 20 ft below the dike crest computed from the set of ground motions for each representative soil profile was selected to compute the k_h. The MHEA for each ground motion and representative soil profile to a depth of 60 ft bgs is provided in Table 2. A MHEA of 0.094g, 0.085g, and 0.077g was selected for Profile 1, Profile 2, and Profile 3, respectively.

As described in the Methodology section, the k_h must be computed assuming an allowable displacement (u). An allowable displacement of 12 inches (30.48 centimeters) was selected for the Slurry 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 7. Thus, a k_h of 0.04 (rounded to the nearest hundredth) was computed for Profiles 2 and 3 (Cross Sections A, B, and C) and a k_h value of 0.05 (rounded to the nearest hundredth) was computed for Profile 1 (Cross Sections D and E).

RESULTS

The safety factor analyses for Cross Sections A through E were performed using the methodology and parameters outlined within this calculation package, and the analysis results are summarized in Table 3. Computed safety factors were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. Cross Sections D and E were computed with the lowest calculated static and seismic safety factors for each analysis case. Figures 7 through 9 and Figures 10 through 12 show the critical values of FS for Cross Section D and Cross Section E, respectively.

CONCLUSIONS

Based on the assumptions, analyses, and results presented within this calculation package, the Slurry Pond at WGS meets the periodic safety factor requirements described within the CCR Rule for existing CCR surface impoundments.

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TABLES

Table 1. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters[2]		Undrained Parameters [2]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill Material[1]	125	33[3]	100	0.65	100
Foundation Materials (Clayey)[1][5]	100	28	0[7]	0.35 to 0.40[5]	100[4]
Foundation Materials (Sandy)	115	32	0	-	-
Riprap Buttress	150	45	0	-	-
Chicora Member	130	50	0	-	-
Williamsburg Formation Clay	105	50	0	-	-
FGD Residuals[6]	95	40	0	0.5	0

Notes:

1. Undrained strength parameters for dike fill and clayey foundation soils were applied for seismic slope stability case only.
2. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the shear strengths of Dike Fill Material, Foundation Soils (Clayey), Foundation Soils (Sandy), and FGD Residuals were reduced during pseudo-static analyses by 20% to account for cyclic degradation during an earthquake. Foundation Materials (Sandy) were modeled with excess porewater pressures (R_u = 0.5) in accordance with recommendations made by Marcuson et al (1990).
3. Dike Fill within Cross Section B was modeled with a ϕ' = 31.7° based on a consolidated undrained (CU) triaxial strength test from GSB-11A. Cross Section B was analyzed to demonstrate adequate safety factors for the perimeter dikes near GSB-11, where soil borings indicated weight of hammer material within the dike fill (Geosyntec, 2013).
4. The minimum undrained shear strength ($S_{u,min}$) was applied for Cross Sections C and D.
5. Cross Sections A, B, and C were assigned a S_u/σ'_{vo} = 0.35; while, Cross Sections C and D were assigned a Su/σ'_{vo} = 0.40 based on the interpretation of CPT soundings in the vicinity of each cross section.
6. FGD residuals were modeled using undrained shear strength ratio in each case.
7. A cohesion intercept of 50 psf was applied to "clayey foundation soils" for Cross Section E under "Maximum Normal Pool" and "Maximum Surcharge Pool" conditions to address a localized slip surface, which was identified as an artifact of conservatively modeling the clayey foundation soil as a purely frictional material in this area.

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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Table 2. Maximum Equivalent Horizontal Acceleration from Site Response Analysis for Slurry Pond Perimeter Dikes

Depth (ft)	Representative Profile 1						Representative Profile 2						Representative Profile 3					
	Maximum Equivalent Horizontal Acceleration (MHEA), g						Maximum Equivalent Horizontal Acceleration (MHEA), g						Maximum Equivalent Horizontal Acceleration (MHEA), g					
	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360	BOS-T1	DEL090	RSN8529	Winyah 1	Winyah 2	YER360
2.5	0.092	0.113	0.093	0.086	0.099	0.125	0.081	0.107	0.100	0.077	0.087	0.119	0.079	0.124	0.096	0.093	0.089	0.145
7.5	0.082	0.095	0.092	0.081	0.086	0.092	0.069	0.092	0.095	0.069	0.081	0.086	0.066	0.096	0.087	0.084	0.081	0.091
12.5	0.079	0.091	0.085	0.078	0.083	0.092	0.068	0.081	0.084	0.068	0.078	0.083	0.063	0.081	0.069	0.077	0.074	0.079
17.5	0.080	0.090	0.076	0.074	0.083	0.090	0.066	0.077	0.074	0.067	0.075	0.081	0.064	0.077	0.067	0.072	0.069	0.076
20	0.085	0.093	0.078	0.076	0.087	**0.094**	0.070	0.081	0.070	0.068	0.077	**0.085**	0.066	0.077	0.065	0.069	0.071	**0.077**
22.5	0.081	0.088	0.074	0.072	0.083	0.089	0.067	0.077	0.067	0.065	0.074	0.081	0.063	0.073	0.062	0.066	0.068	0.073
27.5	0.079	0.083	0.071	0.068	0.079	0.087	0.065	0.072	0.063	0.061	0.069	0.077	0.061	0.068	0.058	0.062	0.066	0.071
32	0.076	0.076	0.066	0.064	0.074	0.082	0.063	0.066	0.058	0.057	0.063	0.072	0.060	0.067	0.056	0.060	0.065	0.070
36	0.072	0.070	0.062	0.060	0.069	0.076	0.061	0.063	0.052	0.053	0.059	0.067	0.059	0.061	0.054	0.056	0.062	0.066
40	0.068	0.066	0.056	0.057	0.063	0.071	0.058	0.058	0.048	0.051	0.055	0.061	0.057	0.058	0.049	0.052	0.056	0.061
44	0.064	0.062	0.051	0.054	0.057	0.064	0.055	0.054	0.047	0.049	0.052	0.055	0.055	0.054	0.046	0.048	0.051	0.055
48	0.061	0.059	0.049	0.052	0.056	0.060	0.052	0.051	0.045	0.045	0.049	0.052	0.052	0.051	0.045	0.046	0.049	0.052
52.5	0.067	0.058	0.047	0.049	0.055	0.063	0.057	0.049	0.043	0.044	0.048	0.056	0.059	0.050	0.044	0.045	0.049	0.055
60	0.074	0.057	0.046	0.049	0.054	0.065	0.066	0.048	0.043	0.045	0.046	0.058	0.066	0.049	0.043	0.046	0.049	0.058

Notes:

1. BOS-T1, DEL090, RSN8529, Winyah 1, Winyah 2, and YER360 refer to the selected ground motions applied for WGS during the Site Response Analysis (Attachment 6).
2. A critical depth of 20 ft was selected for the Slurry Pond slope stability analysis. MHEA values of 0.094g, 0.085g, and 0.077g were selected for Profile 1, Profile 2, and Profile 3, respectively.
3. Profile 1 is representative for Cross Sections D and E. Profile 2 is representative for Cross Section C. Profile 3 is representative for Cross Sections A and B.

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

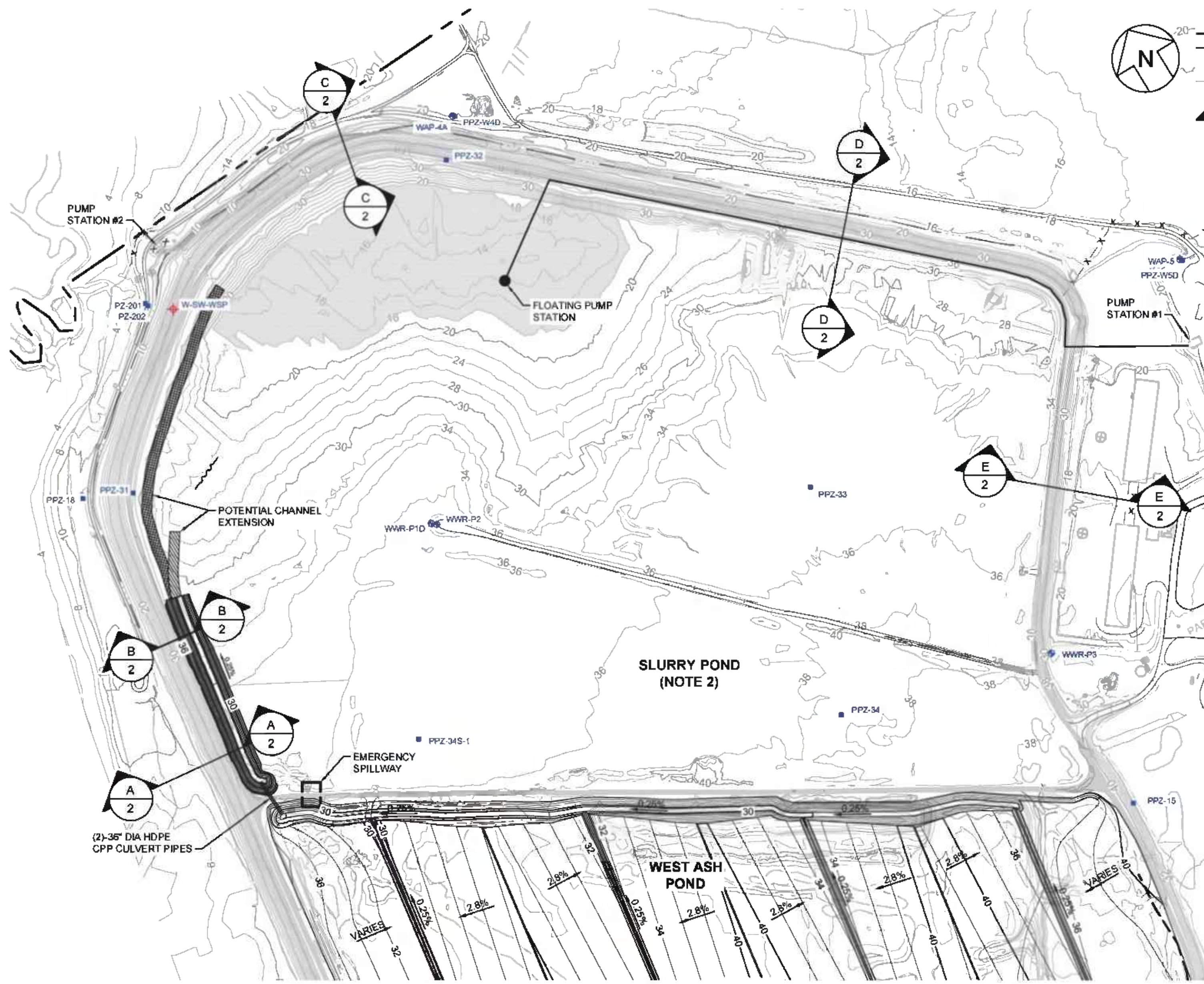
Table 3. Summary of Safety Factor Analysis Results

Factor of Safety Case	**Target FS**	**Cross Section A**	**Cross Section B**	**Cross Section C**	**Cross Section D**	**Cross Section E**
Static FS- Maximum Normal Storage Pool	1.50	1.95	1.95	1.61	*1.51*	1.60
Static FS- Maximum Surcharge Pool	1.40	1.79	1.85	1.61	*1.41*	*1.41*
Seismic FS- Maximum Normal Storage Pool	1.00	1.24	1.24	1.10	1.05	*1.03*
Liquefaction FS[1]	1.20	N/A	N/A	N/A	N/A	N/A

Notes:

1. The liquefaction safety factor was not evaluated since dike fill soils and foundation soils from soil borings and CPT soundings through the perimeter dikes were not found to be liquefiable (Attachment 7).
2. The lowest computed safety factor for each analysis case was italicized. The computed minimum FS's for Cross Section D are shown in Figures 8 through 10; while computed minimum FS's for Cross Section E are shown in Figures 11 through 13, as these cross sections were found to contain the lowest computed safety factors.

FIGURES



LEGEND

30	DESIGN MAJOR GRADE CONTOUR
10	EXISTING MAJOR GRADE CONTOUR
A/2 — A/2	CROSS SECTION LOCATION
W-SW-WSP	EXISTING STAFF GAUGE
WAP-4A, WWR-P1D	MONITORING WELL
PZ-201, PPZ-15, PPZ-W4D	PIEZOMETER

NOTES:

1. TEMPORARY PZ-31 AND PZ-32 WERE INSTALLED IN JUNE 2014 TO MEASURE THE PHREATIC SURFACE WITHIN THE SLURRY POND PERIMETER DIKES. PZ-31 AND PZ-32 WERE DESTROYED IN OCTOBER 2015 AND JUNE 2013, RESPECTIVELY.
2. THE FREE WATER LEVEL IN THE SLURRY POND IS MAINTAINED AT AN ELEVATION OF 19.6 FT NATIONAL GEODETIC VERTICAL DATUM OF 1928 (NGVD29) BY A FLOATING PUMP STATION. THE MAXIMUM SURCHARGE POOL WITHIN THE SLURRY POND WAS COMPUTED BASED ON THE INFLOW DESIGN FLOOD (IDF) AS 35.3 FT NGVD29.
3. PZ-201 AND PZ-202 WERE INSTALLED BY PAUL C. RIZZO AND ASSOCIATES (PCRA) IN 1999 AND HAVE SINCE BEEN ABANDONED.

0 300' 600'

SCALE IN FEET

WINYAH GENERATING STATION
CROSS SECTION LOCATION MAP

Geosyntec consultants

FIGURE 1

PROJECT NO: GSC5242 | SEPTEMBER 2016

M:\S\SANTEE COOPER\SANTEE COOPER-WINYAH\0028-WINYAH H&H ANALYSES\FIGURES\F-0-SC-585-00-F0028-038

Written by: J. McNash Date: 10/15/2016 Reviewed by: A. Haskell /M. Zhu/G. Rix Date: 10/15/2016

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

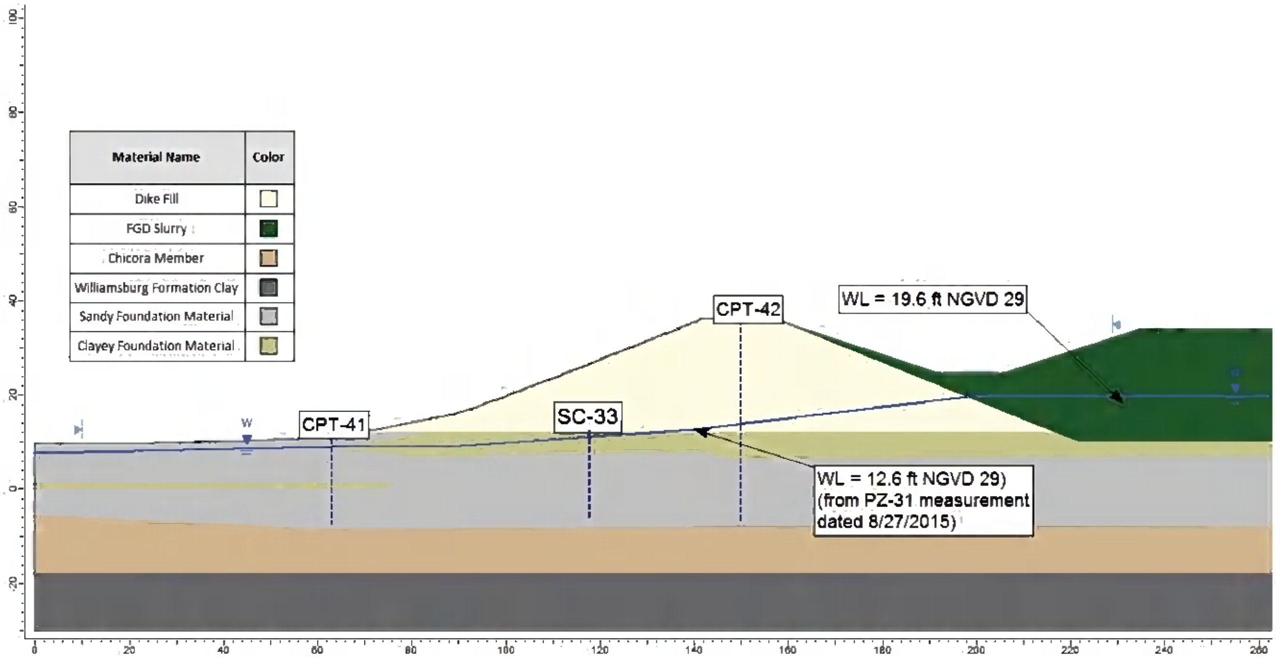


Figure 2. Cross Section A Geometry during Maximum Normal Storage Pool (as implemented within SLIDE®)

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

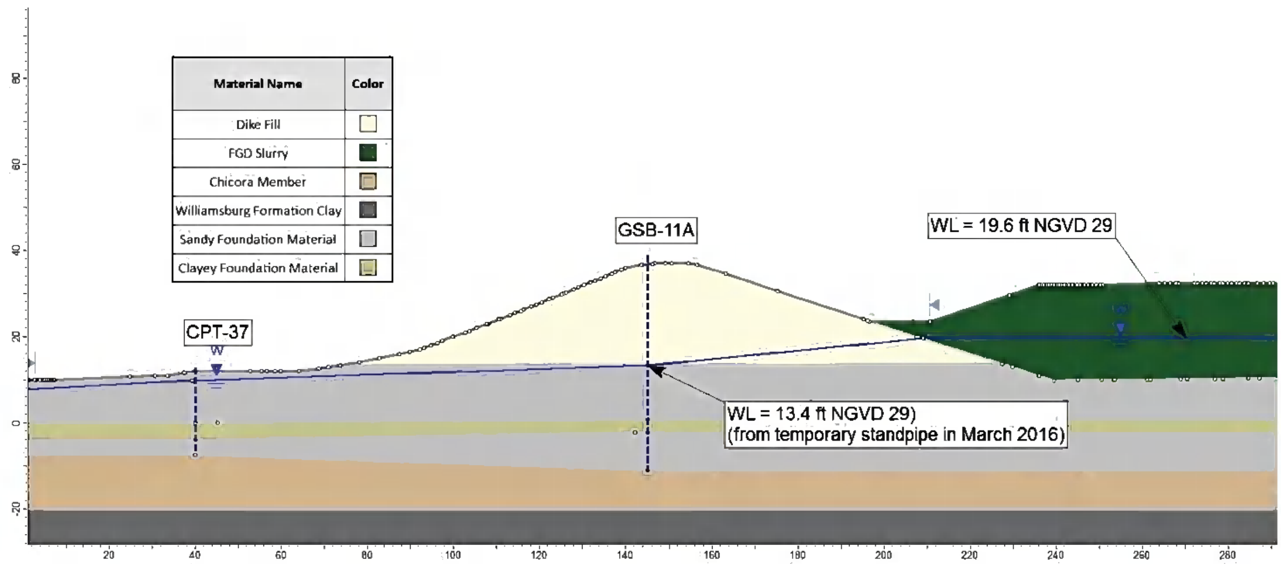


Figure 3. Cross Section B Geometry during Maximum Normal Storage Pool (as implemented within SLIDE®)

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

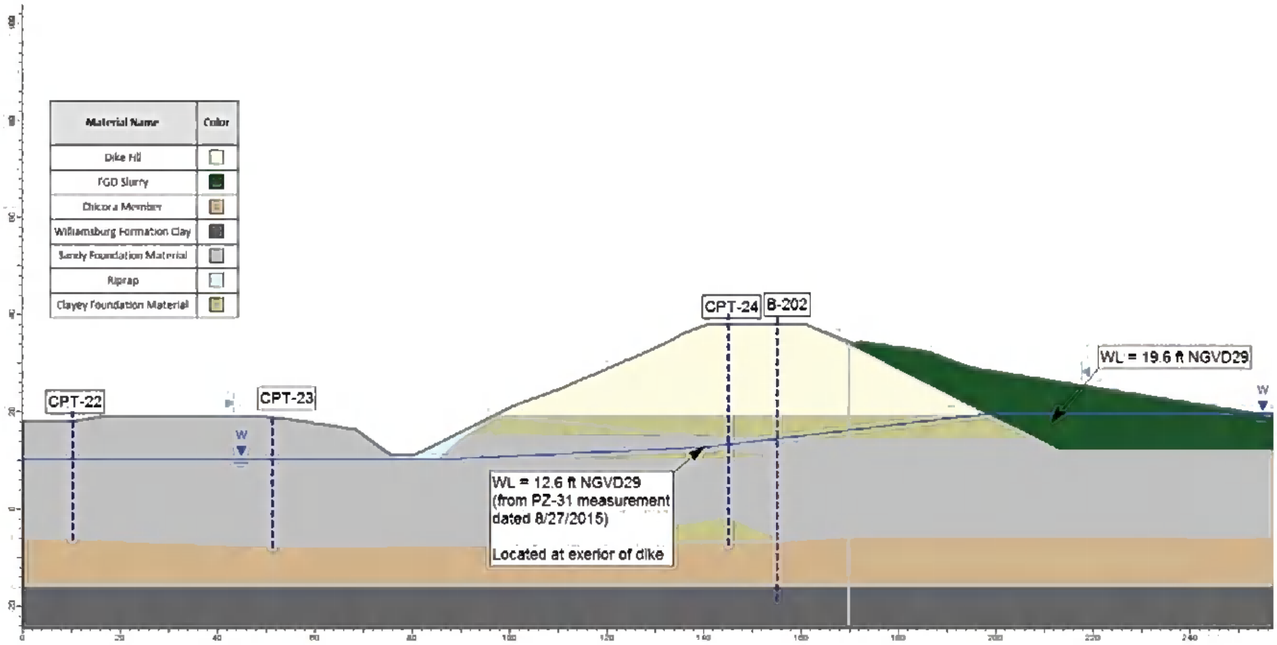


Figure 4. Cross Section C Geometry during Maximum Normal Storage Pool (as implemented within SLIDE®)

Note:

1. A 2H:1V rip-rap buttress is constructed within the stormwater channel at the base on the perimeter dikes within this area. The base of the stormwater channel is also lined with 1-ft of riprap.

Written by:	J. McNash	Date:	10/15/2016	Reviewed by:	A. Haskell /M. Zhu/G. Rix	Date:	10/15/2016

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

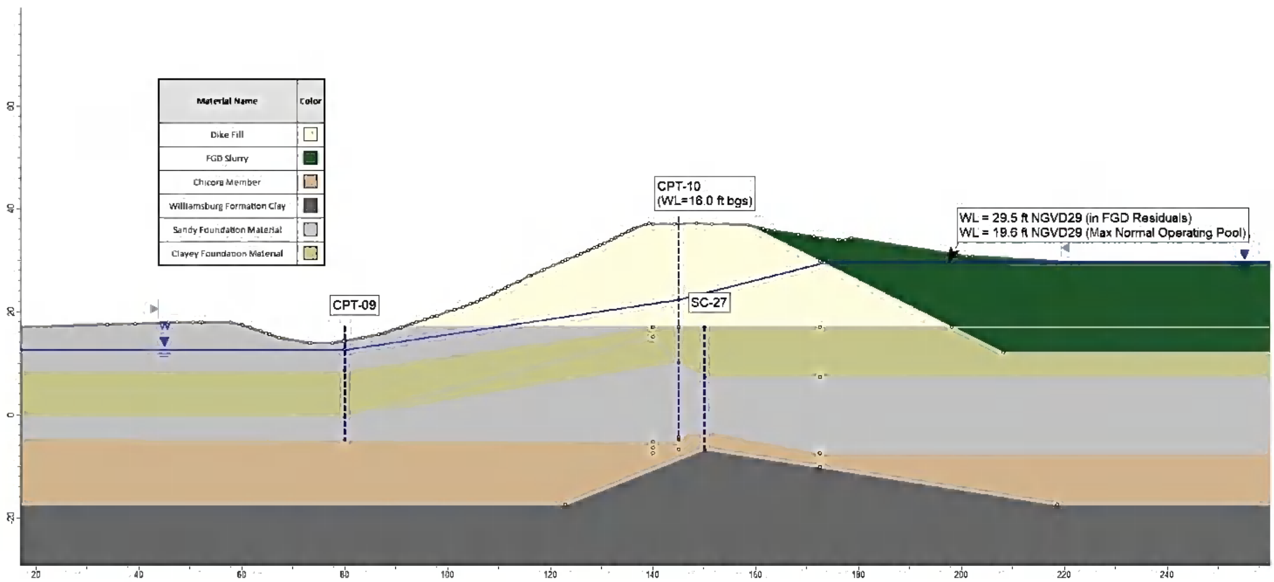


Figure 5. Cross Section D Geometry during Maximum Normal Storage Pool (as implemented within SLIDE®)

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

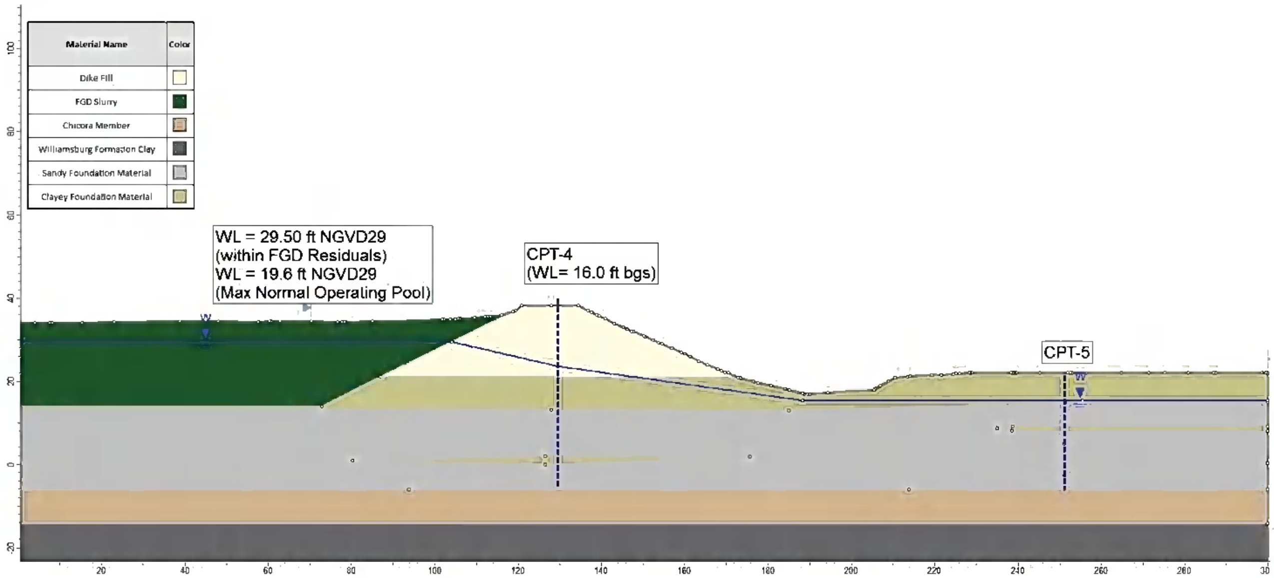


Figure 6. Cross Section E Geometry during Maximum Normal Storage Pool (as implemented within SLIDE®)

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

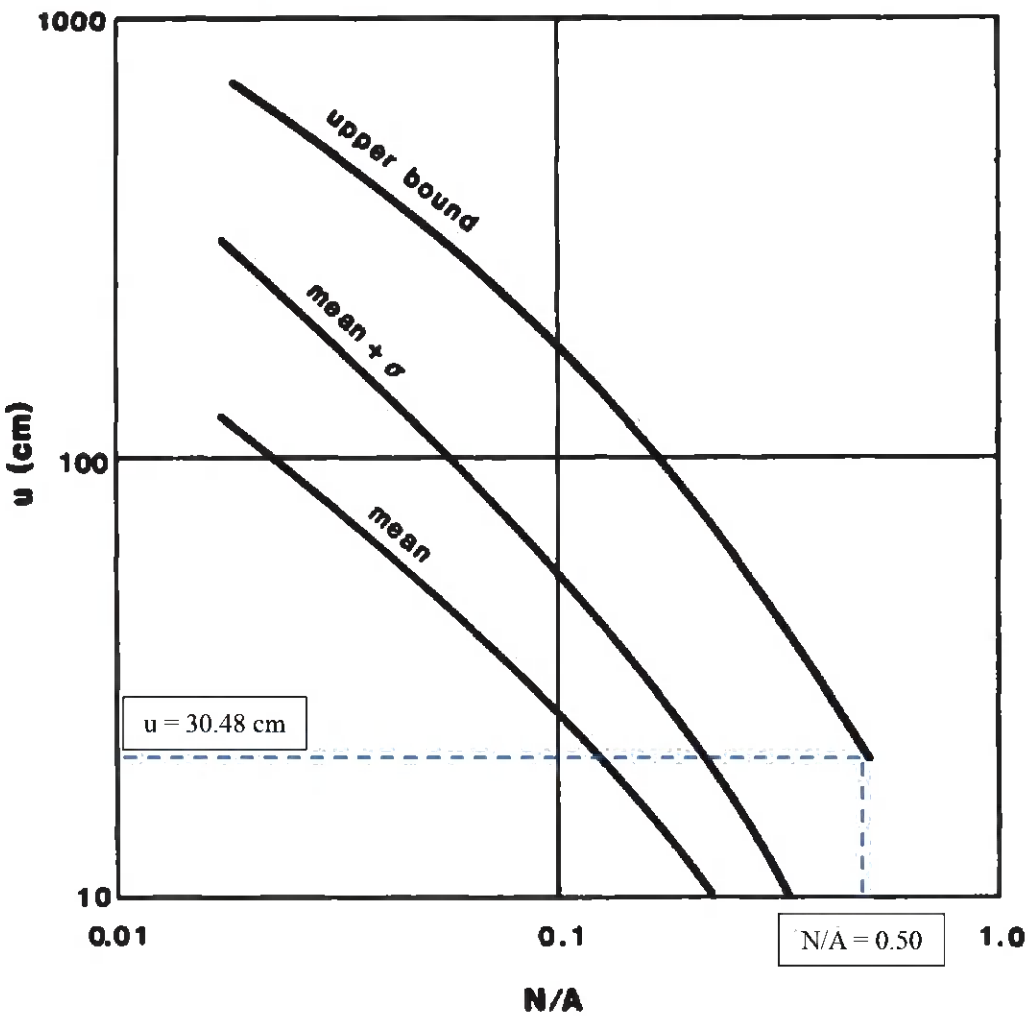


Figure 7. Allowable Displacement (u) vs. N/A (from Figure 7 of Hynes-Griffin and Franklin, 1984)

Notes:

1. An allowable deformation (u) of 12 inches (30.48 cm) and the "Upper Bound" curve were selected during these analyses.
2. A ratio of N/A of 0.50 was selected assuming 12 inches of displacement.

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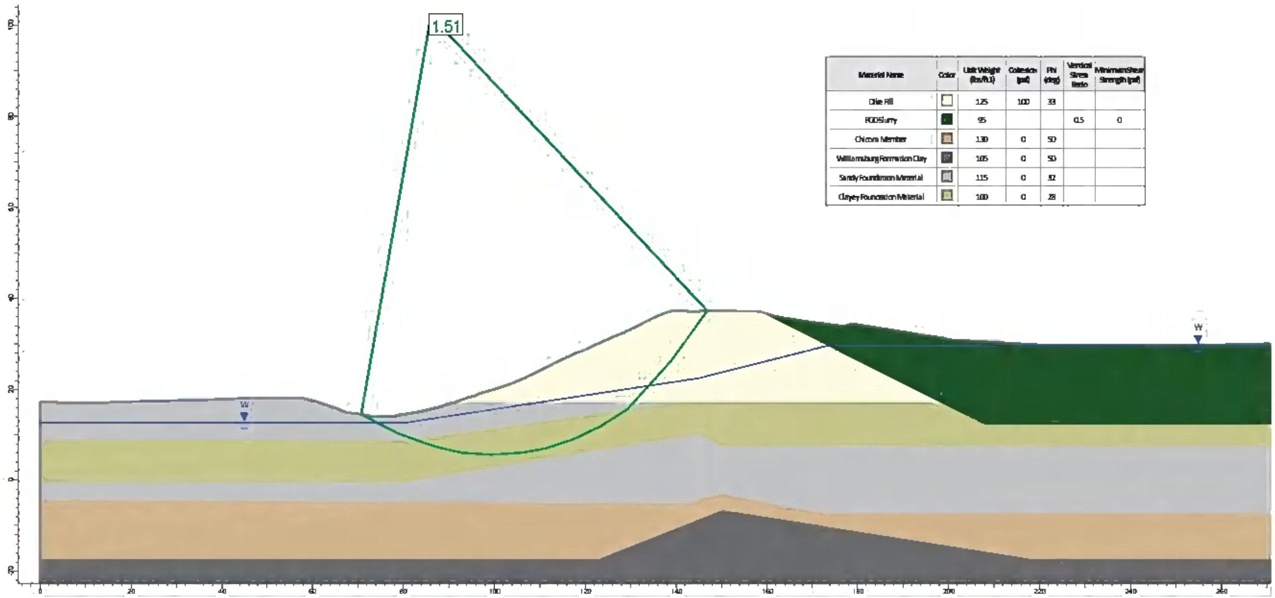

Figure 8. Calculated Critical Factor of Safety for Cross Section D
(Static Factor of Safety - Maximum Normal Storage Pool)

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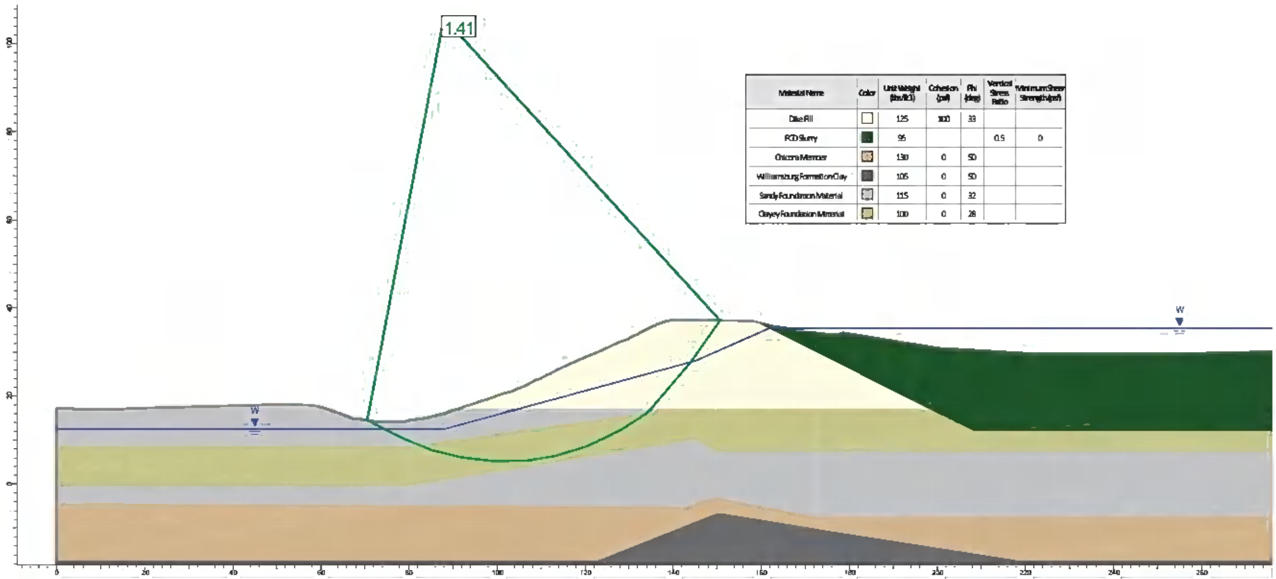

Figure 9. Calculated Critical Factor of Safety for Cross Section D
(Static Factor of Safety - Maximum Surcharge Pool)

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

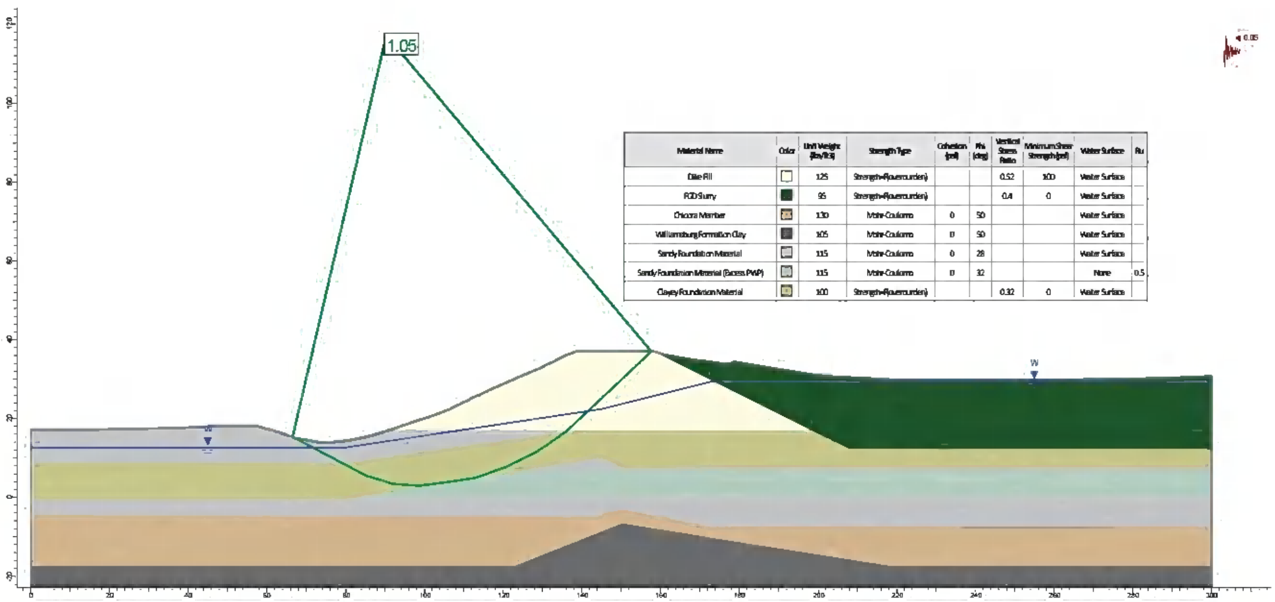

Figure 10. Calculated Critical Factor of Safety for Cross Section D
(Seismic Factor of Safety – Maximum Normal Storage Pool)

Notes:

1. The undrained shear strength ratio was reduced by 20% to account for the influence of cyclic degradation.
2. The sandy foundation material that was observed to contain a factor of safety against liquefaction (FS_{LIQ}) below 1.4 was modeled with excess pore pressures (R_u = 0.5) from guidance provided in Marcuson et al (1990). For Cross Section D, this zone was estimated between elevations 0 and 7 ft NGVD29 based on calculations for CPT-13 (See details of the liquefaction analysis in Attachment 7).

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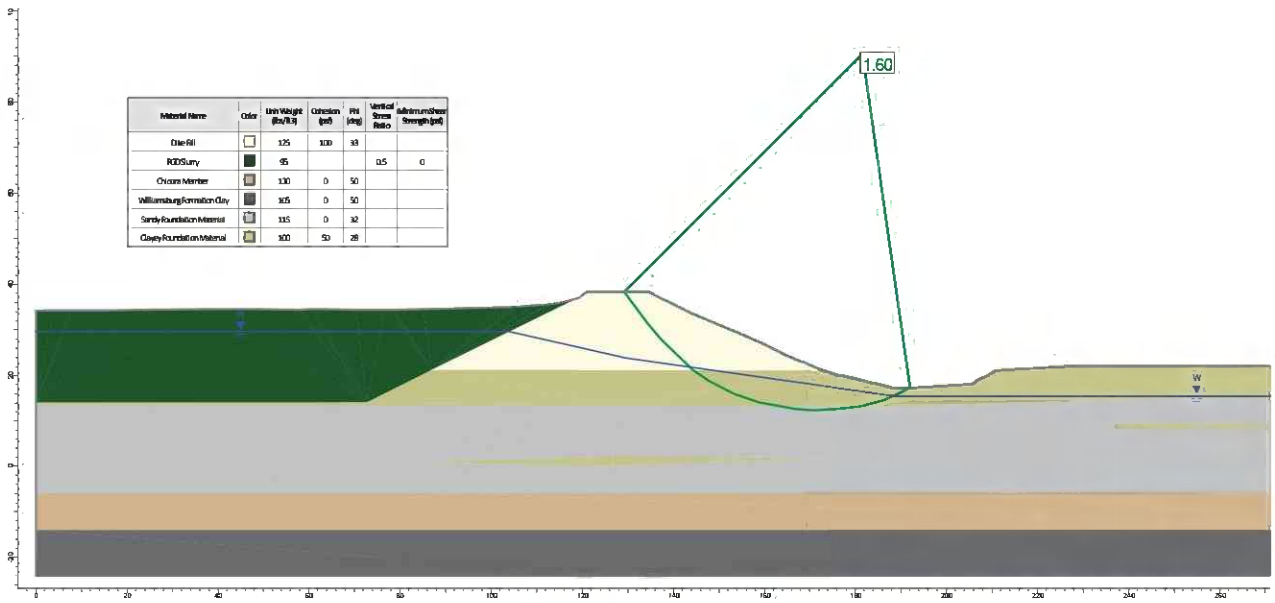

Figure 11. Calculated Critical Factor of Safety for Cross Section E
(Static Factor of Safety - Maximum Normal Storage Pool)

Written by: **J. McNash** Date: **10/15/2016** Reviewed by: **A. Haskell /M. Zhu/G. Rix** Date: **10/15/2016**

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

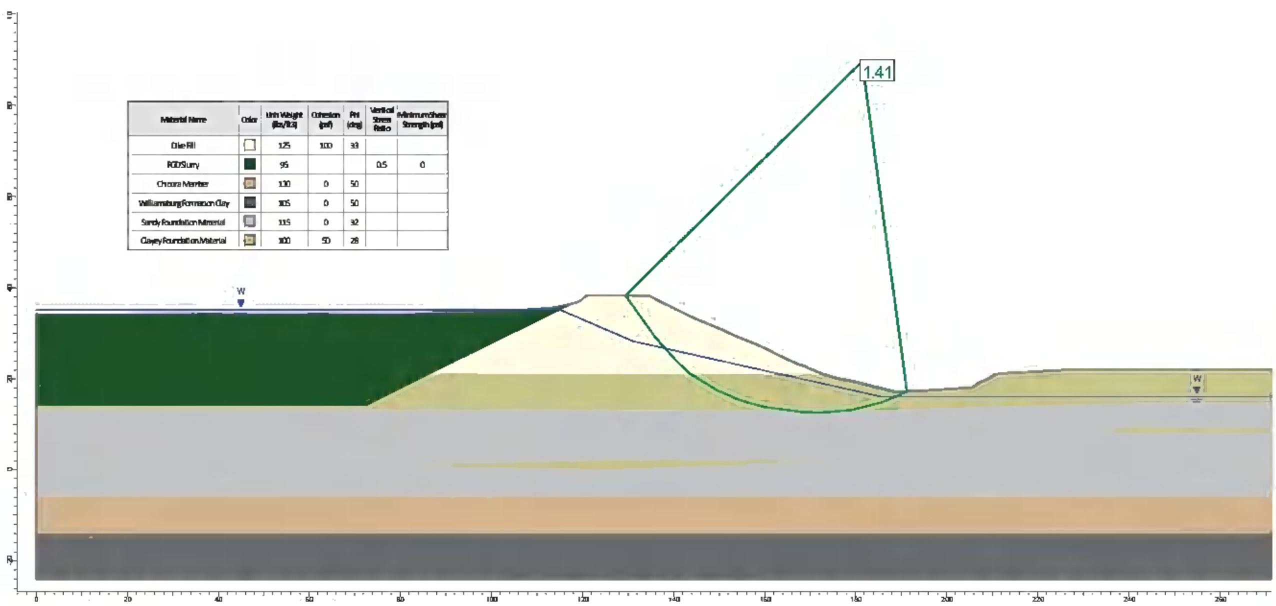

Figure 12. Calculated Critical Factor of Safety for Cross Section E (Static Factor of Safety - Maximum Surcharge Pool)

Notes:

1. The phreatic surface through the Cross Section E under the Maximum Surcharge Pool conditions was assumed to be approximately 10 ft below the dike crest, which is consistent with the 24-hr water level measurement from a hollow stem auger boring nearby (HSA-1). During this boring, the water level within the Slurry Pond had been maintained at an elevation of 34.25 ft NGVD29 for several years and was considered similar to the steady state conditions during maximum surcharge pool conditions (35.4 ft NGVD29).
2. A cohesion intercept of 50 psf was applied to the shallow "clayey foundation soils" for this analysis case. Without the cohesion intercept, a localized slip surface with a FS of 1.27 would be calculated, which was identified as an artifact of conservatively modeling the clayey foundation soil as a purely frictional material in this area. The assumed cohesion intercept of 50 psf was considered appropriate for the clayey materials.

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

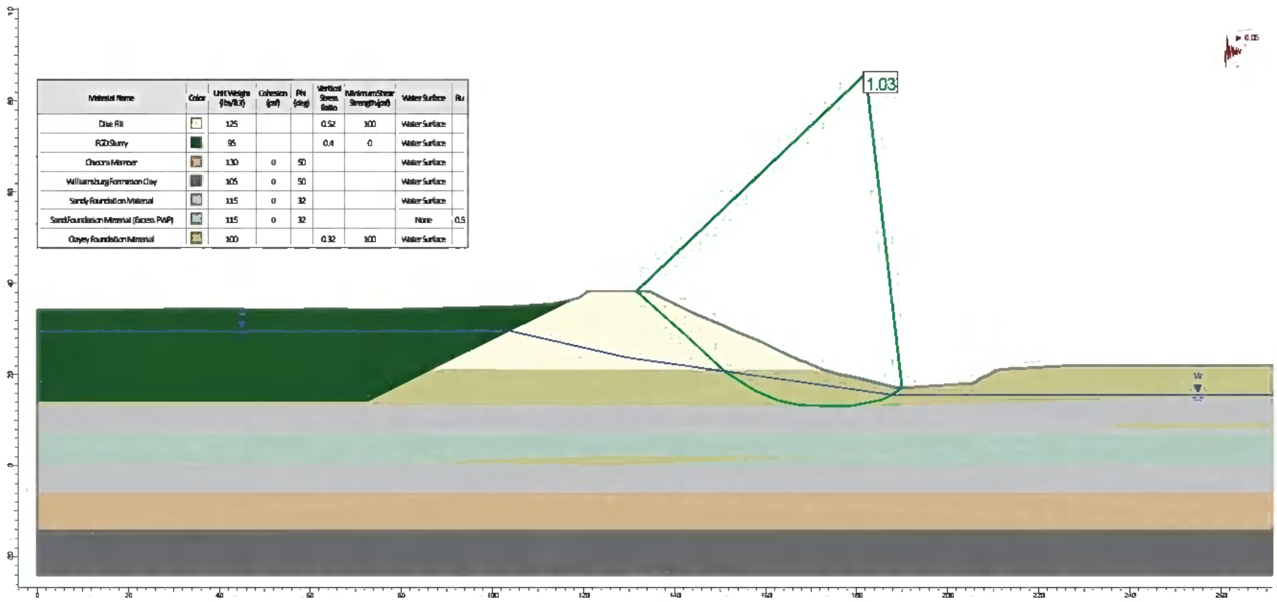

Figure 13. Calculated Critical Factor of Safety for Cross Section E (Seismic Factor of Safety – Maximum Normal Storage Pool)

Notes:

1. The undrained shear strength ratio was reduced by 20% to account for the influence of cyclic degradation.
2. The sandy foundation material that was observed to contain a factor of safety against liquefaction (FS_{LIQ}) below 1.4 was modeled with excess pore pressures (R_u = 0.5) from guidance provided in Marcuson et al (1990). For Cross Section E, this zone was estimated between elevations 0 and 7 ft NGVD29 based on calculations for CPT-10 (See details of the liquefaction analysis in Attachment 7).