Report of Geotechnical Exploration
Big Dam Swamp Fire Station No. 15
Andrews, South Carolina
S&ME Project No. 1463-16-019

Prepared for:

Georgetown County
129 Screven Street, Suite 239
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Prepared by:

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May 20, 2016

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Georgetown County
129 Screven Street, Suite 239
Georgetown, South Carolina 29440

Attention: Mr. Kyle Prufer

Reference: **Report of Geotechnical Exploration**
Big Dam Swamp Fire Station No. 15
Andrews, South Carolina
S&ME Project No. 1463-16-019

Dear Mr. Prufer:

S&ME, Inc. has completed the subsurface exploration for the referenced project after contract execution on April 21, 2016. Our exploration was conducted in general accordance with our Proposal No. 14-1600320, dated April 20, 2016.

The purpose of this study was to characterize the surface and subsurface soils on the proposed site, and to provide recommendations for site preparation and earthwork, foundation types and seismic design values, on-site soil suitability, pavement subgrade preparation, and pavement section thickness. This report presents the findings of our exploration along with our conclusions and recommendations.

S&ME, Inc. appreciates this opportunity to be of service to you. Please contact us if you have questions concerning this report or any of our services.

Sincerely,

S&ME, Inc.

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION S&ME, INC. No. C00478

Chelsea Jones
Staff Professional

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 21248 RON FOREST JR. 5/20/16

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Senior Engineer

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❖ Executive Summary

For your convenience, this report is summarized in outline form below. This brief summary should not be used for design or construction purposes without reviewing the more detailed information presented in the remainder of this report.

1. **Soil Conditions:** Topsoil and plowzone was observed in all borings and was about 1 to 1.5 feet thick. Beneath the topsoil, the soil profile consists of a silty to clayey sand layer (Stratum I) to depth of about 10 to 12 feet. Underlying the silty/clayey sands, a layer of dense sands (Stratum II) was encountered to a depth of about 17.5 to 19 feet. Beneath this layer, an interbedded silty sand, silt, and clay layer (Stratum III) was encountered to the sounding termination depths of 25 and 29.5 feet. Test sounding SCPT-1 encountered refusal to further advancement at 29.5 feet.
2. **Subsurface Water:** Water was not encountered within the hand auger borings which were advanced to a depth of 3 to 4 feet. Water levels within the cone soundings were interpreted from pore pressure readings to be approximately 3.2 to 8.6 feet below the existing ground surface. The infiltration rate in the proposed pond area was measured to be about 5.7 inches per hour.
3. **Liquefaction and Seismic Hazards:** Liquefaction of sands during the code design earthquake does not appear to be a significant risk at this site based on the soil conditions observed.
4. **Seismic Site Class:** Based on the average shear wave velocities estimated at this site of 763 feet per second, Seismic Site Class D parameters appear to be appropriate for design of the new facility. The following seismic design parameters apply: S_{DS} = 0.71g, S_{D1} = 0.37g, and Peak Ground Acceleration (PGA_M) = 0.58g. For a structure having a seismic use group classification of IV, the S_{DS} and S_{D1} values obtained are consistent with Seismic Design Category D as defined in section 1613.5 of the IBC, 2012 edition.
5. **Shallow Foundations:** Shallow foundations may be used to support the building assuming that the structure can be designed to tolerate the predicted static settlements associated with the building loads.

   - Considering the provided structural loads, we recommend an allowable bearing capacity of 2,500 psf for design of isolated shallow spread footings. The estimated total static settlement under the assumed loads is approximately ¾ inch or less.
   - Careful evaluation of the bearing conditions within the open footing excavations will be important during construction due to the loose condition of the upper 3 feet of the silty sand and clayey sand soils. In order to provide suitable bearing conditions in some portions of the building foundations, undercutting and replacement of the bearing soils with clean, washed gravel within the upper few feet beneath bearing grade elevation may become necessary.

6. **Pavements:** Flexible (asphalt) pavements are not recommended for use in areas that will be traveled by the fire trucks. Only rigid Portland cement concrete pavements should be used in those areas. Flexible pavements may be used in the employee parking lot.

   - For light-duty flexible pavements, we recommend a minimum pavement section consisting of 2 inches of SCDOT Type C surface course hot mixed asphalt (HMA) over at least 6 inches of compacted graded aggregate base course (GABC).
   - For heavy-duty rigid (concrete) pavement areas, we recommend a 4,000 psi compressive strength Portland cement concrete thickness of at least 7 inches with steel reinforcement

(such as dowel baskets) at the load transfer joints, overlying a compacted graded aggregate base course thickness of 6 inches, overlying a drainage layer consisting of at least 6 inches of open-graded, manufactured granitic gravel meeting the gradation of SCDOT No. 57 or No. 67 stone. Non-woven geotextile filter fabric (Mirafi 140N) is recommended to be placed between the GABC layer and the drainage layer, and woven geotextile (Mirafi HP-370) is recommended to be placed between the drainage layer and the subgrade.

- We have been involved in several fire station pavement repair projects over the years, and most of the pavement deterioration has been attributed to poor subsurface drainage. The gravel drainage layer approach has been implemented in these pavement repair projects with success. While adding a nominal initial cost, the long term savings of using this approach are expected to be quite significant.

1.0 Introduction

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning earthwork, foundations, pavements, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations.

A site plan showing the approximate exploration location is included in Appendix I. The sounding logs, hand auger logs, infiltration test results, discussion of the field exploration procedures, and legends of soil classification and symbols are included in Appendix II. Appendix III contains the results of the laboratory testing and our laboratory test procedures.

1.1 Site and Project Description

Project information was provided in an email from Mr. Kyle Prufer (County of Georgetown) to Tommy Still (S&ME, Inc.) on April 14, 2016. The email contained a Plat, showing a division of parcel 4 prepared by Parker Land Surveying, LLC and dated January 6, 2016. Additional information was provided in a telephone conversation between Mr. Michael Walker of Tych & Walker Architects and Ron Forest Jr., of S&ME on April 18, 2016. During this call, Mr. Walker informed us that this new fire station would likely consist of a two-bay truck garage, metal-framed, single-story building with slab on grade construction, and would likely have an attached office space. A preliminary site layout sketch was provided and was used to locate our test soundings (see also Figure 2 in Appendix I).

1.2 Site Description

The site is located off of County Line Road/Highway 41 and Bid Dam Swamp Drive in Andrews, South Carolina. A site vicinity map is attached in Appendix 1 as Figure 1. The lot currently exists as a vacant lot with grown up straw grass.

1.3 Project Description

The proposed building consists of a single-story fire station on a lot that is approximately 6.36 acres with the development located on the 2.7 acre eastern parcel. Detailed project drawings were not provided to us; we anticipate that the structure will be supported on shallow foundations and may include cold-formed metal framing or masonry walls and a soil-supported slab on grade. We were not provided with any structural load information. We have assumed based on our previous experience with similar projects that column and wall loads will not exceed 75 kips and 5 kips per linear foot, respectively. It is also anticipated that no more than 1 to 2 feet of new fill placement will be required during site grading to achieve design grade elevations.

If more than 2 feet of new fill will be required to achieve design grade elevations, it is important that you contact us because additional fill height increases the potential for settlement of the subgrade under the weight of the fill embankment, which in turn may affect the estimated total and differential settlement magnitudes of the building.

2.0 Exploration Procedures

2.1 Field Exploration

Between April 27, 2016 and May 9, 2016, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

- We performed a site walkover, observing features of topography, existing structures, ground cover, and surface soils at the project site.
- We established one seismic cone penetration test (SCPT) sounding location and two cone penetrometer test (CPT) sounding locations. We also established locations for three hand auger boring and dynamic cone penetrometer (DCP) test locations at the site within the proposed parking and driving area. A test location sketch is attached in Appendix I as Figure 2.
- We advanced one SCPT sounding (SCPT-1) within the approximate future building footprint to a depth of 29.5 feet, one CPT sounding (CPT-2) within the approximate future building footprint to a depth of 25 feet, and one CPT sounding (CPT-3) to a depth of 10 feet in the proposed retention pond. SCPT-1 was originally assigned to be advanced to a depth of 40 feet, but encountered refusal to further advancement of the drilling tools at a depth of 29.5 feet.
- Within the SCPT sounding, downhole shear wave velocity measurements were obtained at approximate 1 meter depth intervals until the sounding was terminated. In the SCPT/CPT soundings, an electronically instrumented cone penetrometer was hydraulically pushed through the soil to measure tip point stress, pore water pressure, and sleeve friction. The data was then used to determine soil stratigraphy and to estimate soil parameters such as preconsolidation stress, friction angle, and undrained shear strength.
- We advanced a hand auger boring without penetration testing at each of the sounding locations (SCPT-1, CPT-2 and CPT-3) within the future building footprint and retention pond to a depth of 4 feet each.
- We advanced three hand auger borings (HA-1 through HA-3) within the future pavement areas to a depth of 4 feet each. In conjunction with these hand auger borings, DCP testing was performed at approximate one-foot intervals in each boring in general accordance with ASTM STP 399, "Dynamic Cone for Shallow In-Situ Penetration Testing" to provide us with an index for estimating soil strength parameters and relative consistency of the near-surface soils encountered.
- The subsurface water level at test location was measured in the field at the time of our field work, or interpreted from CPT pore pressure readings.

A brief description of the field exploration procedures performed, as well as the sounding and hand auger boring logs, is attached in Appendix II.

2.2 Laboratory Testing

After the recovered soil samples were brought to our laboratory, a geotechnical professional examined and/or tested each sample to estimate its distribution of grain sizes, plasticity, moisture condition, color, presence of lenses and seams, and apparent geologic origin in general accordance with ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)".

The resulting classifications are presented on the hand auger boring logs, included in Appendix II. Similar soils were grouped into representative strata on the logs. The strata contact lines represent approximate boundaries between soil types. The actual transitions between soil types in the field are likely more gradual in both the vertical and horizontal directions than those which are indicated on the logs.

We performed the following quantitative ASTM-standardized laboratory tests to help classify the soils and formulate our conclusions and recommendations. The laboratory tests performed included the following:

- One bulk sample tested in general accordance with ASTM D 2216, "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass", to measure the in situ moisture content of the soil.
- One bulk sample tested in general accordance with ASTM D 422, "Standard Test Method for Particle Size Analysis of Soils," without hydrometer, to measure the distribution of particle sizes greater than 75 µm.
- One bulk sample tested in general accordance with ASTM D 1140, "Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve", to measure the percent clay and silt fraction.
- One bulk sample tested in general accordance with ASTM D 4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils", to measure the plasticity of the soil.
- One bulk sample tested in general accordance with ASTM D 1557, "Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 lbf/ft^3)", to measure the moisture-density relationship of the soil.
- One bulk sample recompacted and tested in general accordance with ASTM D 1883, "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils", to evaluate soil support characteristics for pavements.

The laboratory test results and procedures for the above listed tests are attached to this report in Appendix III.

3.0 Site and Surface Conditions

This section of the report describes the general site and surface conditions observed at the time of our exploration.

3.1 Topography

We observed that the proposed construction area appears to be relatively level, but is lower in elevation than the surrounding roadways, indicating that some fill (perhaps up to 1 to 2 feet) is likely to be needed to reach design subgrade elevations for construction. Ground surface elevations were not directly surveyed by S&ME; therefore, for the purpose of our boring logs and to illustrate our subsurface cross-sectional soil profile (Figure 3 and in Appendix I), the ground surface level was set to zero. Topsoil was encountered at all borings and was about 1 foot to 1.5 feet thick. The lower portion of the topsoil may reflect plowzone materials from previous agricultural activity.

3.2 Local Geology

The site lies within the Coastal Terraces Region of the Lower Coastal Plain of South Carolina. The topography of this region is dominated by a series of archaic beach terraces, exposed by uplifting of the local area over the last one million years. The lower coastal plain terraces are relatively young Quaternary age features, exhibit only minor surface erosion, and can be traced large distances on the basis of surface elevation. Each terrace forms a thin veneer over older, consolidated marine shelf or terrestrial Coastal Plain residual soils that are Cretaceous to Tertiary in age.

4.0 Subsurface Conditions

The generalized subsurface conditions at the site are described below. For more detailed descriptions and stratifications at test locations, the respective sounding and hand auger boring logs should be reviewed in Appendix II.

4.1 Description of Subsurface Soils

This section describes subsurface soil conditions observed at the site.

4.1.1 Stratum I: Very Stiff Sandy Clays and Clayey Sands

Underlying the topsoil, an upper layer of clayey soils consisting of poorly graded sand with clay (USCS Classification "SP-SC"), clayey sand (SC), silty clay (CL-ML), and sandy lean clay (CL) were encountered to a depth of approximately 10 to 12 feet. Sounding CPT-3 was terminated within this stratum at a depth of 10 feet. Within the CPT soundings, the soils of Stratum I exhibited tip stresses ranging from 25 tons per square foot (tsf) to about 160 tsf but were typically in the range of 30 tsf to 60 tsf, indicating typically very stiff conditions. The sleeve stresses in these soils ranged from near 0.75 tsf to 4 tsf, typical of very stiff soils. The upper few feet of this stratum was less dense, as evidenced by DCP penetration resistances of 4 to 11 blows per increment (bpi) in hand auger borings HA-1 through HA-3.

A composite bulk sample was collected from the upper portion of Stratum I and subjected to natural moisture content, grain size distribution, and plasticity testing. The soil was collected from the proposed pond at approximately 1 to 2 feet below grade near sounding CPT-3, and was classified as silty sand (SM) with a fines content of 20.2% passing the No. 200 sieve. This sample exhibited non-plastic behavior and was brown in color. The natural moisture content was measured to be 1.6%. The modified Proctor maximum dry density was 109.6 pounds per cubic foot (pcf) at an optimum moisture content of 9.2 percent. The CBR value was measured to be 19.0 percent at 95 percent compaction (ASTM D 1557).

4.1.2 Stratum II: Dense Sands

Underlying the upper sandy clays and clayey sands and beginning at a depth of about 10 to 12 feet, a stratum of dense sands (Stratum II) was encountered to a depth of about 17.5 to 19 feet. Soils of this stratum were interpreted to consist of poorly-graded sand (SP). The soils of this stratum exhibited tip stresses typically ranging from about 100 tsf to about 180 tsf, indicating a medium dense to dense relative density. The sleeve stresses in these soils ranged from near 0.15 tsf to 0.4 tsf, consistent with dense sands.

4.1.3 Stratum III: Loose Silty Marine Sand with Silt and Clay Seams

Beneath the dense sands of Stratum II, a stratum of sand with silt and clay seams (Stratum III) was encountered to the termination depth of our deepest soundings at 25 and 29.5 feet. Soils of this stratum were interpreted to consist of silty sand (SM), silty clay (CL-ML), clay (CL) and poorly graded sand (SP). The soils of this stratum exhibited tip stresses ranging from about 10 tsf to 50 tsf, indicating a firm to stiff consistency. The sand seams in SCPT-1 exhibited tip stresses ranging from 75 tsf to 140 tsf, indicating a medium dense to dense relative density. The sleeve stresses in these soils ranged from about 0 tsf to 1.0 tsf, consistent with stiff clays.

4.2 Subsurface Water

Water was not encountered within the hand auger borings which were advanced to a depth of 3 to 4 feet. Water levels within the cone soundings were interpreted from pore pressure readings to be approximately 3.2 to 8.3 feet below the existing ground surface. Subsurface water levels may fluctuate seasonally at the site, being influenced by rainfall variations and other factors. Because of the presence of near-surface clayey sands, this site is susceptible to the buildup of perched ground water in shallow, near-surface lenses. Perched water is typically a seasonal issue, and may create greater difficulties during earthwork if the grading is performed during the wetter times of the year.

4.3 Measured Infiltration Rates

We conducted one double-ring infiltrometer test at a depth of about 12 inches below the existing ground surface at test location I-1, near sounding CPT-3 (see the attached Figure 2 for a test location sketch). The testing was conducted in general accordance with American Society for Testing and Materials (ASTM) procedure D-3385 entitled, "Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer".

The Double-Ring Infiltrometer consists of two concentric rings and a driving plate. The outer ring is 24 inches in diameter, and the inner ring is 12 inches in diameter. The two rings are driven into the ground and partially filled with water. The double ring design helps prevent divergent flow in layered soils. The outer ring acts as a barrier to encourage only vertical flow from the inner ring.

The water level is maintained for a specific period of time, depending on the type of soil and permeability level. The volume of water needed to maintain a specified level and the time factors are recorded and converted into a specific infiltration rate.

The stabilized (saturated) infiltration rate measured at our test location was 5.7 inches per hour. A summary of the field test results is presented in Appendix II.

When choosing the value for infiltration rate that is ultimately used in design, the designer needs to consider the variability of the soils with lateral extent and with depth, and understand that a slight change in the clay or silt fines content or water table elevation could have a significant impact upon the infiltration rate. As fines content increases, infiltration rate is likely to decrease. Also, it is important to recognize that soils that are saturated or located below the ground water level will accept no flow.

5.0 Seismic Site Class and Design Parameters

Seismic-induced ground shaking at the foundation is the effect taken into account by seismic-resistant design provisions of the International Building Code (IBC). Other effects, including landslides and soil liquefaction, must also be considered.

5.1 IBC Site Class Determination

As of July 1, 2013, the 2012 edition of the International Building Code (IBC) has been adopted for use in South Carolina. We classified the site as one of the Site Classes listed in IBC Section 1613.3, using the procedures described in Chapter 20 of ASCE 7-10.

We understand that the 2015 edition of the IBC becomes effective in South Carolina for projects permitted after July 1, 2016. (Source: http://www.llr.state.sc.us/POL/BCC/); therefore, it is important to note that if this project is not permitted for construction prior to July 1, 2016, a seismic update to this report may become necessary.

The initial step in site class definition is a check for the four conditions described for Site Class F, which would require a site-specific evaluation to determine site coefficients F_A and F_V. Soils vulnerable to potential failure under item 1) including quick and highly sensitive clays or collapsible weakly cemented soils were not observed in the soundings. Three other conditions, 2) peats and highly organic clays; 3) very high plasticity clays (H>25 feet); and 4) very thick soft/medium stiff clays were also not evident in the soundings performed.

One other determining characteristic, liquefaction potential under seismic conditions, was assessed. Soils were assessed qualitatively for liquefaction susceptibility based on their age, stratum, mode of deposition, degree of cementation, and size composition. This assessment considered observed liquefaction behavior in various soils in areas of previous seismic activity.

Our analysis, which is more fully described below, indicates that significant liquefaction of subsoils appears unlikely to occur at this site in the event of the design magnitude earthquake due primarily to the clayey and cohesive characteristics of the majority of the soil profile. Any liquefaction that occurs in the sandy seams is expected to be limited to discontinuous pockets and isolated lenses, due to the relatively high density of the sands.

5.1.1 Liquefaction Analysis

Liquefaction of saturated, loose, cohesionless soils occurs when they are subject to earthquake loading that causes the pore pressures to increase, and effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results.

We performed a liquefaction analysis based on the design earthquake prescribed by the 2012 edition of the International Building Code (IBC 2012), the "simplified procedure" as presented in Youd et al. (2001), and recent research concerning the liquefaction resistance of aged sands (Hayati & Andrus, 2008; Andrus et al. 2009; Hayati & Andrus, 2009). Our analysis was based upon a peak ground surface acceleration of 0.58g.

To help evaluate the consequences of liquefaction, we also computed the Liquefaction Potential Index (LPI), which is an empirical tool used to evaluate the potential for liquefaction to cause damage. The LPI considers the factor of safety against liquefaction, the depth to the liquefiable soils, and the thickness of the liquefiable soils to compute an index that ranges from 0 to 100. An LPI of 0 means there is no risk of liquefaction; an LPI of 100 means the entire profile is expected to liquefy. The level of risk is generally defined below.

- **LPI < 5** – surface manifestation and liquefaction-induced damage not expected.
- **5 ≤ PI ≤ 15** – moderate liquefaction with some surface manifestation possible.
- **LPI > 15** – severe liquefaction and foundation damage is likely.

The average LPI for this site was estimated to be less than 5, which indicates that the risk of surface damage due to liquefaction is low. However, due to the presence of water, and the pockets of loose sands that were intermittently observed at different depths and within different thickness zones within the test soundings, it is possible that soil liquefaction may occur in discontinuous pockets and isolated lenses during seismic shaking associated with the design level earthquake. Our analysis shows that, in the event that this occurred, the anticipated settlements associated with the liquefaction are expected to be relatively small in magnitude.

Based on the test sounding data, we determined that site response factors F_A and F_V corresponding to Site Class D would be applicable to determine spectral values for design. This recommendation is provided based on the average weighted shear wave velocities measured to a depth of 29.5 feet and interpolated to a depth of 100 feet. The average weighted shear wave velocity was estimated to be 763 feet per second, which is greater than the minimum of 600 feet per second that is required for consideration of Site Class D design parameters.

5.2 Seismic Design Coefficients for Site Class D

Selection of the base shear values for structural design for earthquake loading is the responsibility of the structural engineer. However, for the purpose of evaluating seismic hazards at this site, S&ME has evaluated the spectral response parameters for the site using the general procedures outlined under the 2012 International Building Code Section 1613.3. This approach utilizes a mapped acceleration response spectrum reflecting a targeted risk of structural collapse equal to 1 percent in 50 years to determine the spectral response acceleration at the top of seismic bedrock for any period. The 2012 IBC seismic provisions of Section 1613 use the 2008 Seismic Hazard Maps published by the National Earthquake Hazard Reduction Program (NEHRP) to define the base rock motion spectra.

The Site Class is used in conjunction with mapped spectral accelerations S_S and S_1 to determine Site Amplification Coefficients F_A and F_V from tables 11.4-1 and 11.4-2 in section 11.4.7 of ASCE 7-10. For purposes of computation, the Code includes probabilistic mapped acceleration parameters at periods of 0.2 seconds (S_S) and 1.0 seconds (S_1), which are then used to derive the remainder of the response spectra at all other periods. The mapped S_S and S_1 values represent motion at the top of seismic bedrock, defined as the Site Class B-C boundary. The surface ground motion response spectrum, accounting for inertial effects within the soil column overlying rock, is then determined for the design earthquake using spectral coefficients F_A and F_V for the appropriate Site Class.

The design ground motion at any period is taken as 2/3 of the smoothed spectral acceleration as allowed in section 1613.3.4. The design spectral response acceleration values at short periods, S_{DS}, and at one second periods, S_{D1}, are tabulated below for the unimproved soil profile using the IBC 2012 criteria.

The 2012 IBC specifically references ASCE 7-10 for determination of peak ground acceleration value for computation of seismic hazard. Peak ground acceleration is separately mapped in ASCE 7-10 and corresponds to the geometric mean Maximum Credible Earthquake (MCE_G). The mapped PGA value is adjusted for site class effects to arrive at a design peak ground acceleration value, designated as PGA_M.

Table 5-1: Spectral Response Acceleration Parameters

Site Class D	
Parameter	**Design Value**
F_A	1.12
F_V	1.77
S_{MS}	1.06 g
S_{M1}	0.56 g
S_{DS}	0.71 g
S_{D1}	0.37 g
PGA_M	0.58

5.2.1 Seismic Design Category

For a structure having a Risk Category classification of IV the S_{DS} and S_{D1} values obtained are consistent with "Seismic Design Category D" as defined in section 1613.3.5 of the IBC.

6.0 Conclusions and Recommendations

The conclusions and recommendations included in this section are based on the project information outlined previously and the data obtained during our exploration. If the construction scope is altered, the proposed building location is changed, or if conditions are encountered during construction that differ from those encountered by the borings or soundings, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

6.1 Site Preparation

The following recommendations are provided regarding site preparation and earthwork:

1. While subsurface water was not observed within the upper 4 feet at the time of our exploration, and the near surface soils are sandy, it remains prudent to implement and maintain temporary drainage measures during construction to divert water away from the construction area. Surface and subsurface water conditions that occur during construction will determine the need for and

extent of drainage measures. The site civil engineer should be consulted regarding the locations and design detail of any permanent pavement underdrain systems that may be desired.

2. Strip surface vegetation and topsoil, where encountered, and dispose of outside the building footprint.
3. After the stripping operation is complete and site drainage has been established, the stripped surface in areas to receive fill should be densified with a heavy vibratory roller prior to placement of new fill. This is to help improve the density of the near-surface loose silty and clayey sands prior to fill placement. Moisture conditioning by the addition of water or drying of soils should be expected to be required prior to densification. The surface should be densified to at least 98 percent of the modified Proctor maximum dry density (ASTM D 1557) to a depth of at least 8 inches.
4. Following densification, the densified surface should be proofrolled under the observation of the geotechnical engineer (S&ME) by making repeated passes with a fully-loaded dump truck. The proofrolling should be conducted only during dry weather. Areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to any new fill placement or slab or pavement construction, as determined by the geotechnical engineer.
5. If any soft, clayey or silty soils are exposed by the stripping operation, they should also be proofrolled, but not until they have first been stabilized. Areas of rutting or pumping soils, as indicated by the proofroll, may require selective undercutting or further stabilization prior to fill placement, as determined by the geotechnical engineer. Stabilization may take the form of removal and replacement, plowing and drying, or other means as determined by the geotechnical engineer.
6. Pavement areas should also be proofrolled at final soil subgrade elevation under the observation of the geotechnical engineer (S&ME). If any areas of instability are observed during the proofroll, further stabilization should be performed, as determined by the geotechnical engineer.

6.2 Fill Placement and Compaction Recommendations

Where new fill soils are to be placed, the following recommendations apply:

1. Prior to fill placement, sample and test each proposed fill material to determine suitability for use, maximum dry density, optimum moisture content, and natural moisture content. It is recommended that the fill soils used to build up the pad for the structure and pavements meet the following minimum requirements: plasticity index of 6 percent or less; clay/silt fines content of not greater than 25 percent. Typically this would include USCS soil classifications SW, SP, SW-SC, SW-SM, SP-SC, and SP-SM, as well as some SM and SC soils. Based upon our laboratory test data, the soil we sampled from the proposed pond appears to meet these criteria to a depth of 4 feet, so the pond borrow may be of suitable soil type to re-use as structural backfill, but it should be considered that these soils may be wet if borrowed from at or below the water table..

2. Where fill soil is required, structural fill should be compacted throughout to **at least 98 percent** of the modified Proctor maximum dry density (ASTM D 1557). Compacted soils should not exhibit pumping or rutting under equipment traffic. Loose lifts of fill should be no more than 10 inches thick prior to compaction. Structural fill should extend at least 5 feet from the edge of structures and pavements before being allowed to exhibit a lower level of compaction.

3. Where present, the subsurface water level should be maintained at least 2 feet below any surface to be densified prior to beginning compaction. This is to reduce the risk of the compaction operations drawing water up to the surface and deteriorating it.
4. All fill placement should be witnessed by an experienced S&ME soils technician working under the guidance of the geotechnical engineer. In general, at least one field density test for every 2,500 square feet should be conducted for each lift of soil in large area fills, with a minimum of 2 tests per lift. At least one field density test should be conducted for each 150 cubic feet of fill placed in confined areas such as isolated undercuts and in trenches, with a minimum of 1 test per lift.

6.3 Shallow Foundation Recommendations

The following recommendations are provided for the design and construction of shallow foundations at this site for the proposed structure.

1. The proposed building may be supported on shallow foundations using isolated footings and slab-on-grade construction as planned. A net available bearing pressure of up to **2,500 psf** should be used for design of individual spread footings and wall footings that are extended to bear within residual coastal plain deposits or within structural fill compacted as recommended in the "Fill Placement and Compaction Recommendations" section 6.2 of this report.
2. It should be anticipated that where footings bear directly on fill, the previously placed fill soils exposed in the bottom of the footings may need to be tamped to increase their density prior to the placement of foundation concrete. Foundations which are extended to bear within loose silty or clayey sands may require the removal of up to several feet of the loose materials and replacement with open-graded, washed, clean gravel such as SCDOT No. 57 or No. 67 stone, or densification of the bearing surfaces after excavation and prior to footing construction. This should be a decision made at the time of construction based upon the results of DCP tests that are performed in the footing excavations by a representative of the geotechnical engineer (S&ME).
3. Even if smaller dimensions are theoretically allowable from a bearing pressure consideration, the minimum wall footing width should be at least 18 inches, and the minimum column footing width should be 24 inches, to avoid punching shear. Footings should be embedded to a minimum depth of at least 12 inches, or the depth specified on the drawings, whichever is greater.
4. Have the geotechnical engineer (S&ME) observe each cleaned footing excavation prior to concrete placement to measure that the required level of soil compaction and bearing capacity is present at the foundation bearing surface. Also, have the geotechnical engineer observe any undercut areas in footings prior to backfilling, in order to confirm that poor soils have been removed and that the exposed subgrade is suitable for support of footings or backfill.
5. For the purposes of settlement estimation, we assumed the structures will be constructed near existing grade elevations.

   A. Considering a 5 kip per linear foot wall load, a 250 psf floor slab and fill soil uniform area load, and a 2,500 psf spread footing bearing pressure, the estimated post construction static settlement of a typical wall strip footing will likely be on the order of ¾ inch or less.
   B. Considering a 75 kip column load, a 250 psf floor slab and fill soil uniform area load, and a 2,500 psf spread footing bearing pressure, the estimated post-construction static settlement of a typical column footing will likely be on the order of ¾ inch or less.

C. Differential settlements between individual walls and columns are typically on the order of 50 percent of the maximum total settlement value under static loading, or in this case, $^{3}/_{8}$ inch or less.

6.4 Grade Slab Support and Construction

The following recommendations are given for the support and construction of soil-supported grade slabs:

1. Soils similar to those penetrated by the borings are anticipated to provide adequate support to proposed soil-supported grade slabs, assuming preparation and compaction of the subgrade as recommended above. A modulus of subgrade reaction (k) of 200 lbs/in^3 (pci) is recommended for use for reinforcing design.
2. A vapor barrier such as "Visqueen," or equivalent, should be placed over the subgrade prior to placing concrete to limit moisture infiltration into finished spaces.
3. Place a layer of at least 6 inches of compacted granular materials below the floor slab. Granular materials used may consist of a crushed, well-graded gravel blend such as SCDOT Graded Aggregate Base Course (GABC), or an open-graded, manufactured washed gravel such as SCDOT No. 57 or No. 67 stone. If washed gravel is used as the underslab layer, then the contractor should plan on using a pump truck to place the floor slab concrete since these materials are cohesionless and are difficult to drive vehicles on. If GABC is used, then either a pump truck or direct discharge from concrete batch trucks may be appropriate depending upon the circumstances. If GABC is used, this underslab layer should be compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D 1557), and tested for density by a representative of S&ME.
4. Have the geotechnical engineer observe a proofroll of all slab subgrades prior to concrete placement. Softened soils may need to be undercut or stabilized before concrete placement.

6.5 Pavement Section Design and Construction Recommendations

Flexible (asphalt) pavements are not recommended for use in areas that will be traveled by the fire trucks. Only rigid Portland cement concrete pavements should be used in those areas. Flexible pavements may be used in the employee parking lot.

We assume that new pavement subgrades will be constructed atop compacted structural fill soils compacted to **at least 98 percent** of the modified Proctor maximum dry density. We have performed our evaluations assuming that a CBR value of at least 19 percent will be available from subgrade soils compacted to 98 percent, which is similar to the CBR test results obtained from the composite sample that we collected on site. If soils exhibiting a CBR value of less than 19 percent at 98 percent compaction are to be used on this project, these recommendations may require revision.

Traffic volumes for the proposed development were not provided to us in preparation for our exploration and pavement section analysis. Based upon our previous experience on similar fire station projects, we have assumed traffic load information. A required capacity of about 880,000 Equivalent Single Axle Loads (ESALs) was estimated for the rigid (concrete) pavements subjected to fire truck/rescue traffic. The volumes for light-duty asphalt pavements are based on an assumption of 60 passenger vehicle or light truck trips per day. Both sections assume a design life of 20 years. The resulting recommended pavement section components are provided in Table 6-1 below.

For flexible pavements, the pavement thickness computations were made using the AASHTO method, assuming an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 95 percent. Assuming that only SCDOT approved source materials will be used in flexible pavement section construction, we used a structural layer coefficient of 0.44 for the HMA layers and a coefficient of 0.18 for the graded aggregate base course (GABC).

Rigid pavement design assumes an initial serviceability of 4.5 and a terminal serviceability index of 2.5, and a reliability factor of 90 percent. Assuming that appropriately designed load transfer devices (dowels) will be used at the joints in the rigid pavement, we used an average load transfer coefficient of 3.2. We also assumed a minimum 28-day design compressive strength of at least 4,000 psi for the PCC. A sub-base drainage factor of 1.0 was assigned, based upon the assumption that the sub-base soils will consist of granular soils.

If reinforced joint design with appropriate load transfer devices (such as steel dowels) is not provided, then the rigid pavement section thickness design would need to be reconsidered using a higher load transfer coefficient, which may result in an increase in the pavement section thickness to maintain a similar ESAL capacity.

Table 6-1: Recommended Minimum Pavement Sections[(a)]

Pavement Area	Theoretical Applied Traffic Load 20 years (ESALs)	HMA Surface Course Type C (inches)	4,000 psi Doweled Joint Concrete Pavement (inches)	Compacted SCDOT Graded Aggregate Base Course [GABC] (inches)	Gravel Drainage Layer (Washed No. 57 or No. 67 Granite) (inches)
Light-Duty Flexible (Asphalt)	94,000	2.0	---	6.0	---
Heavy-Duty Rigid (Concrete)	880,000	---	7.0	6.0	6.0

(a) Single-stage construction and soil compaction as recommended is assumed; S&ME, Inc. must observe pavement subgrade preparation and pavement installation operations.

6.5.1 *Pavement Drainage Systems*

The site civil engineer should determine the specific layout of the drainage system for the project based on these recommendations.

1. Within the rigid concrete pavement areas, a gravel drainage blanket layer 6 inches in thickness should be constructed along with the proper base course and pavement section. See also Figure 4 in Appendix I for a typical pavement section detail showing the drainage layer.

    A. The drainage layer, located between the soil subgrade and the graded aggregate base course, should consist of a washed, open graded, manufactured granitic gravel meeting the gradation of SCDOT No.57 or No. 67 stone.

B. Non-woven geotextile filter fabric (Mirafi 140N) is recommended to be placed between the GABC layer and the drainage layer, to provide separation and filtration;
C. Woven geotextile (Mirafi HP-370) is recommended to be placed between the drainage layer and the subgrade to provide separation and tensile reinforcement.
D. The gravel drainage layer should be at least 6 inches in thickness.

2. In addition to the gravel drainage (blanket) layer described above, perimeter underdrains may also be considered by the civil engineer for inclusion in the pavement area design due to the presence of the shallow silty and clayey sands that may promote the development of near-surface perched water conditions at certain times of the year, although shallow perched groundwater was *not observed* at the time of our exploration.

   A. The site civil engineer should be consulted regarding the type and location of the perimeter underdrains, if any. Our experience is that two types of underdrain systems are commonly used in this locality, depending upon the traffic application and the preferences of the civil engineer. One commonly used system is a gravel-filled, fabric-wrapped trench, or "French drain" containing an embedded perforated plastic HDPE pipe. Another type of system that we often see used is an edge drain product such as AdvantEdge by ADS, Inc. This is a fabric-wrapped, perforated HDPE slot style drain. Some engineers have used a combination of these two systems.
   B. If the civil engineer incorporates perimeter French drains into the subsurface drainage system design, then the French drains should be constructed using the same No. 57 or No. 67 stone, and should be wrapped in a non-woven geotextile, such as Mirafi's 140N Series fabric. French drains should tie into the nearest storm sewer catch basin, or other discharge points as directed by the site civil engineer.

3. If there are landscaped islands in the parking lot, do not fill the islands with clayey or silty (impermeable) spoils that may impede the movement of water into the underdrains.

6.5.2 General Pavement Section Construction Recommendations

The following general recommendations are provided regarding pavement construction:

1. Fill placed in pavement areas should be compacted to **at least 98 percent** of the ASTM D 1557 maximum dry density as recommended previously in this report. Prior to pavement section installation, all exposed pavement area subgrades should be methodically proofrolled at final subgrade elevation under the observation of S&ME, Inc., and any identified unstable areas should be repaired as directed.
2. The stone base course underlying pavements should consist of a graded aggregate base course (GABC) as specified by the SCDOT 2007 Standard Specifications for Highway Construction, Section 305. Proposed materials for use should be provided by a SCDOT-approved source. **Do not substitute "commercial grade" base course for SCDOT-approved base course material.**
3. As stated in the SCDOT Section 305 specification, all new base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC T-140). Base courses should not exhibit pumping or rutting under equipment traffic. Heavy compaction equipment is likely to be required in order to achieve the required base course compaction, and the moisture content of the material will likely need to be maintained very near the optimum moisture content in order to

facilitate proper compaction. S&ME, Inc. should be contacted to perform field density and thickness testing of the base course prior to paving.

4. Experience indicates that a thin surface overlay of asphalt pavement may be required in about 7 to 10 years due to normal wear and weathering of the surface. Such wear is typically visible in several forms of pavement distress, such as aggregate exposure and polishing, aggregate stripping, asphalt bleeding, and various types of cracking. There are means to methodically estimate the remaining pavement life based on a systematic statistical evaluation of pavement distress density and mode of failure. We recommend the pavement be evaluated in about 7 years to assess the pavement condition and remaining life.
5. Construct the HMA surface course in accordance with the specifications of Section 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition). Construct HMA intermediate courses in accordance with the specifications of Section 402 of this same specification.
6. It is very important for this project that the asphaltic concrete be properly compacted, as specified in Section 401.4 of the SCDOT specification. Asphaltic concrete that is insufficiently compacted will show wear much more rapidly than if it were properly compacted.
7. Sufficient testing should be performed during flexible pavement installation to confirm that the required thickness, density, and quality requirements of the pavement specifications are followed.
8. For rigid pavements, we recommend air-entrained ASTM C 94 joint reinforced Portland cement concrete that will achieve a minimum compressive strength of at least 4,000 psi at 28 days after placement, as determined by ASTM C 39. We also recommend that the pavement concrete be constructed in a manner which at least meets the minimum standards recommended by the American Concrete Institute (ACI).
9. We recommend that at least 1 set of 5 cylinder specimens be cast by S&ME per every 50 cubic yards of concrete placed or at least once per placement event in order to measure achievement of the design compressive strength. We also recommend that S&ME be present on site to observe concrete placement.

7.0 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our boring and sounding locations may not become evident until construction. If variations appear evident, then we should be provided the opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structure are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers. Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.

Appendices

Appendix I

Site Vicinity Plan

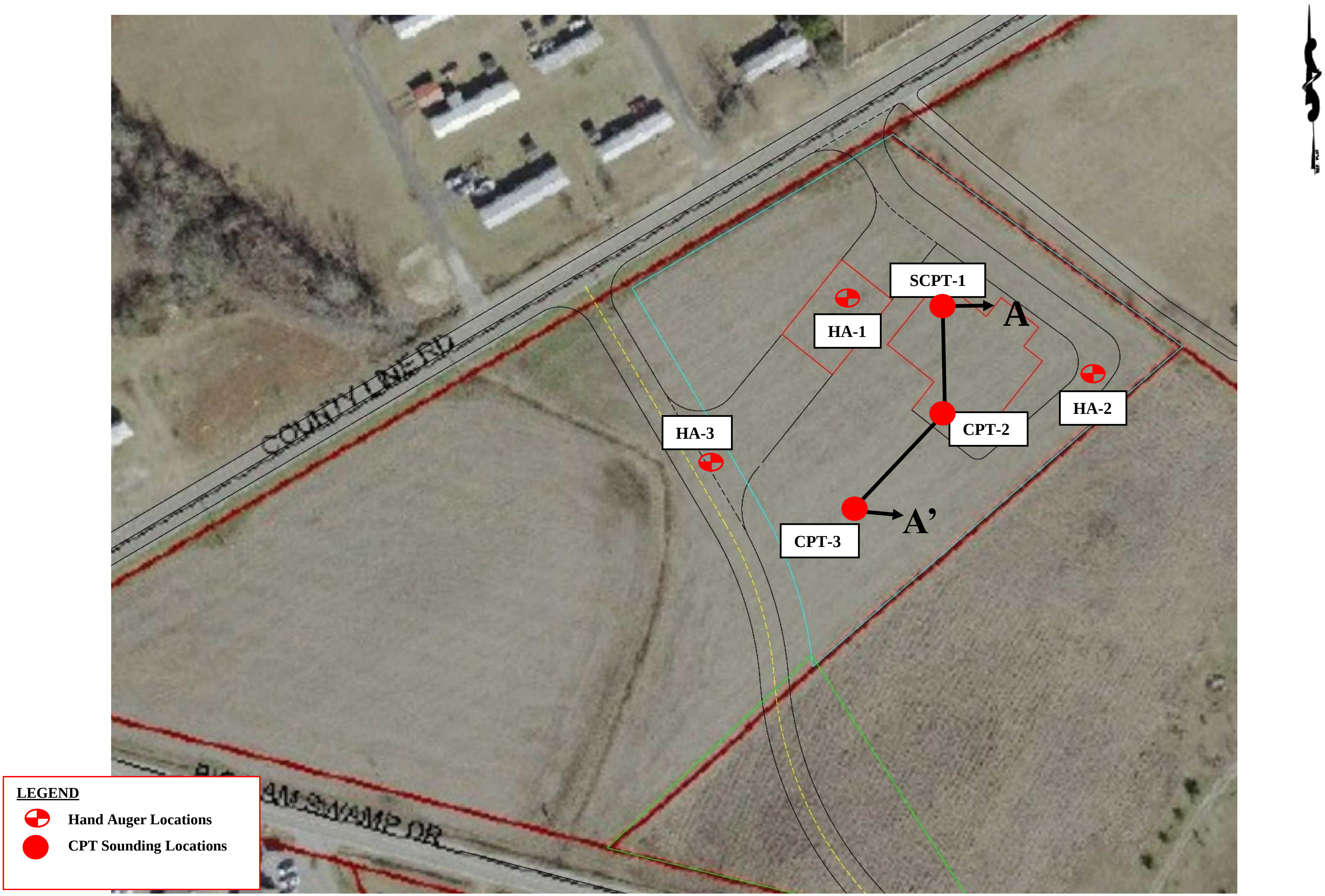
Boring Location Sketch

Interpreted Subsurface Profile
A-A'

Typical Pavement Section Detail

Shear Wave Velocity Profile

SCALE:	Not To Scale	S&ME	**SITE VICINITY PLAN** **Big Dam Swamp Fire Station No.15** **Andrews, South Carolina**	FIGURE NO **1**
SOURCE:	Google Earth			
DATE:	May, 2016		JOB NO. **1463-16-019**	
DRAWN BY:	CDJ			



SCALE:	Not To Scale
SOURCE:	Google Earth
DATE:	April, 2015
DRAWN BY:	CDJ

S&ME

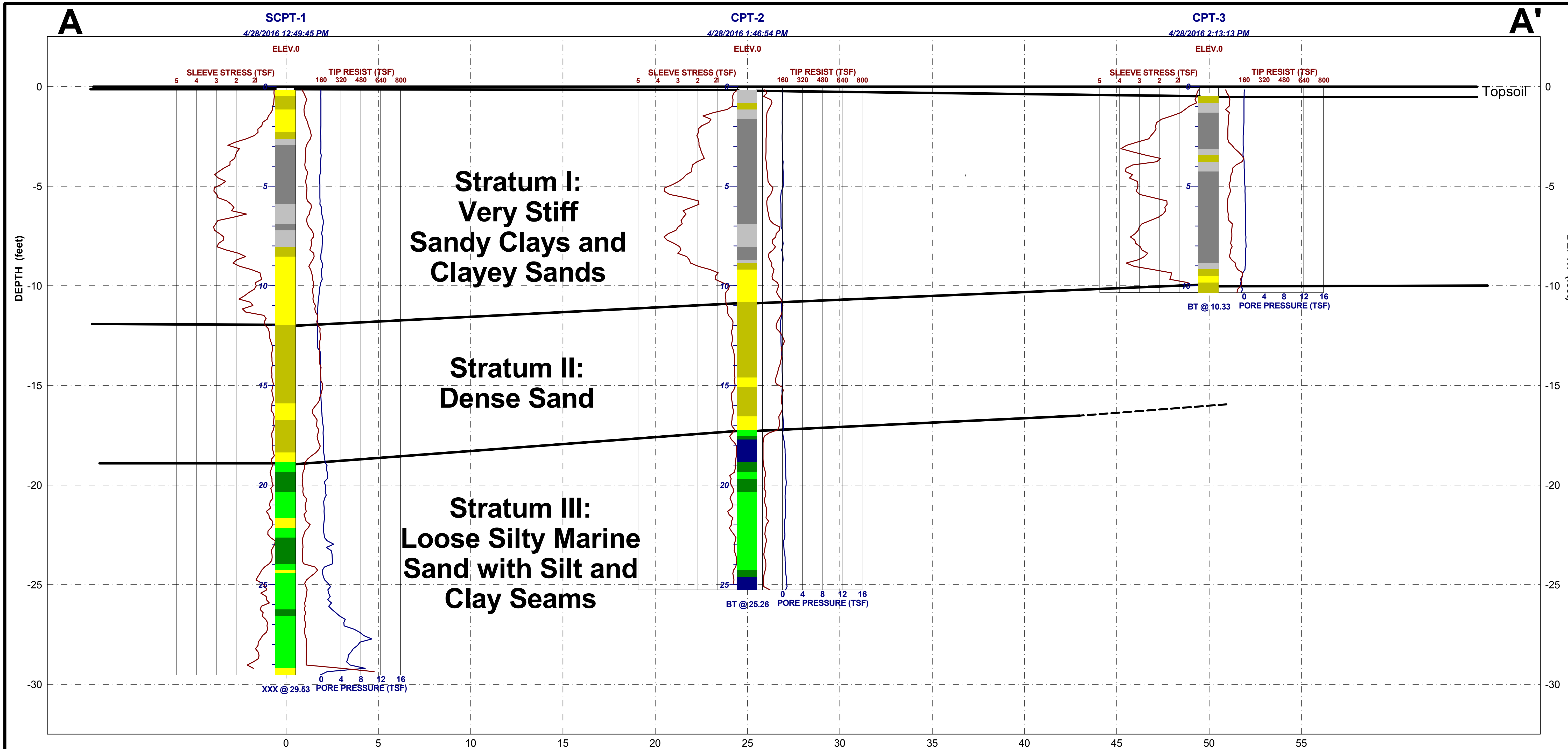
Boring Location Sketch

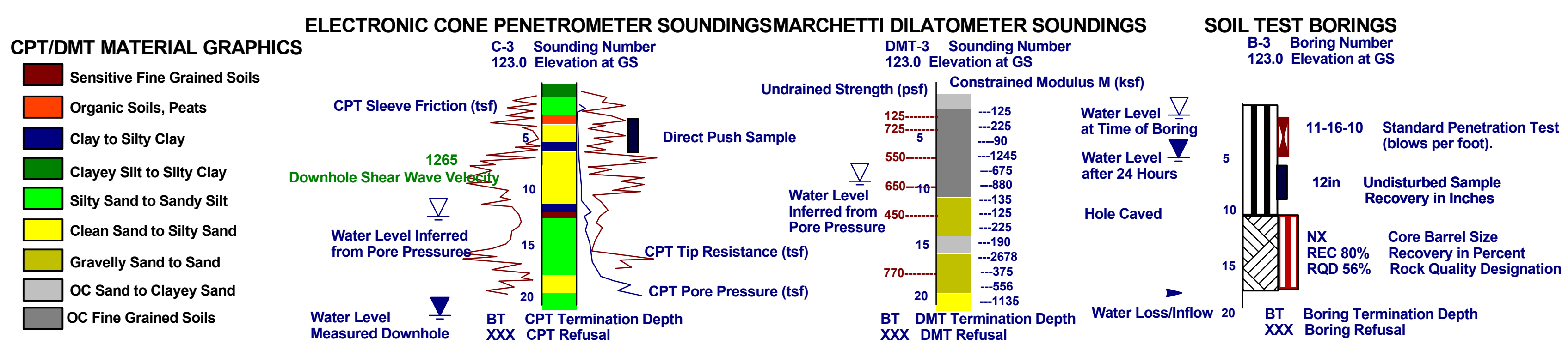
Big Dam Swamp Fire Station No. 15

Andrews, South Carolina

JOB NO. **1463-16-019**

FIGURE NO **2**





The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

SUBSURFACE PROFILE

FIGURE: #3

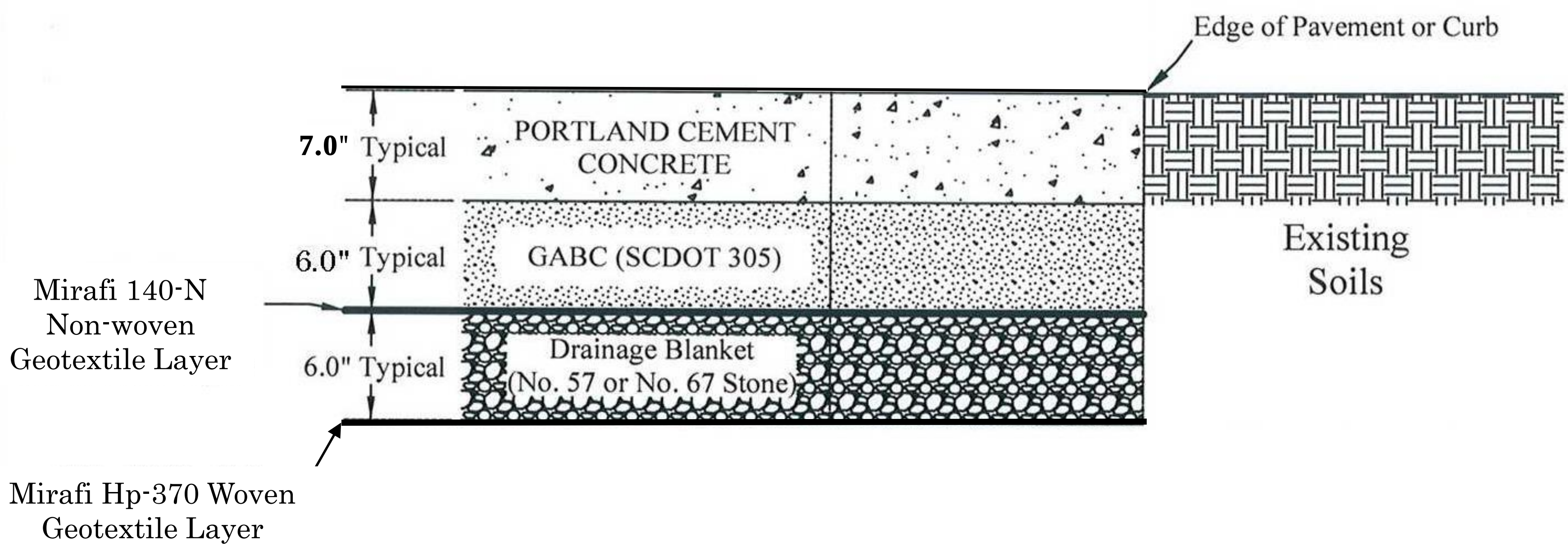
PROJECT: Big Dam Swamp Fire Station No. 15

LOCATION: Andrews, South Carolina

JOB NO: 1463-16-019

DATE: 5/13/16

S&ME



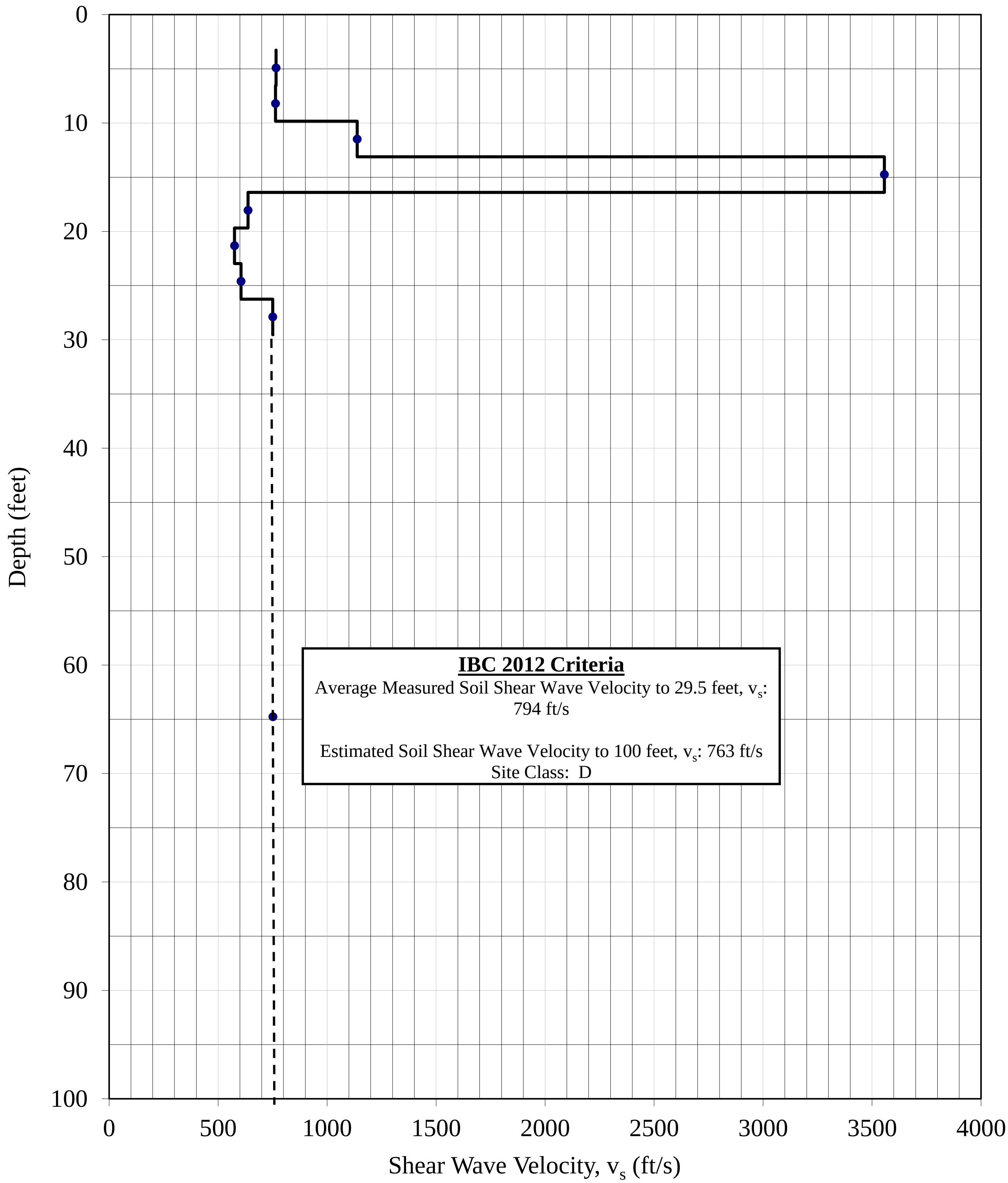
SCALE: Not To Scale	S&ME	**Typical Pavement Section Detail** **Big Dam Swamp Fire Station No. 15** **Andrews, South Carolina**	FIGURE NO **4**
CHECKED BY: RPF			
DATE: May, 2016		JOB NO. **1463-16-019**	
DRAWN BY: CDJ			

Shear Wave Velocity Calculations

Big Dam Swamp Fire Station No. 15
Georgetown County, SC

Sounding ID: **SCPT-1**
Date: 04/28/16

Project Number: **1463-16-019**



* Site Class based on 2012 International Building Code - *Table 1613.5.2 - SITE CLASS DEFINITIONS*

Appendix II

Summary of Exploration Procedures

CPT Soil Classification Legend

CPT Logs

Soil Classification Chart

Hand Auger Logs

Infiltration Rate of Soils in Field

❖ Summary of Exploration Procedures

The American Society for Testing and Materials (ASTM) publishes standard methods to explore soil, rock and ground water conditions in Practice D-420-98, "*Standard Guide to Site Characterization for Engineering Design and Construction Purposes*." The boring and sampling plan must consider the geologic or topographic setting. It must consider the proposed construction. It must also allow for the background, training, and experience of the geotechnical engineer. While the scope and extent of the exploration may vary with the objectives of the client, each exploration includes the following key tasks:

- Reconnaissance of the Project Area
- Preparation of Exploration Plan
- Layout and Access to Field Sampling Locations
- Field Sampling and Testing of Earth Materials
- Laboratory Evaluation of Recovered Field Samples
- Evaluation of Subsurface Conditions

The standard methods do not apply to all conditions or to every site. Nor do they replace education and experience, which together make up engineering judgment. Finally, ASTM D 420 does not apply to environmental investigations.

❖ Reconnaissance of the Project Area

We walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures.

Checks for Hazardous Conditions - State law requires that we notify the Palmetto Utility Protection Service (PUPS) before we drill or excavate at any site. PUPS is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. PUPS forwarded our location request to the participating utilities. Location crews then marked buried lines with colored flags within 72 hours. They did not mark utility lines beyond junction boxes or meters. We checked proposed sampling points for conflicts with marked utilities, overhead power lines, tree limbs, or man-made structures during the site walkover.

❖ Boring and Sampling

Electronic Cone Penetrometer (CPT) Soundings

CPT soundings consist of a conical pointed penetrometer which is hydraulically pushed into the soil at a slow, measured rate. Procedures for measurement of the tip resistance and side friction resistance to push generally follow those described by ASTM D-5778, "*Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils*."

A penetrometer with a conical tip having a 60 degree apex angle and a cone base area of 10 cm^2 was advanced into the soil at a constant rate of 20 mm/s. The force on the conical point required to penetrate the soil was measured electronically every 50 mm penetration to obtain the *cone resistance* q_c. A friction sleeve is present on the penetrometer immediately behind the cone tip. The force exerted on the sleeve was measured electronically at a minimum of every 50 mm penetration and divided by the surface area of the sleeve to obtain the *friction sleeve resistance value* f_s A pore pressure element mounted immediately behind the cone tip was used to measure the pore pressure induced during advancement of the cone into the soil.

CPT Soil Stratification

Using ASTM D-5778 soil samples are not obtained. Soil classification was made on the basis of comparison of the tip resistance, sleeve resistance and pore pressure values to values measured at other locations in known soil types, using experience with similar soils and exercising engineering judgment.

Plots of normalized tip resistance versus friction ratio and normalized tip resistance versus penetration pore pressure were used to determine soil classification (Soil Behavior Type, SBT) as a function of depth using empirical charts developed by P.K. Robertson (1990). The friction ratio soil classification is determined from the chart in the appendix using the normalized corrected tip stress and the normalized corrected tip stress and the normalized friction ratio.

At some depths, the CPT data fell outside of the range of the classification chart. When this occurred, no data was plotted and a break was shown in the classification profile. This occasionally occurred at the top of a penetration as the effective vertical stress is very small and commonly produced normalized tip resistances greater than 1000.

To provide a simplified soil stratigraphy for general interpretation and for comparison to standard boring logs, a statistical layering and classification system was applied the field classification values. Layer thicknesses were determined based on the variability of the soil classification profile, based upon changes in the standard deviation of the SBT classification number with depth. The average SBT number was determined for each successive 6-inch layer, beginning at the surface. Whenever an additional 6-inch increment deviated from the previous increment, a new layer was started, otherwise, this material was added to the layer above and the next 6-inch section evaluated. The soil behavior type for the layer was determined by the mean value for the complete layer.

Hand Auger Borings with Dynamic Cone Penetrometer Testing

Auger borings were advanced using hand operated augers. The soils encountered were identified in the field by cuttings brought to the surface. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. Dynamic Cone Penetrometer (DCP) testing was performed in conjunction within the borings in general accordance with ASTM STP 399, "*Dynamic Cone for Shallow In-Situ Penetration Testing*". At selected intervals, the augers were withdrawn and soil consistency measured with a dynamic cone penetrometer. The conical point of the penetrometer was first seated 1-3/4 inches to penetrate any loose cuttings in the boring, then driven two additional 1-3/4 inch increments by a 15 pound hammer falling 20 inches. The number of hammer blows required to achieve this penetration was recorded. When properly evaluated by qualified professional staff, the blow count is an index to the soil strength. Hand auger borings were backfilled with soil cuttings after termination of drilling.

Water Level Determination

Water levels in the boreholes were measured during the onsite exploration by measuring depths from the existing grade to the current water level using a tape.

Backfilling of Borings

Once subsurface water levels were obtained, boring spoils were backfilled into the open bore holes. Bore holes were backfilled to the existing ground surface. The CPT sounding holes are only 2 inches in diameter and were not backfilled.

CPT Soil Classification Legend

Zone	Q_t/N	Description
1	2	Sensitive, Fine Grained
2	1	Organic Soils-Peats
3	1.5	Clays-Clay to Silty Clay
4	2	Silt Mixtures-Clayey Silt to Silty Clay
5	3	Sand Mixtures-Silty Sand to Sandy Silt
6	4.5	Sands-Clean Sand to Silty Sand
7	6	Gravelly Sand to Sand
8	1	Very Stiff Clay to Clayey Sand*
9	2	Very Stiff, Fine Grained*

(*) Heavily Overconsolidated or Cemented

Robertson's Soil Behavior Type (SBT), 1990			
Group #	**Description**	**Ic**	
		Min	**Max**
1	Sensitive, fine grained	N/A	
2	Organic soils - peats	3.60	N/A
3	Clays - silty clay to clay	2.95	3.60
4	Silt mixtures - clayey silt to silty clay	2.60	2.95
5	Sand mixtures - silty sand to sandy silt	2.05	2.60
6	Sands - clean sand to silty sand	1.31	2.05
7	Gravelly sand to dense sand	N/A	1.31
8	Very stiff sand to clayey sand (High OCR or cemented)	N/A	
9	Very stiff, fine grained (High OCR or cemented)	N/A	

Soil behavior type is based on empirical data and may not be representative of soil classification based on plasticity and grain size distribution.

Relative Density and Consistency Table			
SANDS		SILTS and CLAYS	
Cone Tip Stress, qt (tsf)	Relative Density	Cone Tip Stress, qt (tsf)	Consistency
Less than 20	Very Loose	Less than 5	Very Soft
20 - 40	Loose	5 - 15	Soft to Firm
40 - 120	Medium Dense	15 - 30	Stiff
120 - 200	Dense	30 - 60	Very Stiff
Greater than 200	Very Dense	Greater than 60	Hard

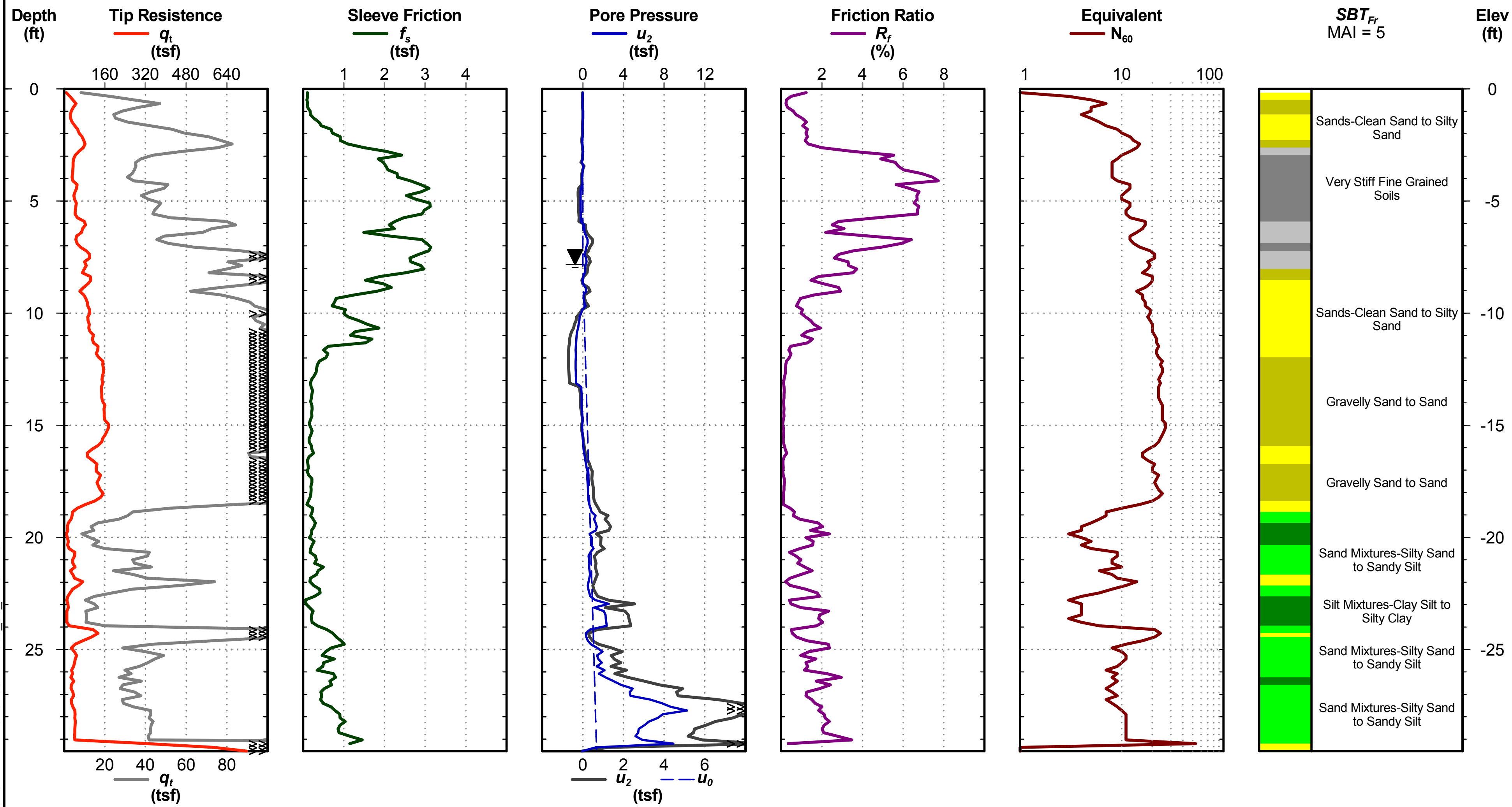
Big Dam Swamp Fire Station No. 15
Andrews, South Carolina
S&ME Project No: 1463-16-019

Cone Penetration Test

SCPT-1

Date: Apr. 28, 2016
Estimated Water Depth: 7.83 ft
Rig/Operator: Austin Fowler

Total Depth: 29.5 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.44



SCPT-1

Electronic Filename: BDFS SCPT-1.cpt

S&ME

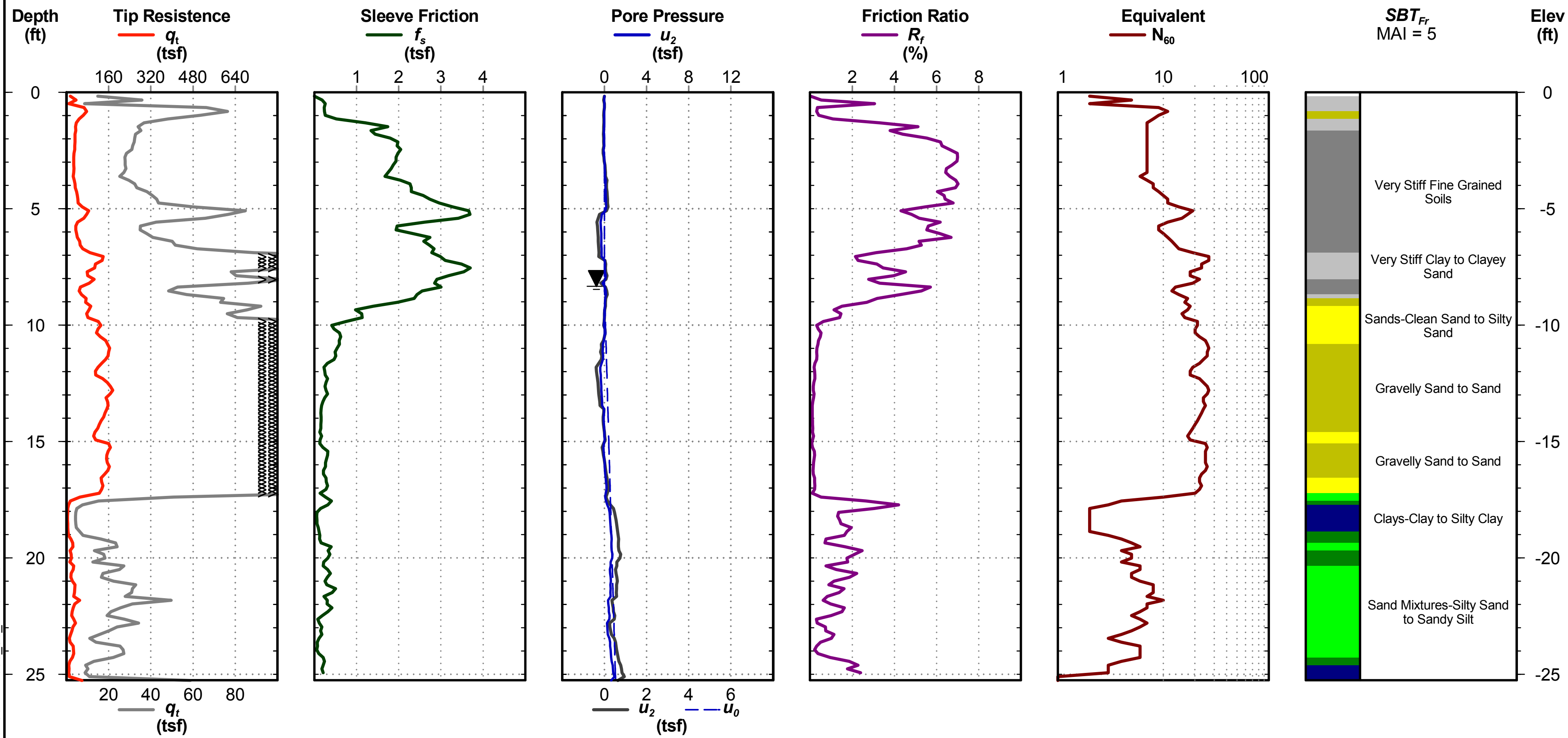
Big Dam Swamp Fire Station No. 15
Andrews, South Carolina
S&ME Project No: 1463-16-019

Cone Penetration Test

CPT-2

Date: Apr. 28, 2016
Estimated Water Depth: 8.33 ft
Rig/Operator: Austin Fowler

Total Depth: 25.3 ft
Termination Criteria: Target Depth
Cone Size: 1.44



Electronic Filename: BDFS CPT-2.cpt

CPT-2

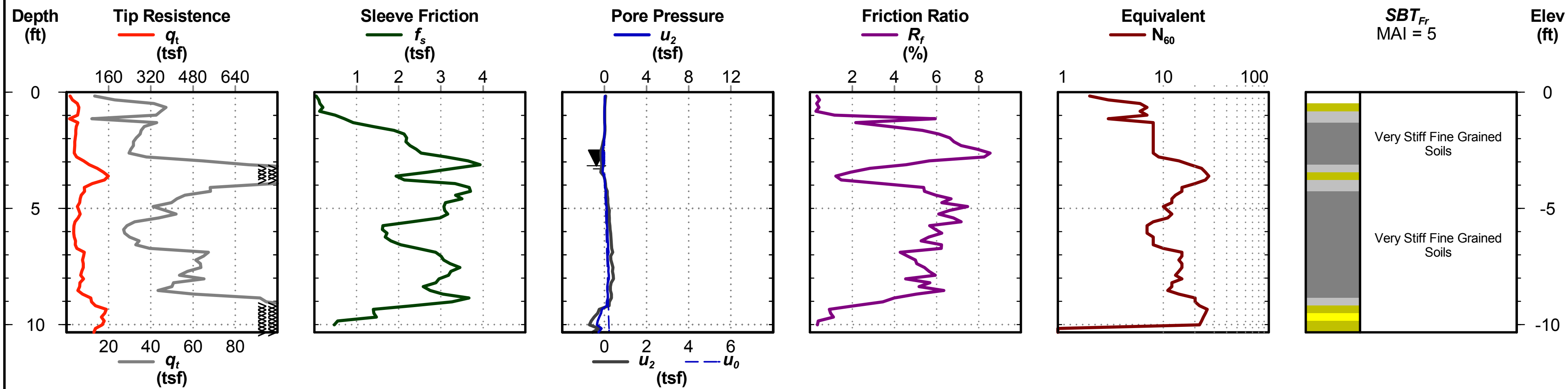
Big Dam Swamp Fire Station No. 15
Andrews, South Carolina
S&ME Project No: 1463-16-019

Cone Penetration Test

CPT-3

Date: Apr. 28, 2016
Estimated Water Depth: 3.17 ft
Rig/Operator: Austin Fowler

Total Depth: 10.3 ft
Termination Criteria: Target Depth
Cone Size: 1.44



CPT-3

Electronic Filename: BDFS CPT-3.cpt

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

PROJECT: Big Dam Swamp Firestation No. 15, Andrews, South Carolina, 1463-16-019		HAND AUGER BORING LOG: CPT-1
DATE STARTED: 5/9/16	DATE FINISHED: 5/9/16	NOTES: Elevation Unknown
SAMPLING METHOD: Hand Auger	PERFORMED BY: C. Jones	
WATER LEVEL: **Not Encountered**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
0–1		TOPSOIL - Approximately 12 inches.		
1–4		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange, brown, moist.		
4		Boring terminated at 4 ft Target Depth		

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: Big Dam Swamp Firestation No. 15 Andrews, South Carolina 1463-16-019	HAND AUGER BORING LOG: CPT-2

DATE STARTED: **5/9/16**	DATE FINISHED: **5/9/16**
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **C. Jones**
WATER LEVEL: **Not Encountered**	

NOTES: **Elevation Unknown**

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
0–1		TOPSOIL - Approximately 12 inches.		
1–4		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange, brown, gray, moist.		
4		Boring terminated at 4 ft Target Depth		

S&ME

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT:	Big Dam Swamp Firestation No. 15 Andrews, South Carolina 1463-16-019	HAND AUGER BORING LOG: CPT-3/I-1

DATE STARTED: **5/9/16**	DATE FINISHED: **5/9/16**	NOTES: **Elevation Unknown**
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **C. Jones**	
WATER LEVEL: **Not Encountered**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
0–1		TOPSOIL - Approximately 12 inches.		
1–4		SILTY SAND (SM) - Mostly fine to medium sand, few low plasticity fines, brown, orange, tan, moist.		

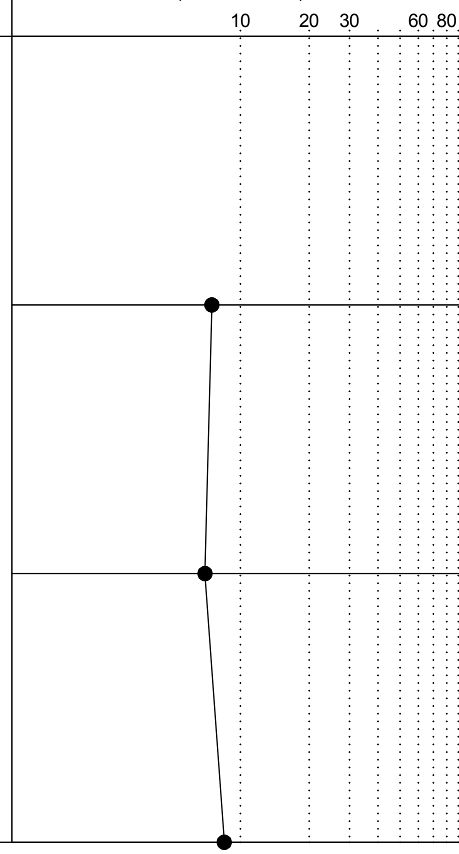
Boring terminated at 4 ft
Target Depth

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT:	Big Dam Swamp Firestation No. 15 Andrews, South Carolina 1463-16-019	HAND AUGER BORING LOG: HA-1

DATE STARTED: **5/9/16**	DATE FINISHED: **5/9/16**	NOTES: **Elevation Unknown**
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **C. Jones**	
WATER LEVEL: **Not Encountered**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.) 10 20 30 60 80	DCP VALUE
0–1.5		TOPSOIL - Approximately 1.5 feet.				
1						8
1.5–3		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange, tan, gray, brown, moist, loose.				
2						7
3		- - - - Boring terminated due to borehole collapse. Boring terminated at 3 ft				9



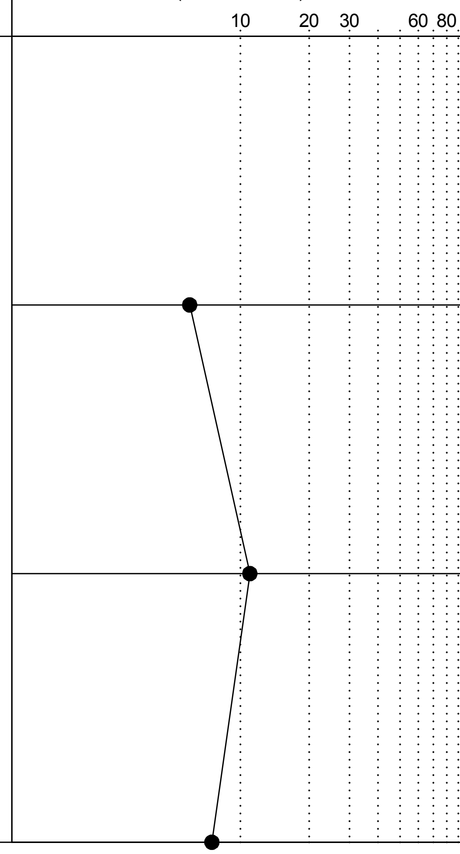
DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

S&ME

PROJECT:	Big Dam Swamp Firestation No. 15 Andrews, South Carolina 1463-16-019	**HAND AUGER BORING LOG: HA-2**

DATE STARTED:	**5/9/16**	DATE FINISHED:	**5/9/16**	NOTES: **Elevation Unknown**
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**C. Jones**	
WATER LEVEL:	**Not Encountered**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)	DCP VALUE
0–1		TOPSOIL - Approximately 12 inches.				
1		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange, tan, gray, red, moist, medium dense to loose.				6
2						11
3		- - - - Boring terminated due to borehole collapse. Boring terminated at 3 ft				8



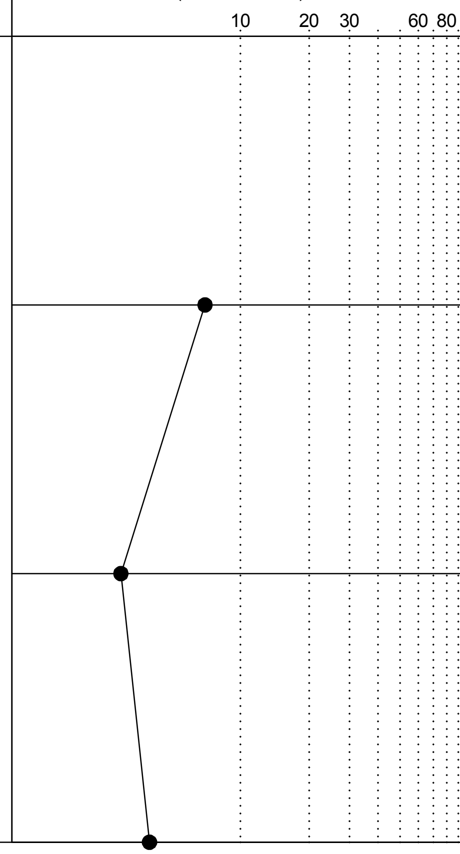
DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: Big Dam Swamp Firestation No. 15 Andrews, South Carolina 1463-16-019	HAND AUGER BORING LOG: HA-3

DATE STARTED: 5/9/16	DATE FINISHED: 5/9/16	NOTES: **Elevation Unknown**
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **C. Jones**	
WATER LEVEL: Not Encountered		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DCP VALUE
0–1		TOPSOIL - Approximately 12 inches.			
1–2		POORLY GRADED SAND WITH CLAY (SP-SC) - Mostly fine to medium sand, few low plasticity fines, dark gray, moist, loose to very loose.			7
2–3		SILTY CLAY (CL-ML) - Mostly low to medium plasticity fines, some fine sand, dark gray, wet, soft.			3
3		- - - - Boring terminated due to borehole collapse. Boring terminated at 3 ft			4

DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)



DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

TABLE 1: INFILTRATION RATE OF SOILS IN FIELD
(BY DOUBLE RING INFILTROMETER)

JOB NAME :	Big Dam Swamp Fire Station No. 15				
JOB NO. :	1463-16-019	**REPORT NO. :** I-1	**TEST DATE :** 05/09/16	**INVESTIGATOR :**	CJ
BORING NO. :	I-1	**DEPTH / ELEV. :** -1 Feet		**REVIEWED BY :**	RF
BORING LOCATION :	See Test Location Sketch				
SOIL DESCRIPTION :	Clayey Sand (SC), Poorly Graded Sand with Clay (SP-SC)				

CONSTANTS	AREA CM^2	DEPTH OF LIQUID CM	MARIOTTE TUBE NO.	ΔVOLUME / ΔH CM^3 / CM
INNER RING	729.7	15.2	**1**	1
ANNULAR SPACE	2105.0	15.2	**2**	1

READING NO.		DATE	TIME	ELAPSED TIME	FLOW READINGS				LIQUID TEMP.	INFILTRATION RATE		REMARKS
					INNER RING		ANNULAR SPACE			INNER	ANNULAR	
			HR:MIN:SEC	MINUTES	READING CM	FLOW CM^3	READING CM	FLOW CM^3	^{O}C	IN. / HOUR	IN. / HOUR	GROUND TEMP. 48 ^{O}F
1	S	05/09/16										
	E	05/09/16		13		4600				11.5		
2	S	05/09/16										
	E	05/09/16		9		2500				9.0		
3	S	05/09/16										
	E	05/09/16		9		2000				7.2		
4	S	05/09/16										
	E	05/09/16		9		2000				7.2		
5	S	05/09/16										
	E	05/09/16		9		2000				7.2		
6	S	05/09/16										
	E	05/09/16		9		2000				7.2		
7	S	05/09/16										
	E	05/09/16		11		2000				5.9		
8	S	05/09/16										
	E	05/09/16		20		3550				5.7		
9	S	05/09/16										
	E	05/09/16		20		3500				5.7		
10	S											
	E											

Appendix III

Summary of Laboratory Procedures

Laboratory Test Results

❖ Summary of Laboratory Procedures

Examination of Recovered Soil Samples

Soil and field records were reviewed in the laboratory by the geotechnical professional. Soils were classified in general accordance with the visual-manual method described in ASTM D 2488, "*Standard Practice for Description and Identification of Soils (Visual-Manual Method)*". Representative soil samples were selected for classification testing to provide grain size and plasticity data to allow classification of the samples in general accordance with the Unified Soil Classification System method described in ASTM D 2487, "*Standard Practice for Classification of Soils for Engineering Purposes*". The geotechnical professional also prepared the final boring and sounding records enclosed with this report.

Moisture Content Testing of Soil Samples by Oven Drying

Moisture content was determined in general conformance with the methods outlined in ASTM D 2216, "*Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil or Rock by Mass*." This method is limited in scope to Group B, C, or D samples of earth materials which do not contain appreciable amounts of organic material, soluble solids such as salt or reactive solids such as cement. This method is also limited to samples which do not contain contamination.

A representative portion of the soil was divided from the sample using one of the methods described in Section 9 of ASTM D 2216. The split portion was then placed in a drying oven and heated to approximately 110 degrees C overnight or until a constant mass was achieved after repetitive weighing. The moisture content of the soil was then computed as the mass of water removed from the sample by drying, divided by the mass of the sample dry, times 100 percent. No attempt was made to exclude any particular particle size from the portion split from the sample.

Liquid and Plastic Limits Testing

Atterberg limits of the soils was determined generally following the methods described by ASTM D 4318, "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.*" Albert Atterberg originally defined "limits of consistency" of fine grained soils in terms of their relative ease of deformation at various moisture contents. In current engineering usage, the *liquid limit* of a soil is defined as the moisture content, in percent, marking the upper limit of viscous flow and the boundary with a semi-liquid state. The *plastic limit* defines the lower limit of plastic behavior, above which a soil behaves plastically below which it retains its shape upon drying. The *plasticity index* (PI) is the range of water content over which a soil behaves plastically. Numerically, the PI is the difference between liquid limit and plastic limit values.

Representative portions of fine grained Group A, B, C, or D samples were prepared using the wet method described in Section 10.1 of ASTM D 4318. The liquid limit of each sample was determined using the multipoint method (Method A) described in Section 11, or the one-point method (Method B) described in Section 13. The liquid limit is by definition the moisture content where 25 drops of a hand operated liquid limit device are required to close a standard width groove cut in a soil sample placed in the device.

Multi-Point Method
After each test, the moisture content of the sample was adjusted and the sample replaced in the device. The test was repeated to provide a minimum of three widely spaced combinations of N versus moisture content. When plotted on semi-log paper, the liquid limit moisture content was determined by straight line interpolation between the data points at N equals 25 blows.

One-Point Method
The procedure for the one-point method is the same as the multi-point method except that the number of blows required to close the groove is 20 to 30. If less than 20 or more than 30 blows are required, the water content of the soil is adjusted and the procedure is repeated. The liquid limit is determined in accordance with Section 14.

The plastic limit was determined using the procedure described in Sections 15 through 17 of ASTM D 4318. A selected portion of the soil used in the liquid limit test was kneaded and rolled by hand until it could no longer be rolled to a 3.2 mm thread on a glass plate. This procedure was repeated until at least 6 grams of material was accumulated, at which point the moisture content was determined using the methods described in ASTM D 2216

Grain Size Analysis of Samples

The distribution of particle sizes greater than 75 mm was determined in general accordance with the procedures described by ASTM D 421, "*Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants", and D 422, "Standard Test Method for Particle Size Analysis of Soils*," except that the hydrometer portion of the test standard was not utilized. During preparation samples were divided into two portions. The material coarser than the No. 30 U.S. sieve size fraction was dry sieved through a nest of standard sieves as described in Article 6. Material passing the No. 30 sieve was independently passed through a nest of sieves down to the No. 200 size.

Percent Fines Determination of Samples

A selected specimen of soils was washed over a No. 200 sieve after being thoroughly mixed and dried. This test was conducted in general accordance with ASTM D 1140, "*Standard Test Method for Amount of Material Finer Than the No. 200 Sieve*." Method A, using water to wash the sample through the sieve without soaking the sample for a prescribed period of time, was used and the percentage by weight of material washing through the sieve was deemed the "percent fines" or percent clay and silt fraction.

Compaction Tests of Soils Using Modified Effort

Soil placed as engineering fill is compacted to a dense state to obtain satisfactory engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure the required compaction and water contents are achieved. Test procedures generally followed those described by ASTM D1557, "*Standard Test Methodfor Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 lbf/ft^3).*"

The relationship between water content and the dry unit weight is determined for soils compacted in either 4 or 6 inch diameter molds with a 10 lbf rammer dropped from a height of 18 inches, producing a compactive effort of 56,000 lbf/ft^3. ASTM D 1557 provides three alternative procedures depending on material gradation:

Method A

All material passes No. 4 sieve size
4 inch diameter mold
Shall be used if 20 percent or less by weight is retained on No. 4 sieve
Soil in 5 layers with 25 blows per layer

Method B

All material passes 3/8 inch sieve
4 inch diameter mold
Shall be used if 20 percent by weight is retained on the No. 4 sieve and 20 percent or less by weight is retained on the 3/8 Inch sieve.
Soil in 5 layers with 25 blows per layer

Method C

All material passes $^3/_4$ inch sieve
6-inch diameter mold
Shall be used if more than 20 percent by weight is retained on the 3/8 inch sieve and less than 30 percent is retained on the $^3/_4$inch sieve.
Soil in 5 layers with 56 blows per layer

Soil was compacted in the mold in five layers of approximately equal thickness, each compacted with either 25 or 56 blows of the rammer. After compaction of the sample in the mold, the resulting dry density and moisture content was determined and the procedure repeated. Separate soils were used for each sample point, adjusting the moisture content of the soil as described in Section 10.2 (Moist Preparation Method). The procedure was repeated for a sufficient number of water content values to allow the dry density vs. water content values to be plotted and the *maximum dry density* and *optimum moisture content* to be determined from the resulting curvilinear relationship

Laboratory California Bearing Ratio Tests of Compacted Samples

This method is used to evaluate the potential strength of subgrade, subbase, and base course material, including recycled materials, for use in road and airfield pavements. Laboratory CBR tests were run in general accordance with the procedures laid out in ASTM D 1883, "*Standard Test Methodfor CBR (California Bearing Ratio) of Laboratory Compacted Soils*." Specimens were prepared in standard molds using two different levels of compactive effort within plus or minus 0.5 percent of the optimum moisture content value. While embedded in the compaction mold, each specimen was inundated for a minimum period of 96 hours to achieve saturation. During inundation, the specimen was surcharged by a weight approximating the anticipated weight of the pavement and base course layers. After removing the sample from the soaking bath, the soil was then sheared by jacking a piston having a cross sectional area of 3 square inches into the end surface of the specimen. The piston was jacked 0.5 inches into the specimen at a constant rate of 0.05 inches per minute.

The CBR is defined as the load required to penetrate a material to a predetermined depth, compared to the load required to penetrate a standard sample of crushed stone to the same depth. The CBR value was usually based on the load ratio for a penetration of 0.10 inches, after correcting the load-deflection curves for surface irregularities or upward concavity. However, where the calculated CBR for a penetration of 0.20 inches was greater than the result obtained for a penetration of 0.10 inches, the test was repeated by reversing the specimen and shearing the opposite end surface. Where the second test indicated a greater CBR at 0.20 inches penetration, the CBR for 0.20 inches penetration was used.

CBR (California Bearing Ratio) of Laboratory Compacted Soil

ASTM D 1883

Quality Assurance

S&ME, Inc.Myrtle Beach 1330 Highway 501 Business; Conway, SC 29526

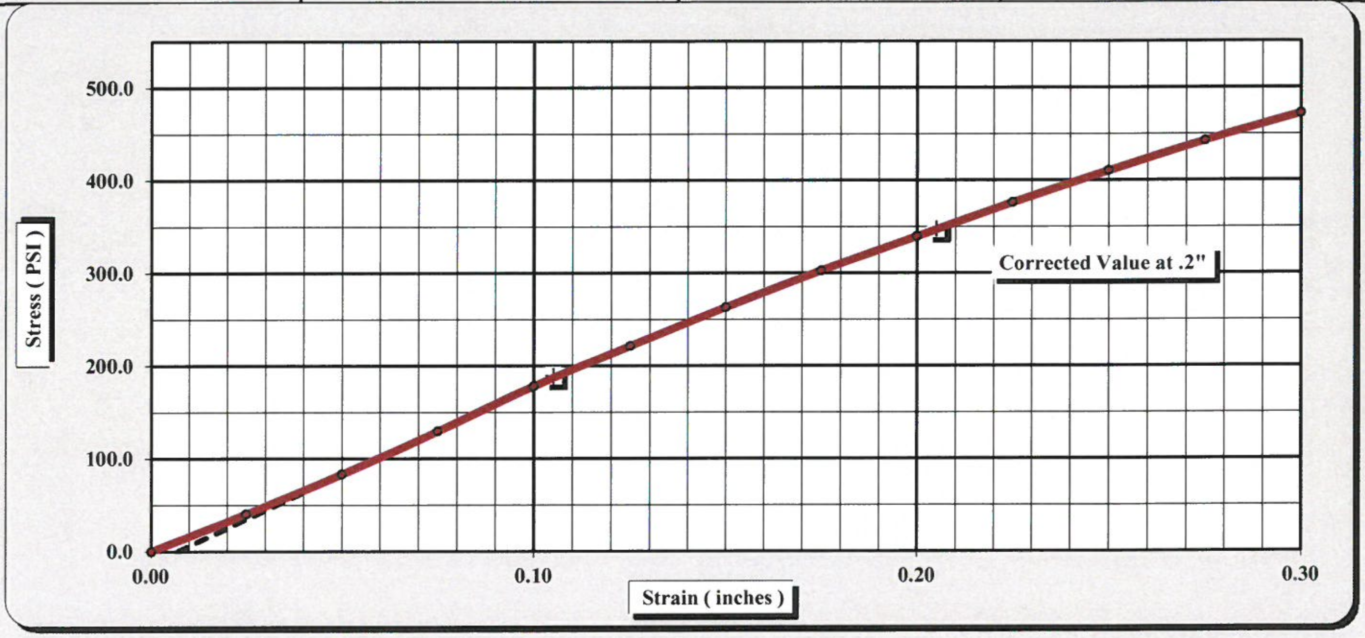
Project #:	**1463-16-019**	Report Date:	5/17/2016
Project Name:	New Big Dam Swamp Fire Station #15	Test Date(s)	5/12/2016
Client Name:	Georgetown County		
Client Address:	129 Screven Street, Suite 239; Georgetown, SC 29440		
Boring #: CPT-3	Sample #: Bulk	Sample Date: 5/9/2016	
Location: Proposed Pond	Lab # 3842	Elevation: 1'-2'	

Sample Description: Brown Silty Sand (SM)

Maximum Dry Density: **109.6 PCF** Optimum Moisture Content: **9.2%**

Compaction Test performed on grading complying with CBR spec. % Retained on the 3/4" sieve: **0.0%**

Uncorrected CBR Values				**Corrected CBR Values**			
CBR at 0.1 in.	**17.8**	**CBR at 0.2 in.**	**22.6**	**CBR at 0.1 in.**	**19.0**	**CBR at 0.2 in.**	**23.2**



CBR Sample Preparation:

Grading was in accordance with the above method and compacted using the 6" diameter CBR mold. ASTM D1883, Section 6.1.1

Before Soaking		*After Soaking*	
Compactive Effort (Blows per Layer)	25	Final Dry Density (PCF)	100.7
Initial Dry Density (PCF)	104.1	Average Final Moisture Content	13.0%
Moisture Content of the Compacted Specimen	9.4%	Moisture Content (top 1" after soaking)	11.5%
Percent Compaction	95.0%	Percent Swell	0.0%

Soak Time:	96 hrs.	Surcharge Weight	20.0	Surcharge Wt. per sq. Ft.	102.0
Liquid Limit	--	Plastic Index	NP	Apparent Relative Density	--

Notes/Deviations/References: Liquid Limit: ASTM D 4318, Classification: ASTM D 2487

Chelsea Jones		Staff Professional	5/23/16
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D698-2
Revision No. : 0
Revision Date: 11/21/07

Moisture - Density Report

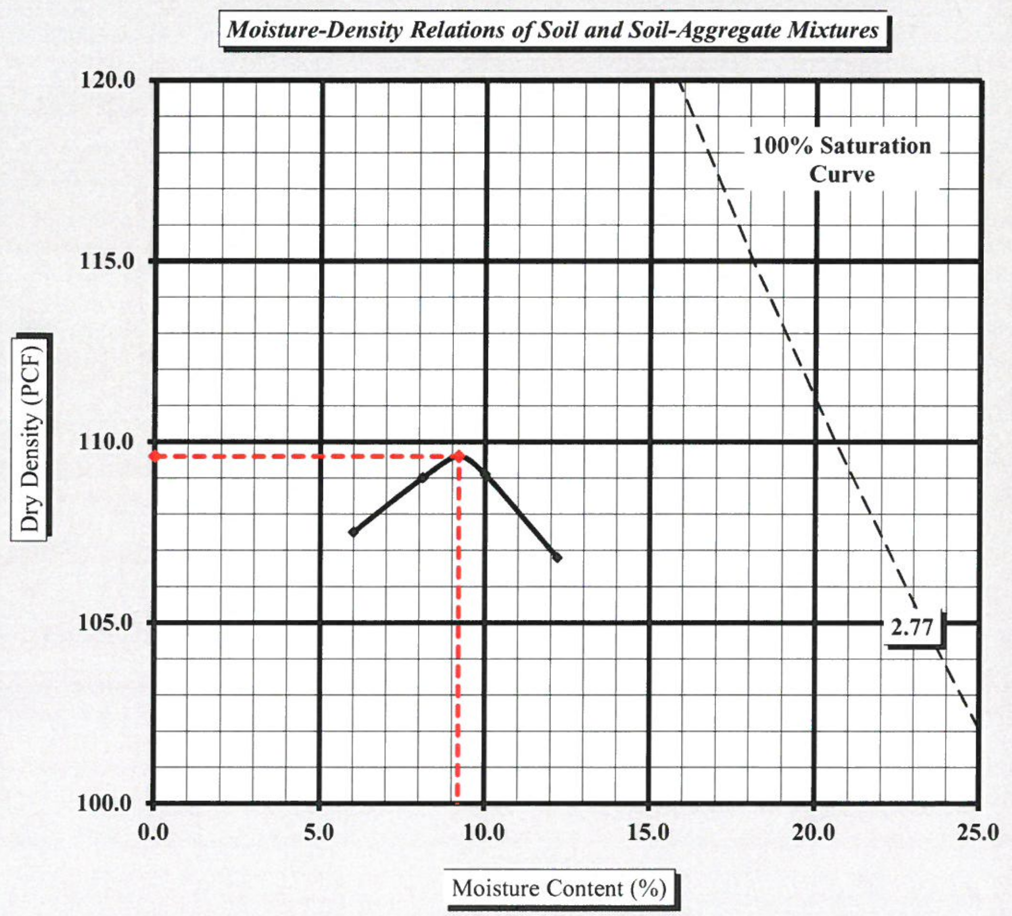
Quality Assurance

S&ME, Inc.- Myrtle Beach 1330 Highway 501 Business; Conway, SC 29526

S&ME Project #:	1463-16-019			Report Date:	5/17/2016
Project Name:	New Big Dam Swamp Fire Station #15			Test Date(s):	5/11/2016
Client Name:	Georgetown County				
Client Address:	129 Screven Street, Suite 239; Georgetown, SC 29440				
Boring #:	CPT-3	Sample #:	Bulk	Sample Date:	5/9/2016
Location:	Proposed Pond	Lab #:	3842	Depth:	1'-2'
Sample Description:	Brown Silty Sand (SM)				

Maximum Dry Density 109.6 PCF. Optimum Moisture Content 9.2%

ASTM D1557 -- Method A



Soil Properties

Natural Moisture Content	1.6%
Specific Gravity of Soil	--
Liquid Limit	--
Plastic Limit	NP
Plastic Index	--

% Passing

3/4"	100.0%
3/8"	1000.0%
#4	100.0%
#20	100.0%
#40	84.5%
#60	62.2%
#200	20.2%

Oversize Fraction

Bulk Gravity	
% Moisture	--
% Oversize	--
MDD	
Opt. MC	

Moisture-Density Curve Displayed: Fine Fraction ☒ Corrected for Oversize Fraction (ASTM D 4718) ☐
Sieve Size used to separate the Oversize Fraction: #4 Sieve ☒ 3/8 inch Sieve ☐ 3/4 inch Sieve ☐
Mechanical Rammer ☒ Manual Rammer ☐ Moist Preparation ☐ Dry Preparation ☒

References / Comments / Deviations:

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 1557: Laboratory Compaction Characteristics of Soil Using Modified Effort

Chelsea Jones		Staff Professional	5/23/16
Technical Responsibility	Signature	Position	Date

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Sieve Analysis of Soils

ASTM D 422

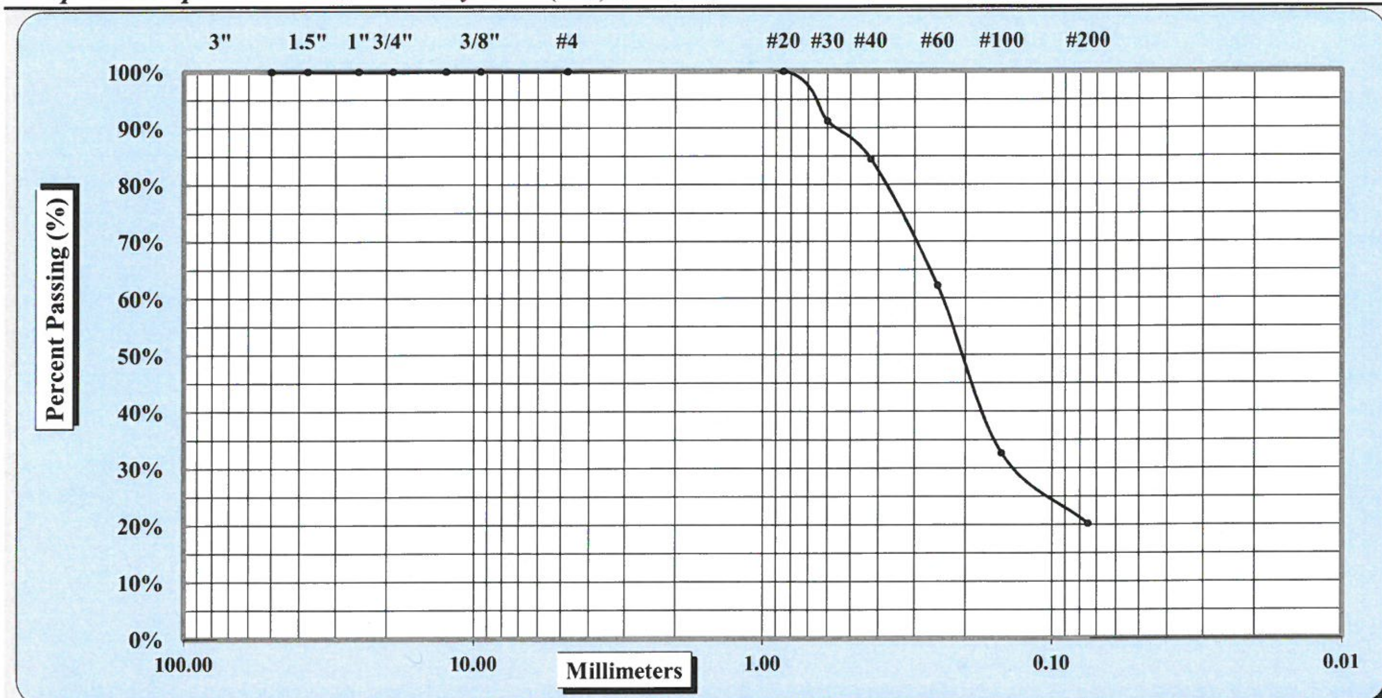
Quality Assurance

S&ME, Inc. - Myrtle Beach 1330 Highway 501 Business; Conway, SC 29526

Project #:	**1463-16-019**	Report Date:	5/17/2016
Project Name:	New Big Dam Swamp Fire Station #15	Test Date(s):	5/16/2016
Client Name:	Georgetown County		
Client Address:	129 Screven Street, Suite 239; Georgetown, SC 29440		

Boring #	CPT-3	Sample #:	Bulk	Sample Date:	5/9/2016
Location:	Proposed Pond	Lab #:	3842	Depth:	1'-2'

Sample Description: Brown Silty Sand (SM)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	0.850 mm	Coarse Sand	0.0%	Fine Sand	64.3%
Gravel	0.0%	Medium Sand	15.5%	Silt & Clay	20.2%
Liquid Limit	--	Plastic Limit	NP	Plastic Index	--
Specific Gravity	--			Moisture Content	1.6%
Coarse Sand	0.0%	Medium Sand	15.5%	Fine Sand	64.3%

Description of Sand & Gravel Particles: Rounded ☐ Angular ☒
Hard & Durable ☒ Soft ☐ Weathered & Friable ☐

Notes / Deviations / References:

Chelsea Jones		Staff Professional	5/23/16
Technical Responsibility	*Signature*	*Position*	*Date*

Form No: TR-D422-WH-1
Revision No. 0
Revision Date: 07/14/08

Sieve Analysis of Soils

S&ME

ASTM D 422

Quality Assurance

S&ME, Inc. - Myrtle Beach 1330 Highway 501 Business; Conway, SC 29526

Project #:	**1463-16-019**	Report Date:	5/17/2016
Project Name:	New Big Dam Swamp Fire Station #15	Test Date(s):	5/16/2016
Client Name:	Georgetown County		
Client Address:	129 Screven Street, Suite 239; Georgetown, SC 29440		

Boring #	CPT-3	Sample #:	Bulk	Sample Date:	5/9/2016
Location:	Proposed Pond	Lab #:	3842	Depth:	1'-2'

Sample Description: Brown Silty Sand (SM)

Description of Sand & Gravel Particles: Rounded ☐ Angular ☒
Hard & Durable ☒ Soft ☐ Weathered & Friable ☐

Particle Size Analysis / Without Hydrometer Analysis				*Material Excluded:*	
Tare No.	**H**	Tare Wt.	**83.0**	Mass of Sample after Wash + Tare Wt.	**152.5**
Total Sample Wet Wt. + Tare Wt.			**171.0**	Mass of Sample after Wash	**69.5**
Total Sample Dry Wt. + Tare Wt.			**169.6**	Mass passing #200	**17.1**
Total Sample Dry Weight			**86.6**	% Passing #200 **(D1140)**	**19.7%**

Sieve Size		**Retained Weight**	**% Retained Between Sieves**	**% Retained**	**% Passing**	**SPECS**
				Cumulative		
Standard	*mm.*	***Cumulative***	*Individual*	***Total Sample***		
2.0"	50.00	0.0	0.0%	0.0%	**100.0%**	
1.5"	37.50	0.0	0.0%	0.0%	**100.0%**	
1.0"	25.00	0.0	0.0%	0.0%	**100.0%**	
3/4"	19.00	0.0	0.0%	0.0%	**100.0%**	
1/2"	12.50	0.0	0.0%	0.0%	**100.0%**	
3/8"	9.50	0.0	0.0%	0.0%	**100.0%**	
#4	4.75	0.0	0.0%	0.0%	**100.0%**	
#20	0.850	0.0	0.0%	0.0%	**100.0%**	
#30	0.600	7.6	8.8%	8.8%	**91.2%**	
#40	0.425	13.4	6.7%	15.5%	**84.5%**	
#60	0.250	32.7	22.3%	37.8%	**62.2%**	
#100	0.150	58.3	29.6%	67.3%	**32.7%**	
#200	0.075	69.1	12.5%	79.8%	**20.2%**	
Pan	<0.075	69.5		**% Passing #200 (D1140) =**		**20.2%**

D2487	Maximum Particle Size	**0.850 mm**	Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**15.5%**
Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**	Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**64.3%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.0%**	% Silt & Clay	< 0.075 mm	**20.2%**

Notes / Deviations / References:

Chelsea Jones		Staff Professional	5/23/16
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D4318-T89-90
Revision No. 0
Revision Date: 11/20/07

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

ASTM D 4318 ☒ *AASHTO T 89* ☐ *AASHTO T 90* ☐ ***Quality Assurance***

S&ME, Inc. Myrtle Beach 1330 Highway 501 Business; Conway, SC 29526

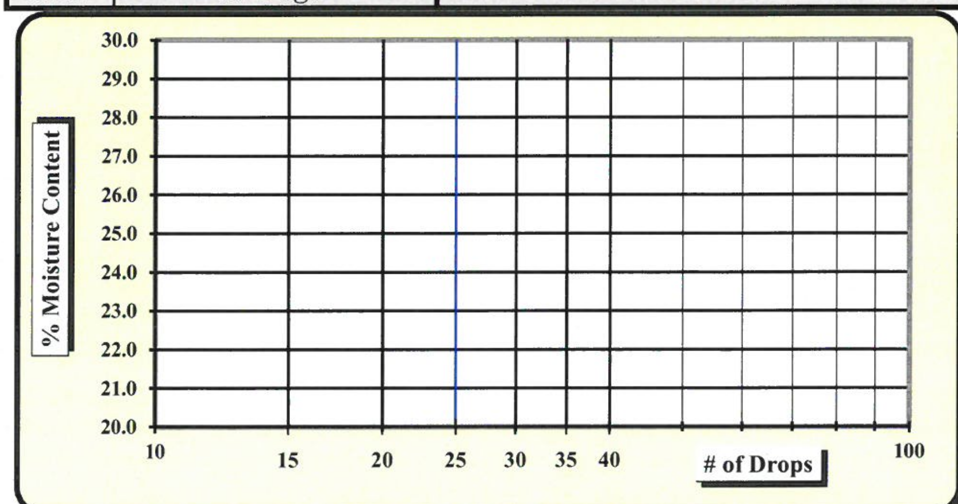
Project #:	**1463-16-019**	Report Date:	5/17/2016
Project Name:	New Big Dam Swamp Fire Station #15	Test Date(s)	5/12/2016
Client Name:	Georgetown County		
Client Address:	129 Screven Street, Suite 239; Georgetown, SC 29440		

Boring #:	CPT-3	Sample #:	Bulk	Sample Date:	5/9/2016
Location:	Proposed Pond	Lab #:	3842	Depth:	1'-2'

Sample Description: Brown Silty Sand (SM)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	00401	2/18/2015	Grooving tool	11368	5/1/2015
LL Apparatus	18801	5/1/2015			
Oven	17745	5/6/2015			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	1	2	3	4	5	6	7	8	9
A	Tare Weight									
B	Wet Soil Weight + A								NP	
C	Dry Soil Weight + A									
D	Water Weight (B-C)									
E	Dry Soil Weight (C-A)									
F	% Moisture (D/E)*100									
N	# OF DROPS							*Moisture Contents determined by ASTM D 2216*		
LL	LL = **F** * FACTOR									
Ave.	*Average*									



One Point Liquid Limit			
N	**Factor**	**N**	**Factor**
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.99	29	1.018
24	0.995	30	1.022
25	1.000		

NP, Non-Plastic	☒
Liquid Limit	
Plastic Limit	**NP**
Plastic Index	
Group Symbol	**SM**
Multipoint Method	☐
One-point Method	☑

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑

Notes / Deviations / References:

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

Chelsea Jones	[signature]	Staff Professional	5/23/16
Technical Responsibility	*Signature*	*Position*	*Date*