Report of Geotechnical Exploration
Waccamaw Elementary Athletic Fields
Pawleys Island, South Carolina
S&ME Project No. 1363-20-017

PREPARED FOR:

Georgetown County Recreation & Community Services
2030 Church Street
Georgetown, South Carolina 29440

PREPARED BY:

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

May 18, 2020

May 18, 2020

Georgetown County Recreation & Community Services
2030 Church Street
Georgetown, South Carolina 29440

Attention: Ms. Beth Goodale

Reference: **Report of Geotechnical Exploration**
Waccamaw Elementary Athletic Fields
Pawleys Island, South Carolina
S&ME Project No. 1363-20-017

Dear Ms. Goodale:

S&ME, Inc. has completed the subsurface exploration for the referenced project after contract execution on April 27, 2020. Our exploration was conducted in general accordance with our Proposal No. 13-2000184, dated April 16, 2020.

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, pavement section thickness and infiltration data from the site. 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. C00473

Kara Fugate, E.I.T.
Staff Professional

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 21248 RON FOREST JR. 5/18/20

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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:** The soil profile generally consists of very loose to dense sands and sand mixtures to a depth of 18 feet (Stratum I). Beneath the Stratum I sands and sand mixtures, soft to stiff silt and clay mixtures were encountered to a depth of approximately 23 ½ feet (Stratum II). Beneath the silty and clayey soils of Stratum II, medium dense to very dense sands were encountered to the maximum exploration depth of 40.1 feet beneath the existing ground surface (Stratum III). Hand auger borings with penetration testing conducted in the proposed parking lot areas indicated generally very loose to loose sands and sand mixtures at the surface at depths of 1 to 4 feet.

   - Tests indicate that the near-surface soils were about 4 percent wet of their optimum moisture content for compaction at the time of our exploration, indicating that some drying of these soils may be required in order to achieve proper compaction.
   - Organic materials were observed in boring HA-1, in the proposed future parking lot area, to a depth of about 1 ¾ feet below the surface. This indicates the potential need to remove and replace some of the upper soils in the parking lot where organics are observed near the surface. The potential volume or removal and replacement could be better quantified by excavating some small test pits around the future parking lot area to better define the depth and lateral extent of the organic materials prior to construction.

2. **Subsurface Water:** At the time of drilling, the CPT sounding (C-8) interpreted the water level to be approximately 6 feet below the existing ground surface. Water was not encountered within hand auger borings HA-3 through HA-8 which were advanced to a depth of 4 feet below the existing ground surface. Water was observed at a depth of 3 ½ feet within hand auger boring HA-1 and at a depth of 1 ½ feet within hand auger boring HA-2, both of which are located on the eastern side of the site. Infiltration testing was performed at four locations within the proposed ball fields, and infiltration rates ranging from 0.2 to 1.8 inches per hour were measured in the upper sandy soils, with 0.2 to 0.3 iph rates occurring in the eastern portion of the site, and 1.7 to 1.8 iph rates occurring in the western portion of the site.
3. **Seismic Site Class and Liquefaction:** Liquefaction of sands during the code design earthquake does not appear likely to occur on a widespread basis within the subsurface soils encountered at the test locations. In our opinion, there is relatively low risk of seismic hazards such as significant bearing capacity loss, lateral spreading, generalized shear failure, and volumetric settlement of the bearing soils under seismic shaking associated with the Code-level earthquake; therefore, Site Class F does not apply. Based on the average measured shear wave velocities of 647 feet per second (fps), and the extrapolated value of 740 fps to a depth of 100 feet, the site classifies as Seismic Site Class D.
4. **Seismic Design Parameters:** The following seismic design parameters apply: S_{DS} = 0.39g, S_{D1} = 0.21g, and Peak Ground Acceleration (PGA_M) = 0.30g. For a structure having a seismic use group classification of I, II, or III, the S_{DS} and S_{D1} values obtained are consistent with Seismic Design Category D as defined in section 1613 of the IBC, 2018 edition and ASCE 7-16.

5. **Shallow Foundations:** Considering the assumed structural loads, and our estimated static settlements, a shallow foundation system appears feasible for support of the buildings assuming that the fill and compaction section of this report are followed.

    - Considering the assumed structural loads, we recommend an allowable net bearing capacity of up to 2,500 pounds per square foot (psf) for design of isolated shallow spread footings for columns and walls. An assumed uniformly applied area load (fill + slab loading + slab self-weight) of 300 psf and a maximum column load of 30 kips were used in the calculations.
    - The estimated total static settlement of a minimum 3 ½ ft. by 3 ½ ft. column footing under a 30 kip load is approximately ¾ inch or less. Differential static settlements between similarly loaded columns are anticipated to be ½ inch or less.
    - Based on an assumed wall load of 4 kips/foot, and considering a uniformly applied area load of 300 psf, and a 2,500 psf shallow foundation bearing pressure, the estimated static post-construction settlement of an individual wall strip footing will likely be ¾ inch or less. Differential settlements between adjacent, similarly loaded walls are anticipated to be ½ inch or less.
    - Because of the very loose condition of the upper sands at some locations, it is likely that the foundation bearing surfaces may need to be densified in place by vibratory compaction methods after the footings have been excavated in order to provide suitable bearing conditions.

6. **Grade Slabs:** Grade slabs may be soil-supported if the site soils are improved as recommended herein, and a modulus of subgrade reaction (k) of 175 pci may be used for slab reinforcing design. We recommend at least a 4 inch thick layer of granular material containing less than 5 percent fines passing the No. 200 sieve be placed immediately beneath the slabs wherever the natural soils do not meet this criteria as determined by ASTM D 1140.
7. **Pavements:** We evaluated these sections considering the existing soils and the recommended soil preparation measures described in this report, and generated a theoretical available traffic capacity.

    - For standard-duty flexible pavements (passenger cars only; no heavy truck or bus traffic), a pavement section consisting of 2 inches of SCDOT Type C surface course hot mixed asphalt (HMA) over 6 inches of compacted graded aggregate base course (GABC) is the recommended minimum. The theoretical available traffic capacity is approximately 70,000 Equivalent 18-kip Single Axle Loads (ESALs) for this pavement section.
    - For heavy-duty flexible pavements (subject to any heavy truck or bus traffic), a pavement section consisting of 1 ½ inches of SCDOT Type C surface course HMA over 1 ½ inches of Type C intermediate course HMA over 6 inches of compacted GABC is the recommended minimum. The theoretical available traffic capacity is approximately 285,000 ESALs for this pavement section.
    - The estimated service life for the asphalt pavements is 10 to 15 years before an overlay is required.
    - For heavy-duty rigid pavement areas subject to truck or bus traffic, and dumpster pads, we recommend a 4,000 psi compressive strength Portland cement concrete thickness of 6 inches, overlying a compacted graded aggregate base course thickness of 6 inches. The theoretical available traffic capacity is approximately 400,000 ESALs for this pavement section. The estimated service life for concrete pavements is typically 15 to 20 years.

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 grading, foundation design for the proposed building, pavements, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration, and discusses our conclusions and recommendations.

A site vicinity map and test location plan showing the approximate test locations are included in Appendix I. The CPT sounding logs and hand auger boring logs, a discussion of the field exploration procedures, and a legend to soil classification and symbols are included in Appendix II. The laboratory test procedures and laboratory test results are included in Appendix III.

1.1 Project Information

Project information was provided via email correspondence between Patrick Williams (SGA Narmour Wright Design) and Worth King (S&ME) on April 16, 2020. The email included a Conceptual Master Plan prepared by SGA and dated August 12, 2010, which depicted requested test locations.

1.2 Site Description

The site is located at the existing Waccamaw Elementary School at 1364 Waverly Road in Pawleys Island, South Carolina. The area explored is the northern portion of the parcel, which is currently partially cleared, and partially wooded. The site also has two ponds on the western side and two existing baseball fields on the eastern side. A site vicinity map is included in Appendix I as Figure 1.

1.3 Project Description

We understand that development will include one new press box, four new athletic fields, and two new paved parking lot areas. We anticipate that the press box construction will consist of reinforced concrete masonry unit (CMU) walls and a cast-in-place, soil-supported, concrete slab-on-grade with shallow spread footing foundations.

1.4 Design Data

1.4.1 Structural Loading Information

We anticipate that the building column and wall loads will not exceed 30 kips and 4 kips per linear foot, respectively. We assume that future site grade elevations will likely remain near existing elevations with cut or fill thicknesses of 2 feet or less. Our assumptions should be confirmed or modified by the client or design team prior to the completion of our work.

1.4.2 Traffic Loading

Traffic trip frequencies and vehicle types were not provided for analysis. Therefore, for the pavement section thickness calculations, we assumed a life term of 20 years and calculated the approximate 18-kip Equivalent Single Axle Loads (ESALs) that would be available for the pavement sections recommended, using estimated trip frequencies and vehicle types. Actual traffic loading may vary from these assumptions. It is important to recognize that the standard-duty traffic assumption includes zero truck or bus traffic. It is assumed that the heavy-duty section will be used in all areas subjected to truck traffic.

1.4.3 Static Settlement Tolerances

In the absence of any special settlement tolerances for the proposed structure, we assumed the tolerance for total post-construction static settlement magnitude to be 1 inch, and the tolerance for differential post-construction static settlement to be ½ inch.

1.4.4 Grade Elevation Changes

Finished floor elevation was not provided at the time of this report but it is assumed for the purposes of our geotechnical analyses that final grade elevations may increase about 1 foot above existing grades.

2.0 Exploration Procedures

2.1 Field Exploration

On April 29 and May 7, 2020, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

1. We performed a site walkover, observing features of topography, ground cover, and surface soils at the project site.
2. Our test locations were assigned at the time of our proposal and are shown on Figure 2 attached to this report.
3. Within the proposed press box building pad, we advanced one Seismic Cone Penetration Test (SCPT) sounding (C-8) to a tip refusal depth of approximately 40.1 feet.
4. In conjunction with the SCPT sounding at location C-8, shear wave velocity measurements were recorded at approximate 1 meter depth intervals[1].
5. We performed a total of eight (8) hand auger borings at test locations HA-1 through HA-8.

    A. Hand auger borings at test locations HA-1, HA-2 and HA-7 were each advanced to a depth of 4 feet and conventional dynamic cone penetrometer (DCP) testing was conducted within the borings. DCP testing was performed at regular depth intervals of approximately 1 foot each within these hand auger borings in general accordance with ASTM STP 399 procedures to help us estimate the relative density and consistency of the subgrade soils. A composite bulk sample was collected from the soils recovered from the hand auger borings. Boring HA-7 was advanced within the native soils not the existing gravel roadway of Cochran Road.

[1] We returned to the site on May 15, 2020 and re-performed the seismic shear wave velocity testing due to a data collection technical difficulty issue that occurred during the May 7, 2020 drilling event.

B. The hand auger borings advanced at test locations HA-3 through HA-6 were initially advanced to a depth of approximately 12 inches. These borings were then used to measure the in-situ K_{sat} values of the soils using the constant-head well permeameter technique. The hand auger borings were then advanced to a depth of 4 feet below the surface after the infiltration test was complete. The web soil survey was used to generate a soil map of the area and this is attached as Figure 3 in Appendix I.
C. The hand auger boring advanced at test location HA-8 was advanced to a depth of 4 feet without penetration testing.
D. Small grab samples of subsurface soil materials were collected from representative subsurface strata within the borings. Within the borings, our engineer observed and documented the subgrade soil types observed.

A brief description of the field exploration procedures performed, as well as a soil classification legend, the SCPT/CPT sounding logs and hand auger boring logs are attached in Appendix II. The shear wave velocity profile is included as Figure 4 in Appendix II.

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

The western portion of the site has grooves varying in depth from a few inches to approximately 2 feet, likely from clearing equipment. The area of the site, to the east, that is currently baseball fields is relatively flat. Standing water was observed on the western portion of the site. It was beyond the scope of our work to perform a survey of the site. For purposes of the boring and sounding logs, the ground surface was set to zero.

3.2 Topsoil

Where encountered, topsoil thickness ranged from 4 inches to 1 foot in thickness. At hand auger borings locations HA-5, HA-6 and HA-8, no topsoil was encountered, but leaves, pinestraw and other vegetation lay on the surface of the ground. Topsoil and rootmat thickness may vary at locations not explored.

3.3 Possible Fill and Organics

At test location HA-1, within the existing ball field, a layer of sand approximately 6 inches thick was encountered beneath the topsoil. Under this poorly graded sand (USCS Classification “SP”), a layer of clayey sand (SC) with some organic material was encountered to a depth of 1 ¾ feet below the surface. This clayey sand was similar to the soil observed near the surface in hand auger boring HA-4.

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 test 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: Upper Very Loose to Dense Sands

Within each of the hand auger borings, poorly graded sand (SP), clayey sand (SC), poorly graded sand with silt (SP-SM) and poorly graded sand with clay (SP-SC) were encountered to the termination depths of 4 feet. These soils were shades of brown and gray, and were moist to wet upon recovery. Where DCPs were performed, this stratum indicated a very loose to loose relative density with penetration resistance values ranging from 1 blow per increment (bpi) to 8 bpi. All eight of the hand auger borings advanced on site terminated at a depth of 4 feet within this stratum.

This sandy layer appears to continue to a depth of about 18 feet below the surface based on the cone sounding data collected from the site at test location C-8. Where encountered in the CPT sounding, the tip resistances ranged from 35 tons per square foot (tsf) to 150 tsf, but typically ranged from 50 tsf to 125 tsf. This indicates a typically medium dense to dense relative density with a few very loose to loose zones. The loose zones occurred in the upper few feet near the ground surface and between depths of about 12 and 18 feet. The medium dense to dense zones occurred between depths of about 4 and 12 feet, with the densest material being observed at a depth of about 6 to 7 feet.

A composite bulk sample of these sands was collected from the proposed pavement areas and transported to our laboratory for testing. The sample was classified as a clayey sand (SC) with a fines content of 20.6 percent passing the No. 200 sieve, a liquid limit of 24, a plastic limit of 11, and a plasticity index of 13. The natural moisture of the bulk sample was measured to be 13.6 percent, which is 3.8 percent wet of the optimum moisture content for compaction of 9.8 percent that was determined by a modified Proctor test. The maximum dry density of the sample was 124.2 pounds per cubic foot. The California Bearing Ratio of this soil when recompacted to about 95 percent of the modified Proctor maximum dry density at optimum moisture content was measured to be 37.6 percent at 0.1 inch of penetration.

4.1.2 Stratum II: Intermediate Soft to Stiff Silt and Clay Mixtures

Underlying Stratum I, silt and clay mixtures were encountered beginning at a depth of about 18 feet and continuing to a depth of about 23 ½ feet. The CPT penetration tip resistance values typically ranged from about 5 tsf to 30 tsf, indicating a typically soft to stiff consistency.

4.1.3 *Stratum III: Lower Medium Dense to Very Dense Sands*

Beneath the intermediate silts and clays of Stratum II, a lower layer of sands was encountered to the termination depth of sounding C-8 at a depth of 40.1 feet. The tip resistances in this stratum typically ranged from 80 tsf to greater than 200 tsf, indicating a medium dense to very dense relative density. There was one thin lens of soft clay observed at a depth of about 29 feet, but it appeared to be less than 1 foot thick.

4.2 Subsurface Water

The subsurface water level was interpreted at a depth of about 6 feet below the ground surface at the time of the exploration, based upon measurements performed within the CPT sounding. Water was not observed in hand auger borings HA-3 through HA-8 at the time of drilling to a depth of 4 feet. Water was observed in boring HA-1 at a depth of 3 ½ feet and in boring HA-2 at a depth of 1 ½ feet. These readings likely represent perched groundwater conditions rather than a stable water table, and appear to be affecting the eastern portion of the site more than the western portion of the site. Subsurface water levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence water elevations.

4.3 Measured Infiltration Rates

We conducted four infiltration tests at depths of 12 inches below the existing ground surface at test locations HA-3 through HA-6 (see the attached Figure 2 of Appendix I) as requested. We conducted each infiltration test using the constant-head well method to measure the K_{sat} values of the in-situ soils. This procedure is described in Methods of Soil Analysis, Part 1, Chapter 29 – Hydraulic Conductivity of Saturated Soils: Field Methods, 29 – 3.2 Shallow Well Pump In Method, pp. 758-763 and in the Soil Science Society of America Journal, Vol. 53, no. 5, Sept. – Oct. 1989, "A Constant-head Permeameter for Measuring Saturated Hydraulic Conductivity of the Vadose Zone" and "Comparison of the Glover Solution with the Simultaneous – Equations Approach for Measuring Hydraulic Conductivity."

The stabilized (saturated) infiltration rate measured was approximately 0.3 inches per hour (iph) at test location HA-3, approximately 0.2 iph at test location HA-4, approximately 1.8 iph at test location HA-5, and approximately 1.7 iph at test location HA-6. The individual test result worksheets are presented in Appendix II.

4.3.1 *SCS Soil Series (Near-Surface Soils)*

We have reviewed the USDA Soil Conservation Service (SCS) map for the site, and have identified three soil series within the project boundaries:

- **Leon Fine Sand (Le):** Approximately thirty-five percent of the site is within the Leon sand soil series, predominately in the eastern portion of the site. These soils are poorly drained, and have a low storage capacity, but are not typically subjected to flooding or ponding of water. This series typically consists of sands to a depth of 80 inches. Infiltration rates are reported by the USDA as typically ranging from 0.06 to 2.0 inches per hour.
- **Echaw Sand (Ec):** Approximately fifty percent of the site is within the Echaw sand soil series, predominately in the western portion of the site. These soils typically consist of sand to a depth of 60 inches. These soils are moderately well drained, and have a high to very high storage capacity; they are

not typically subject to flooding or ponding. Infiltration rates are reported by the USDA as typically being in the range of 2.0 to 20.0 inches per hour. Our infiltration tests were performed within this series and our measured infiltration rates ranged from 0.2 iph to 1.8 iph at a depth of 12 to 18 inches with 0.2 to 0.3 iph rates occurring in the eastern portion of the site, and 1.7 to 1.8 iph rates occurring in the western portion of the site. This is somewhat slower than the USDA reported range of typical values for Echaw Sand.

- **Yauhannah Loamy Fine Sand (Ya):** Approximately 15 percent of the site is within the Yauhannah Loamy Fine sand soil series, predominately in the northeastern corner of the site. This soil series typically consists of a loamy fine sand to sandy clay loam in the upper 75 inches. These soils are moderately well drained, have a moderate storage capacity, and are not prone to ponding or flooding. Infiltration rates are reported by the USDA as typically being in the range of 0.6 to 2.0 inches per hour.

Please see Figure 3 in Appendix I for a site sketch that shows a graphic representation of where each of these soil survey groups is estimated to be located on the site.

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 Building Code Seismic Provisions

As of January 1, 2020, the 2018 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, using the procedures described in Chapter 20 of ASCE 7-16.

The initial step in site class definition is to 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 include the following: 1) quick and highly sensitive clays or collapsible weakly cemented soils, 2) peats and highly organic clays, 3) very high plasticity clays, and 4) very thick soft/medium stiff clays. These soils were not evident in the soundings.

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 liquefaction of subsoils appears unlikely to occur on a widespread basis at this site in the event of the design magnitude earthquake; therefore, Site Class F does not apply.

5.1.1 *Liquefaction of Bearing Soils*

Liquefaction of saturated, loose, cohesionless soils occurs when they are subjected to earthquake loading that causes the pore pressures to increase and the effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results. Earthquake-induced ground surface acceleration at the site was assumed from the building code design peak ground acceleration of 0.30g.

5.1.2 *Liquefaction Potential Index (LPI)*

To evaluate liquefaction potential, we performed analyses using the data obtained in the borings, considering the characteristics of the soil and water levels observed in the boring. The liquefaction analysis was performed based on the design earthquake prescribed by the 2018 edition of the International Building Code, 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).

To help evaluate the consequences of liquefaction, we have 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 ≤ LPI ≤ 15*** – moderate liquefaction with some surface manifestation possible.
- ***LPI > 15*** – severe liquefaction and foundation damage is likely.

The LPI for this site was calculated to be 5 or less; therefore, the risk of surface damage due to liquefaction under seismic shaking is generally low. The primary risk of liquefaction at this site is related to minor magnitudes of surface settlement; surface venting such as sand boils are not expected to occur. We therefore consider that Site Class F conditions do not reasonably apply to this site.

5.2 Selection of Seismic Site Class based on Shear Wave Velocity

Based upon the measured and extrapolated shear wave velocities, this site is determined to be Site Class D. This recommendation is provided based on the shear wave velocity measured at test location C-8 to a depth of 47.6 feet and then extrapolated to a depth of 100 feet. The average weighted shear wave velocity was measured to be 647 feet per second (fps) in the upper 47.6 feet. When extrapolated to a depth of 100 feet, an average shear wave velocity of 740 fps is estimated, which is greater than the 600 fps that is required for consideration of Site Class D design parameters. See Figure 4 in Appendix II for a graph of the shear wave velocity profile.

5.3 Seismic Spectral Design Values

Site Class D parameters are appropriate to determine the seismic site response, and the spectral accelerations and site coefficients for the site are given below in Table 5-1.

Table 5-1: Seismic Design Coefficients

Criteria	Site Class	S_S	S_1	S_{DS}	S_{D1}	PGA_M	Seismic Design Category
2018 IBC	D	0.40	0.14	0.39	0.21	0.30	D

5.4 Seismic Design Category

For a structure having a Risk Category classification of I, II, or III, the S_{DS} and S_{D1} values obtained are consistent with "Seismic Design Category D" as defined in section 1613 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 locations of the structure or pavements changed, or if conditions are encountered during construction that differ from those encountered in the borings, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

Based upon the results of our exploration and our past experience with similar soils in the site vicinity, the site appears generally adaptable for the proposed development. Based on the assumed loading and settlement tolerances, it appears feasible that the structure can be supported on shallow foundation systems with implementation of good drainage and some near-surface ground improvement procedures to dry the soils and remove some organic materials.

6.1 Site Preparation

The following surface preparation recommendations are provided for this site.

1. Drainage should be implemented and maintained as soon as possible prior to construction. Surface and subsurface water conditions at the time of construction, largely influenced by prevailing weather patterns, will determine the need for and extent of drainage measures. Water conditions can change with construction activities and precipitation effects.
2. Strip surface vegetation, root mat, and organic-laden or debris-laden soils where encountered and dispose of outside the building and pavement area footprints.

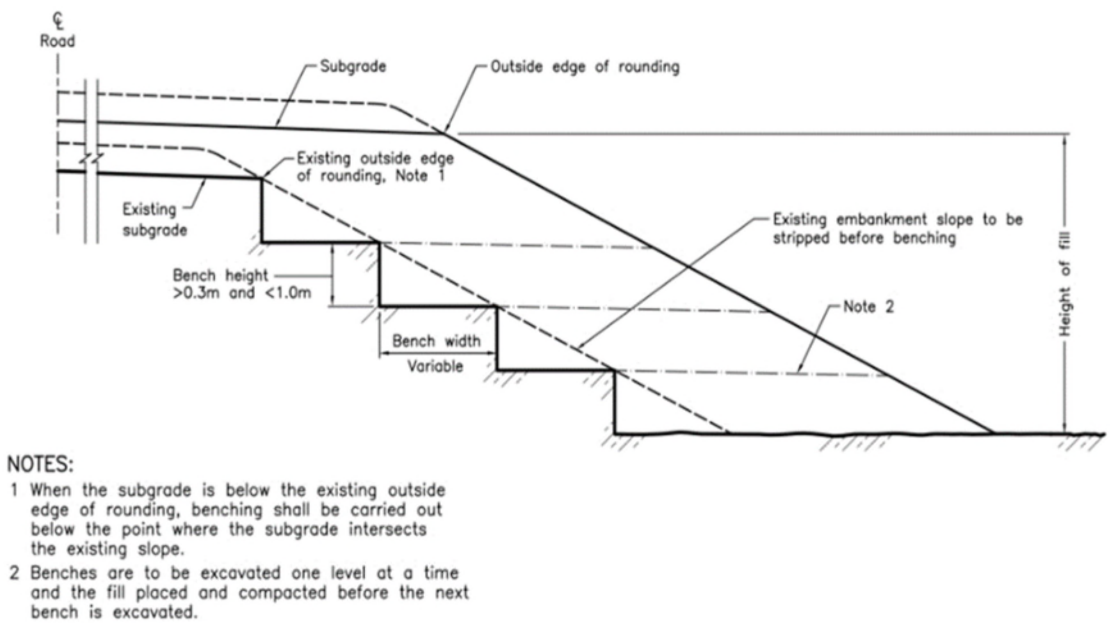
    A. In the future parking lot area, this may include the upper 1 ¾ feet of soil within the existing ball field where buried organics were observed in boring HA-1. The potential volume of material that needs to be removed and replaced could be better quantified by excavating some small test pits around the future parking lot area to better define the depth and lateral extent of the organic materials prior to construction.

3. The cut subgrade in all areas to receive fill should be proofrolled by the contractor under the observation of a representative of the Geotechnical Engineer to observe for stability prior to fill placement. Where needed, based on the results of the proofroll, it may become necessary to perform undercutting and replacement of unstable soils. This is not expected to be a widespread condition at this site. This should be a decision made at the time of construction based on the conditions observed.

6.1.1 *Ditch and Pond Filling*

The ditch that cuts into the site will need to be mucked of all soft sediments and rip-rap prior to fill placement. After dewatering, the side slopes of any ponds or ditches to be backfilled must also be properly benched to accommodate the placement of new fill in horizontal lifts. Fill placed within these areas should be notched into the embankment using a benching procedure as shown in Figure 6-1 below, and the fill lifts shall be placed horizontally into the benches or notches. It is not recommended to place the fill in diagonal lifts parallel to the embankment slope, because this method decreases the stability of the fill and could create a slip plane.

Figure 6-1: Example Benching Diagram for Slopes <3H:1V



Final fill placement limits should take into consideration that all fill located within 5 feet laterally of the building and pavement area footprints must be compacted throughout to at least 95 percent of the modified Proctor maximum dry density, as described in Section 6.2 below.

- Once prepared, have a representative of the Geotechnical Engineer observe all pond and ditch excavations prior to backfilling, to confirm that they are in a suitable condition to receive new fill.

6.2 Fill Placement and Compaction

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 grain size and plasticity characteristics, maximum dry density, optimum moisture content, natural moisture content and pavement support characteristics.

    A. Fill soils to be used as structural fill should meet the following minimum requirements: plasticity index of 15 percent or less; clay/silt fines content of not greater than 25 percent; natural moisture content within 3 percent below to 3 percent above the optimum moisture content for compaction as determined by laboratory Proctor testing.
    B. This may include soils from the following ASTM soil classifications: SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, and/or SC. Not all soils in these categories will comply with the plasticity and fines content requirements; therefore, the contractor should sample each fill material that they propose to use and submit it to the Geotechnical Engineer for determination of its suitability, and measurement of the maximum dry density, optimum moisture content, and natural moisture content.
    C. Where native soils are borrowed and repurposed as fill, the material may need to be dried somewhat to reduce its moisture content to the compactable range.

2. Where fill soil is required under building and pavement areas, structural fill should be compacted throughout to at least 95 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 12 inches thick prior to compaction. Structural fill should extend at least 5 feet beyond the edge of the structures and at least 5 feet beyond the edge of the pavements before being allowed to exhibit a lower level of compaction.
3. In non-structural fill areas only, such as in landscaped areas that are located at least 5 feet outside the footprint of pavements and the footprint of buildings, fill should be compacted to at least 90 percent of the maximum dry density by the modified Proctor criterion (ASTM D 1557).
4. Subsurface water levels should be maintained at least 2 feet below any surface to be densified prior to beginning compaction. This is to prevent the compaction operation from drawing water up to the surface and degrading it.
5. 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 5,000 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 Foundations

The soil profile of the site appears generally suitable to support the proposed building with shallow foundations considering static loading conditions and the assumed maximum column and wall loads. The design engineer needs to confirm that the assumed maximum loads are correct; if actual loads are higher, we should be notified and given a reasonable opportunity to reconsider these recommendations, because it could result in changes to the estimated available bearing capacity and static settlement magnitudes.

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

1. Provided that the recommendations in sections 6.1 through 6.2 of this report are implemented, a net available bearing pressure of up to 2,500 pounds per square foot (psf) may be used for design of individual spread footings and wall footings that are extended to bear upon structural fill compacted as recommended in the fill placement and compaction recommendations section of this report.
2. Lateral capacity of foundations includes a soil lateral pressure and coefficient of friction as described in IBC Section 1806. Assuming that the footings will bear within the compacted fill or compacted similar native surface sands, the foundations will be embedded in material similar to those described as Class 4 in Table 1806.2. Where footings are cast neat against the sides of excavations in natural soils, an allowable bearing pressure of 150 psf per foot depth below natural grade may be used in computations. An allowable coefficient of friction of 0.36, multiplied by the dead load, may be used for computation of sliding resistance. An increase of one-third in the allowable lateral capacity may be considered for load combinations, including wind and earthquake, as permitted by IBC Section 1605.3.2, unless otherwise restricted by design code provisions.
3. Have the Geotechnical Engineer's (S&ME) representative observe each cleaned footing excavation prior to concrete placement to observe that the required degree of soil compaction and bearing capacity is present at the foundation bearing surface.
4. The need for overexcavation in the footing excavations should be a field decision made by the Geotechnical Engineer at the time of construction, using DCP test data, in conjunction with shallow hand auger borings advanced within the footing excavations, to evaluate the consistency of the soils.

   A. In the event that overexcavation of footings is required, S&ME should be present at the site to observe conditions, confirm that poor soils have been removed, and observe that the overexcavated footings are properly backfilled.
   B. Where overexcavation is performed, foundation bearing grades should be reestablished using washed, crushed gravel (such as SCDOT No. 57 stone) placed in densified 12-inch thick lifts. Each footing excavation should be observed and tested for suitability to support the design bearing pressure.

5. 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.
6. Even if smaller dimensions are theoretically allowable from a bearing pressure consideration, the minimum column footing width should be 24 inches and the minimum wall footing width should be 18 inches, to avoid punching shear. Spread footings should be embedded to a minimum depth of 12 inches or to the depth indicated on the drawings, whichever is greater.
7. Footing concrete should be placed the same day that footings are excavated to reduce the potential for exposed bearing soils to be softened due to factors such as weathering or water infiltration.
8. The following discussion is provided regarding the estimated magnitude of settlements under static loading.

A. Based on an assumed maximum column load of 30 kips, and considering a uniformly applied area load (fill + slab loading + slab self-weight) of 300 psf, and a 2,500 psf shallow foundation bearing pressure, the estimated total static settlement of an individual spread footing measuring roughly 3 ½ feet by 3 ½ feet in plan area will likely be ¾ inch or less.
B. Based on an assumed wall load of 4 kips per linear foot, and considering a uniformly applied area load of 300 psf, and a 2,500 psf shallow foundation bearing pressure, the estimated static post-construction settlement of an individual wall strip footing at least 1 ½ feet wide will likely be ¾ inch or less.
C. Differential settlements between adjacent, similarly loaded walls and columns are typically on the order of 50 percent of the total post-construction settlement value under static loading, or in this case, ½ 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 recommended as fill material or the native near surface materials should 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 175 lbs/in^3 may be used for reinforcing design.
2. Structural design should incorporate installation of a vapor barrier prior to placing concrete for grade slab systems, to limit moisture-infiltration into finished spaces, where appropriate.
3. Below the floor slab place a layer of at least 4 inches of compacted granular materials to provide a capillary break between the native soils and the floor slab in finished spaces.

   A. Granular materials used may consist of a clean sand, classifying as USCS type SP or SW and having less than 5 percent silt/clay fines by weight passing the No. 200 sieve when tested by ASTM D1140, or 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.
   B. Native onsite sands may be used as the capillary break layer if they meet the material requirements.
   C. If sand or 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.
   D. If GABC is used, then either a pump truck or direct discharge from concrete batch trucks may be appropriate depending upon the circumstances.
   E. If GABC or sand is used, this underslab layer should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557).

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 Design and Construction

We understand that site pavements may consist of both flexible and rigid pavements. Based upon the assumption that the pavement support soils will consist of compacted fill and near surface sandy soils similar to those we tested, we estimate that an average combined California Bearing Ratio (CBR) value of at least 15 percent will be available for pavement support. This results in a resilient modulus of at least 14,457 psi available for flexible pavement design. This assumes that any fill materials used in the upper 2 feet will have a CBR value of at least 15 percent when properly compacted. If materials having lesser subgrade support values are to be considered for use, the pavement design should be reevaluated and required pavement thickness may need to be increased as a result.

Traffic volumes for the proposed development were not provided to us in preparation for our pavement section analysis; therefore, we have performed our calculations based on typical pavement section thicknesses. These pavement section components are provided in Table 6-1 below.

Flexible pavement design assumes an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 95 percent. ESALs per axle were estimated using data provided in AASHTO literature. 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). A sub-base drainage factor of 1.0 was assigned, based upon the assumption that the sub-base soils will consist of sandy fill soils.

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 the concrete would be reinforced at the joints with steel dowels to improve load transfer efficiency, 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 the ESAL demand is found to be greater than the theoretical values in the table below, the pavement section thicknesses may need adjusted and we can be contacted for further calculations.

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

Pavement Type	Theoretical Available Traffic Capacity (ESALs)	HMA Surface Course Type C (inches)	HMA Intermediate Type C (inches)	4,000 psi Concrete Pavement (inches)	Compacted SCDOT Graded Aggregate Base Course [GABC] (inches)
HMA Flexible Standard-Duty (no trucks/buses)	70,000	2.0	---	---	6.0
HMA Flexible Heavy-Duty (with trucks/buses)	285,000	1.5	1.5	---	6.0
Rigid Concrete	400,000	---	---	6.0	6.0

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

6.5.1 *General Recommendations for Pavement Areas*

1. At least one laboratory California Bearing Ratio (CBR) test should be performed upon a representative soil sample of each soil type which is planned to be used as pavement subgrade material. This is to establish the relationship between relative compaction and CBR for the soil in question, and to confirm that the obtained CBR value at the required level of compaction is equal to or greater than the CBR value utilized during design of the pavement section.
2. All fill placed in pavement areas should be compacted as recommended in Section 6.2 "Fill Placement and Compaction". Prior to placement of graded aggregate base course stone, all exposed pavement subgrades should be methodically proofrolled under the observation of the Geotechnical Engineer (S&ME), and any identified unstable areas should be repaired as directed.

6.5.2 *Base Course and Pavement Section Construction*

The following recommendations are provided for base course and pavement section construction:

1. Prior to placement of base course stone, all exposed pavement subgrades should be methodically proofrolled by the contractor under the observation of the Geotechnical Engineer (S&ME), and any identified unstable areas should be repaired. Pavement subgrades should not exhibit rutting or pumping under the proofroll load. Rutting or pumping areas shall be undercut and replaced and/or stabilized as directed by the engineer.
2. Crushed stone aggregate base material used in pavement section construction should consist of graded aggregate base course (GABC) as defined by Section 305 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007) for either macadam or marine limestone base course. The base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC-T-140).

   A. We do not recommend allowing the substitution of "commercial grade" base course materials that are non-compliant with SCDOT Section 305 gradation requirements.
   B. We do not recommend allowing the substitution of "Coquina" shell base material for the specified GABC material, because Coquina has a weaker structural design coefficient and different drainage properties than GABC,

3. 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 near optimum moisture content in order to facilitate proper compaction.
4. After placement of base course stone, the surface should be methodically proofrolled at final base grade elevation by the contractor under the observation of the Geotechnical Engineer (S&ME), and any identified unstable areas should be repaired. The base course material should not exhibit pumping or rutting under equipment traffic. Rutting or pumping areas shall be undercut and replaced and/or stabilized as directed by the engineer.
5. Construct the surface and intermediate course HMA in accordance with the specifications of Sections 401, 402, and 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition).

6. 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.
7. Experience indicates that a thin surface overlay of asphalt pavement may be required in about 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.

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 contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other representation or warranty either express or implied, is made.

We relied on project information given to us to develop our conclusions and recommendations. If project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so that we can modify our recommendations based on this additional information if necessary. Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report, we should be notified. This report should not be construed to represent subsurface conditions for the entire parent parcel, but applied only to the explored portion of the site.

S&ME should be retained to review the final plans and specifications to confirm that earthwork, foundation, and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME's review of final plans and specifications followed by our observation and monitoring of earthwork and foundation construction activities. Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

Appendices

Appendix I

Site Vicinity Map

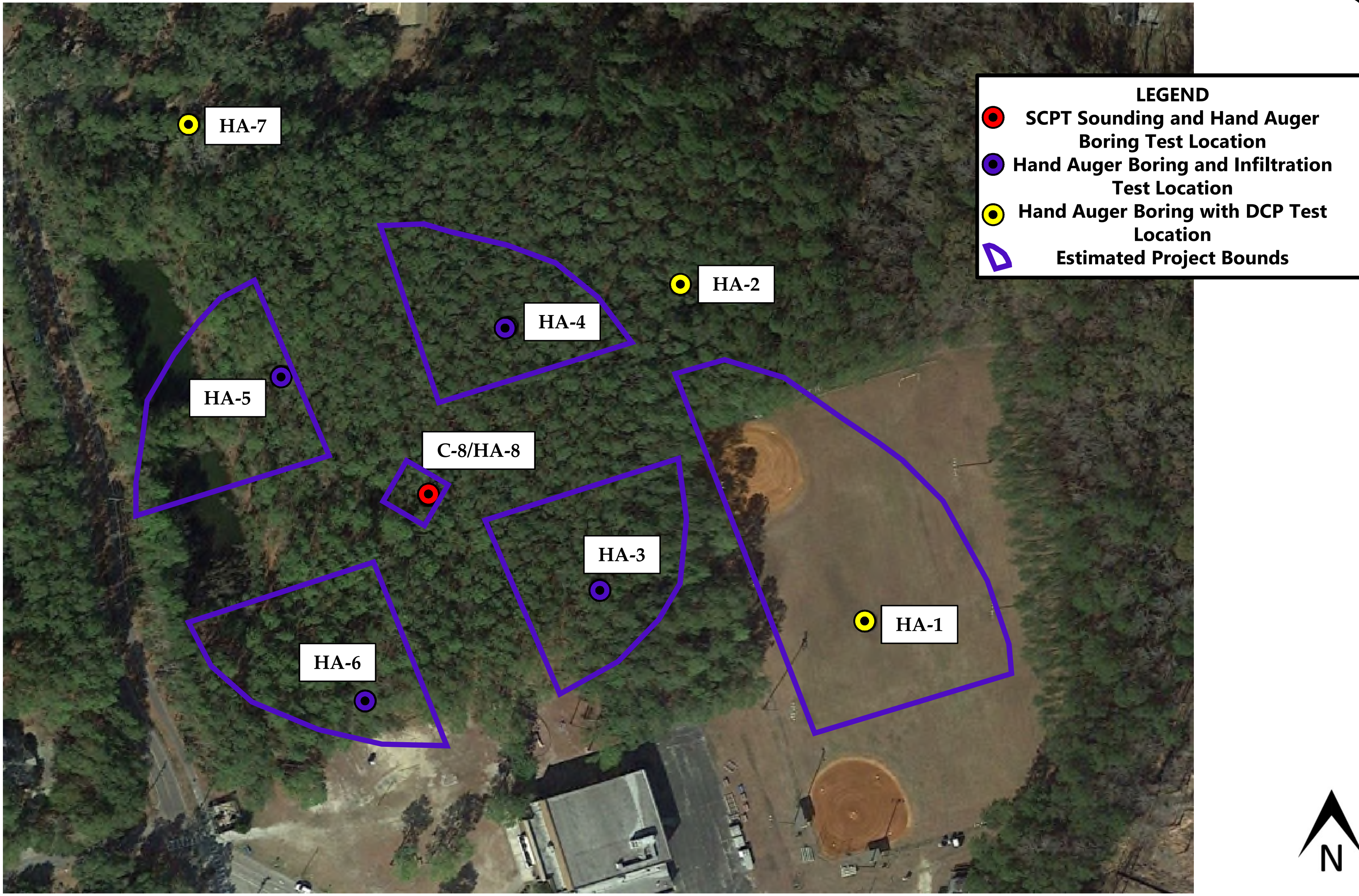
Waccamaw Elementary Athletic Fields
Pawleys Island, South Carolina

SCALE:
AS SHOWN

DATE:
4-28-2020

PROJECT NO.
1363-20-017

FIGURE NO.
1



Test Location Sketch

Waccamaw Elementary Athletic Fields
Pawleys Island, South Carolina

SCALE: AS SHOWN	FIGURE NO. 2
DATE: 4-28-2020	
PROJECT NO. 1363-20-017	

Soil Survey

Waccamaw Elementary Athletic Fields
Pawleys Island, South Carolina

SCALE:	AS SHOWN
DATE:	4-28-2020
PROJECT NO.	1363-20-017
FIGURE NO.	3

Appendix II

♦ 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-420-18, "*Standard Guide for 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 South Carolina (SC 811) before we drill or excavate at any site. SC 811 is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. SC 811 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.

Downhole Shear Wave Velocity Test

Shear wave velocity measurements were performed using a cone penetrometer equipped with geophones, or a seismic cone penetrometer (SCPT). The seismic cone penetrometer measures the travel times of surface generated vibrations to geophones mounted on the penetrometer at various incremental depths in the sounding. At a given depth, the travel time of the first arrival is measured and corrected for the horizontal offset of the source at the surface from the sounding. Interval velocities are calculated by dividing the difference in travel times by the vertical distance between successive measurement depths. Measurements were made at 1 meter intervals – the length of commonly available CPT extension rods – unless otherwise noted.

Hand Auger Borings

Auger borings were advanced using hand-operated augers. The soils encountered were identified in the field by cuttings brought to the surface. Representative samples of the cuttings were placed in plastic bags and transported to the laboratory. A bulk sample was collected from the hand augers, from the upper 4 feet of surface soils. In some of the hand auger borings, soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. Penetration resistance was not measured in all of the hand auger borings.

Dynamic Cone Penetrometer Testing

In some of the hand auger borings, 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. Soil cuttings removed from each hole were collected as a bulk sample for laboratory testing.

Ksat Testing

An additional auger boring was advanced to a target test depth using a hand-operated auger. The soils encountered were identified in the field by cuttings brought to the surface. Within the bore hole the constant-head well permeameter technique (also known as shallow well pump-in technique and bore hole permeameter method) will be used. This procedure is described in Methods of Soil Analysis, Part 1., Chapter 29 – Hydraulic Conductivity of Saturated Soils: Field Methods, 29 – 3.2 Shallow Well Pump In Method, pp. 758-763 and in the Soil Science Society of America Journal, Vol. 53, no. 5, Sept. – Oct. 1989, "A Constant-head Permeameter for Measuring Saturated Hydraulic Conductivity of the Vadose Zone" and "Comparison of the Glover Solution with the Simultaneous – Equations Approach for Measuring Hydraulic Conductivity." This method involved allowing a measured volume of water to percolate through the soil until a steady rate of flow is achieved. This final rate was used to calculate the saturated hydraulic conductivity of the subsoil horizon by the Glover equation.

Water Level Measurement

Subsurface 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.

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.

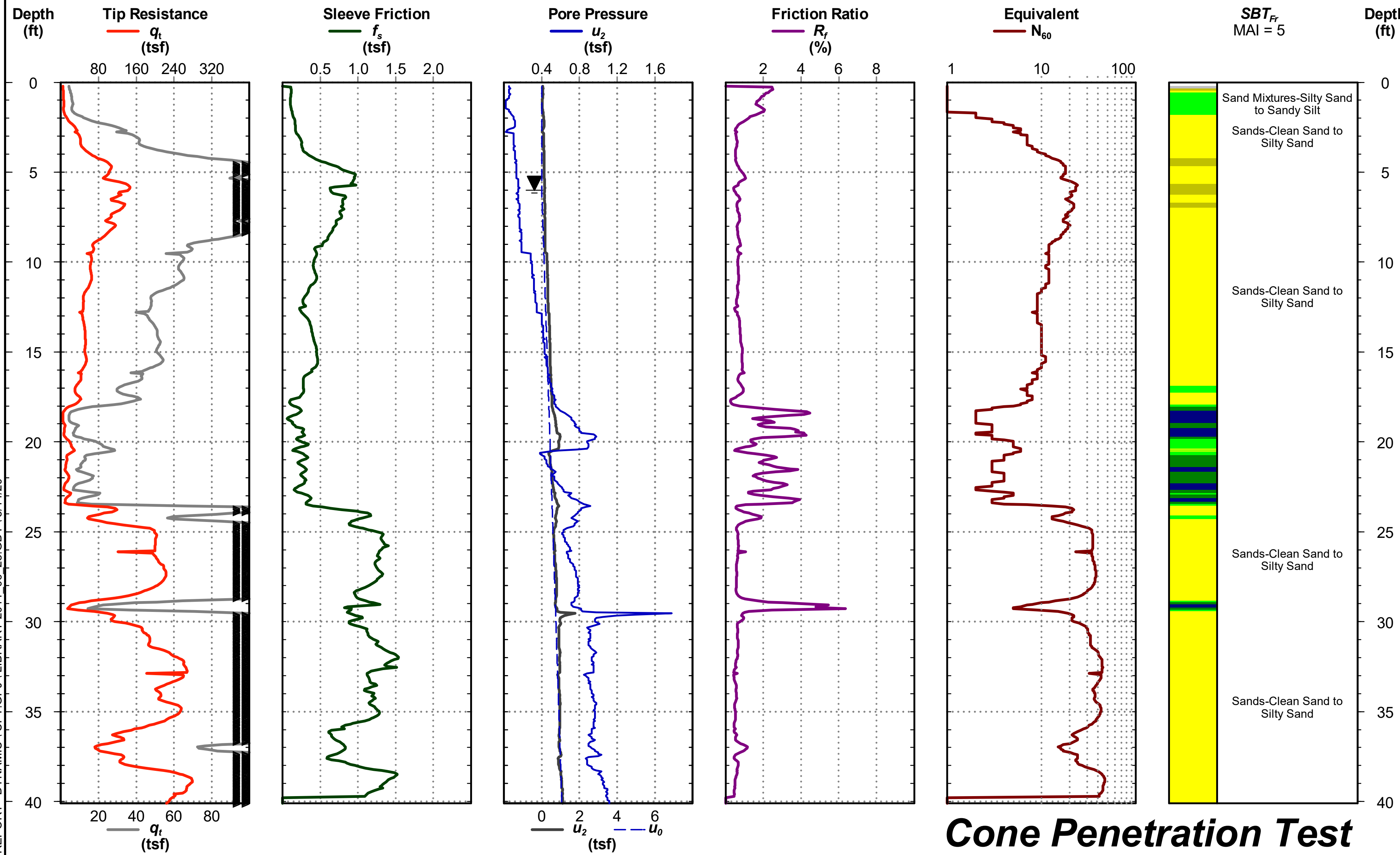
Relative Density and Consistency Table			
SANDS		SILTS and CLAYS	
Cone Tip Stress, qt (tsf)	Relative Density	Cone Tip Stress, qt (tsf)	Consistency
Less than 20	Very Loose	Less than 5	Very Soft
20 - 40	Loose	5 - 15	Soft to Firm
40 - 120	Medium Dense	15 - 30	Stiff
120 - 200	Dense	30 - 60	Very Stiff
Greater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

Sounding ID: C-8

Waccamaw Elementary Athletic Fields
Pawleys Island SC
S&ME Project No: 1363-20-017

Date: May. 7, 2020
Estimated Water Depth: 6 ft
Rig/Operator: ATV/T. Chew

Total Depth: 40.1 ft
Termination Criteria: Target Depth
Cone Size: 1.75



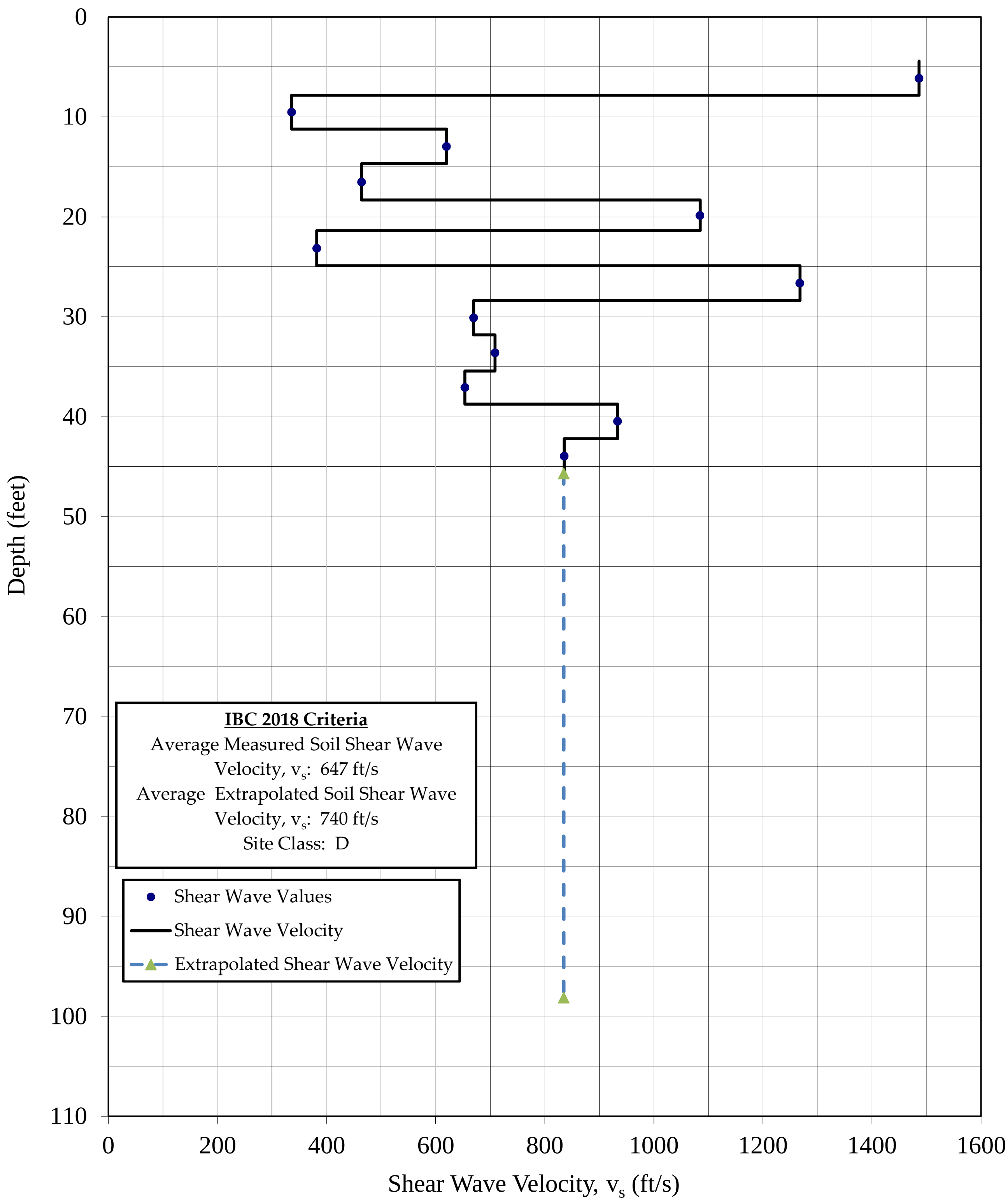
Cone Penetration Test

Electronic Filename: C-8(001)_PD.ECP

Figure 4: Shear Wave Velocity Calculations

Waccamaw Elementary Athletic Fields
Pawley's Island, SC

Sounding ID: **C-8** Project Number: **1363-20-017**
Date: 05/15/20



* Site Class based on 2018 International Building Code - *Table 1613.5.2 - SITE CLASS DEFINITIONS*

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

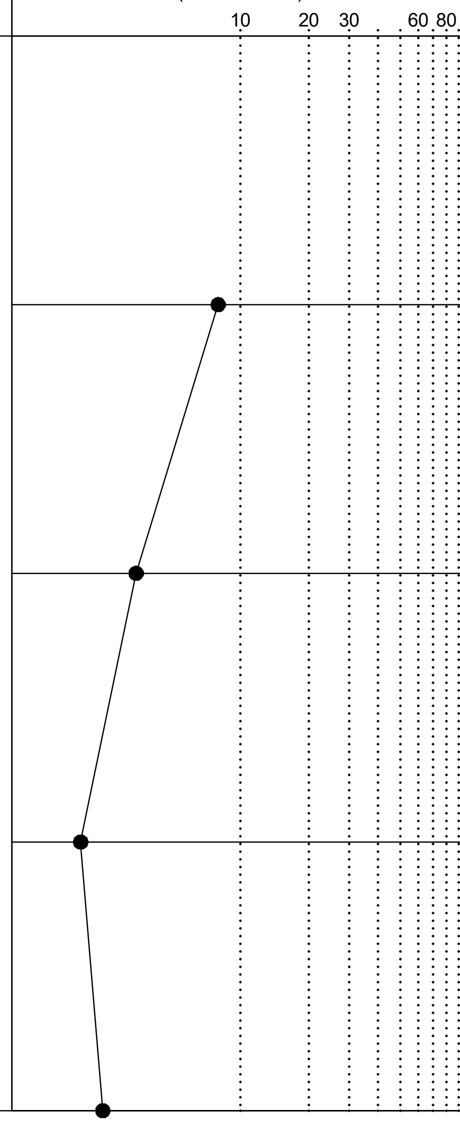
MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - 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:	**Waccamaw Elementary Athletic Fields** **Georgetown, South Carolina** **1363-20-017**	**HAND AUGER BORING LOG: HA-1**

DATE STARTED: **4/1/20**	DATE FINISHED: **4/1/20**	NOTES: Elevation unknown.
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **K. Fugate**	
WATER LEVEL: **3.5' ATD**		

Depth (feet)	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DCP VALUE
	TOPSOIL - Approximately 6 inches thick.			
	POORLY GRADED SAND (SP) - Mostly fine to medium sand, trace fines, orange, moist, loose. Possible Fill.			
1	**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, some organics, brown, wet, loose.			8
2	**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, grey, wet, very loose.			4
	- - - - Some medium to high plasticity fines.			
3				2
			▽	
	- - - - Some low to medium plasticity fines.			
4	Boring terminated at 4 ft			3

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.

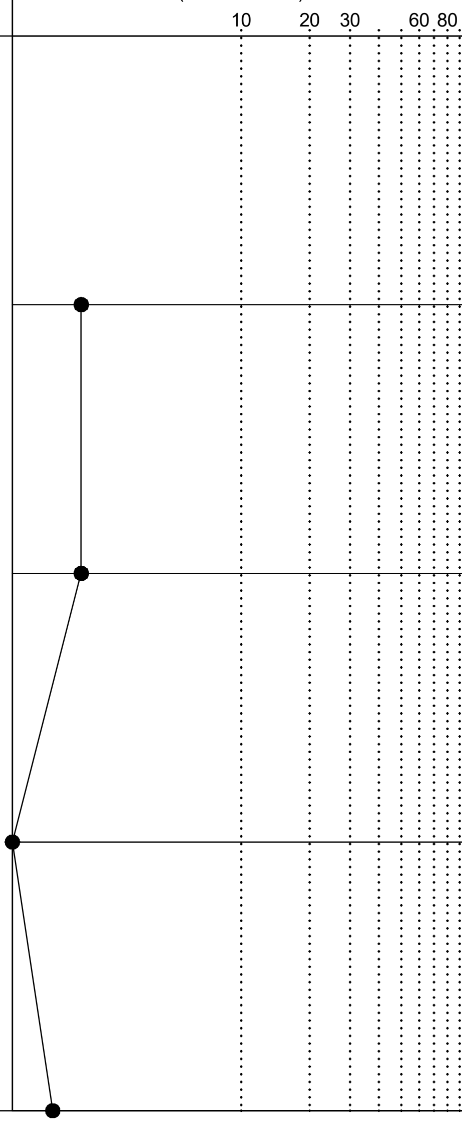
PROJECT:	Waccamaw Elementary Athletic Fields Georgetown, South Carolina 1363-20-017	**HAND AUGER BORING LOG: HA-2**

DATE STARTED:	**4/1/20**	DATE FINISHED:	**4/1/20**
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**K. Fugate**
WATER LEVEL:	**1.5' ATD**		

NOTES:
Elevation unknown.

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DCP VALUE
0–1		**TOPSOIL** - Approximately 1 foot thick.			
1		**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, grey, very loose.			2
1.5				▽	
2					2
3					1
4		Boring terminated at 4 ft			2

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.

PROJECT: Waccamaw Elementary Athletic Fields Georgetown, South Carolina 1363-20-017	**HAND AUGER BORING LOG: HA-3**

DATE STARTED: **4/1/20**	DATE FINISHED: **4/1/20**	NOTES: Elevation unknown.
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **K. Fugate**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL - Approximately 1 foot thick.		
1		**POORLY GRADED SAND WITH SILT (SP-SM)** - Mostly fine to medium sand, few low plasticity fines, tan, moist.		
2		**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, orange, moist to wet.		
3		**POORLY GRADED SAND WITH CLAY (SP-SC)**- Mostly fine to medium sand, few low to medium plasticity fines, red and orange, wet.		
4		Boring terminated at 4 ft		

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.

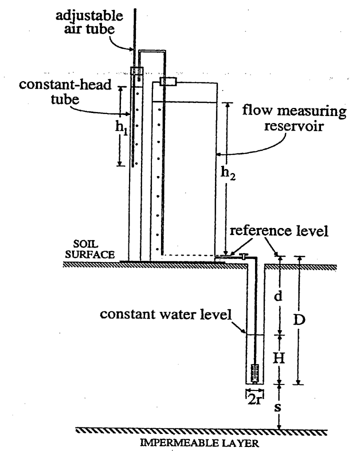
S&ME
"IN-SITU" CONSTANT HEAD PERMEAMETER

Date: 5/1/2020
Location: HA-3
Client: Georgetown County
Project Name: Waccamaw Elementary Atheletic Fields
Project #: 136320017

Hole Depth:	1.50	Feet
Hole Radius (r):	0.13	Feet
Reference Tube to Hole Bottom (D):	1.77	Feet
Water Depth in Hole (H):	1.08	Feet
CHT Tube(s) Setting (h_1):	1.00	Feet

Chamber Used:	0.11	Ft^2

Initial Water in Hole:	1.08	Feet
Final Water in Hole:	1.08	Feet



Ksat = CQ/(2PiH^2)

C = sinh^{-1} (H/r) - [(r/H)^2 + 1]$^{1/2}$ + r/H

sinh^{-1} = inverse hyperbolic sin of a number
H = Height of water in hole (cm)
r = radius of hole (cm)
Q = Constant Flow Rate (Gal/day)
= Cross Sectional Area of Reservior x Length of Drop in Water Column over Time

r = **0.13** ft
H = **1.08** ft

C = **1.96**
Q = **14.58** Gallons/Day

Time (min.)	Drop in Water Column (ft) (ft)	(cm)
0-10	0.229	6.99
10-15	0.073	2.22
15-20	**0.063**	**1.91**
20-25	**0.063**	**1.91**
25-30	**0.063**	**1.91**
Avg.	**0.063**	

Time (min) = 5

Qualitative Classifications(1):

Very Slow	<0.06 iph
Slow	.06-0.2 iph
Mod. Slow	0.2-0.6 iph
Moderate	0.6-2.0 iph
Mod. Rapid	2.0-6.0 iph
Rapid	6.0-20 iph
Very Rapid	>20 iph

(1): Soil Conservation Service (USDA)

Cross Sectional Area = **0.11** ft^2
Length of Drop in Water Column = **17.25** ft/day

K_{sat} = 3.91 Gallons/Day/ft^2

Inches/Hour = **0.261**

Note: Ksat calculations are based on average drop in Water Column (ft) after equilibrium is reached.

PROJECT:	**Waccamaw Elementary Athletic Fields** **Georgetown, South Carolina** **1363-20-017**	**HAND AUGER BORING LOG: HA-4**

DATE STARTED: **4/1/20**	DATE FINISHED: **4/1/20**	NOTES: Elevation unknown.
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **K. Fugate**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL - Approximately 4 inches thick.		
		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, trace organics, grey, wet.		
1		**POORLY GRADED SAND WITH CLAY (SP-SC)** - Mostly fine to medium sand, few low to medium plasticity fines, orange, wet.		
2		**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, grey and orange, wet.		
3		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, brown, moist.		
4		Boring terminated at 4 ft		

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.

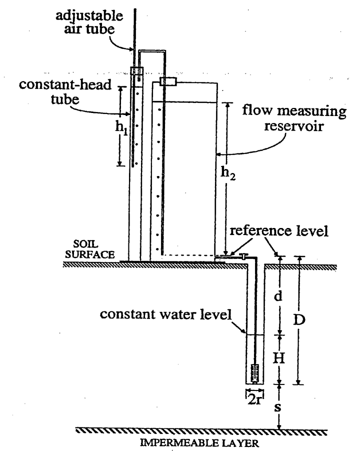
S&ME
"IN-SITU" CONSTANT HEAD PERMEAMETER

Date: 5/1/2020
Location: HA-4
Client: Georgetown County
Project Name: Waccamaw Elementary Atheletic Fields
Project #: 136320017

Hole Depth:	1.00	Feet
Hole Radius (r):	0.08	Feet
Reference Tube to Hole Bottom (D):	1.19	Feet
Water Depth in Hole (H):	0.83	Feet
CHT Tube(s) Setting (h_1):	0.90	Feet

Chamber Used:	0.11	Ft^2

Initial Water in Hole:	0.83	Feet
Final Water in Hole:	0.83	Feet



$K_{sat} = CQ/(2\pi H^2)$

$C = \sinh^{-1}(H/r) - [(r/H)^2 + 1]^{1/2} + r/H$

$\sinh^{-1}$ = inverse hyperbolic sin of a number
H = Height of water in hole (cm)
r = radius of hole (cm)
Q = Constant Flow Rate (Gal/day)
= Cross Sectional Area of Reservior x Length of Drop in Water Column over Time

r = **0.08** ft
H = **0.83** ft

C = **2.09**
Q = **4.86** Gallons/Day

Time (min.)	Drop in Water Column (ft) (ft)	(cm)
0-23 min.	0.099	3.02
23-28 min.	**0.021**	**0.64**
28-33 min.	**0.021**	**0.64**
33-38 min.	**0.021**	**0.64**
Avg.	**0.021**	

Time (min) = 5

Qualitative Classifications(1):

Very Slow	<0.06 iph
Slow	.06-0.2 iph
Mod. Slow	0.2-0.6 iph
Moderate	0.6-2.0 iph
Mod. Rapid	2.0-6.0 iph
Rapid	6.0-20 iph
Very Rapid	>20 iph

(1): Soil Conservation Service (USDA)

Cross Sectional Area = **0.11** ft^2
Length of Drop in Water Column = **5.75** ft/day

K_{sat} = 2.35 Gallons/Day/ft^2

Inches/Hour = **0.157**

Note: Ksat calculations are based on average drop in Water Column (ft) after equilibrium is reached.

PROJECT:	Waccamaw Elementary Athletic Fields Georgetown, South Carolina 1363-20-017	HAND AUGER BORING LOG: HA-5

DATE STARTED:	**4/1/20**	DATE FINISHED:	**4/1/20**
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**K. Fugate**
WATER LEVEL:	**Not encountered.**		

NOTES:
Elevation unknown.

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		POORLY GRADED SAND (SP) - Mostly fine to medium sand, trace fines, trace roots, light to dark grey, moist.		
1		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, light brown, moist.		
2		**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, moist to wet.		
3		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, tan to brown, wet.		
4		Boring terminated at 4 ft		

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.

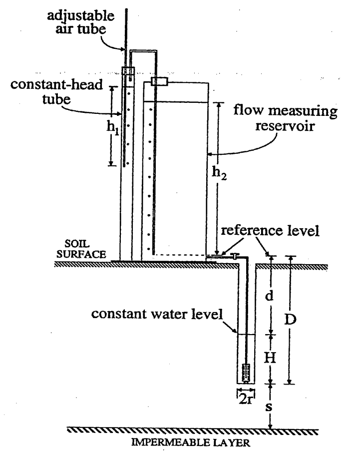
S&ME
"IN-SITU" CONSTANT HEAD PERMEAMETER

Date: 5/1/2020
Location: HA-5
Client: Georgetown County
Project Name: Waccamaw Elementary Atheletic Fields
Project #: 136320017

Hole Depth:	1.00	Feet
Hole Radius (r):	0.13	Feet
Reference Tube to Hole Bottom (D):	1.19	Feet
Water Depth in Hole (H):	0.50	Feet
CHT Tube(s) Setting (h_1):	0.60	Feet

Chamber Used:	0.11	Ft^2

Initial Water in Hole:	0.50	Feet
Final Water in Hole:	0.50	Feet



$K_{sat} = CQ/(2\pi H^2)$

$C = \sinh^{-1}(H/r) - [(r/H)^2 + 1]^{1/2} + r/H$

$\sinh^{-1}$ = inverse hyperbolic sin of a number
H = Height of water in hole (cm)
r = radius of hole (cm)
Q = Constant Flow Rate (Gal/day)
= Cross Sectional Area of Reservior x Length of Drop in Water Column over Time

r = **0.13** ft
H = **0.50** ft

C = **1.31**
Q = **32.83** Gallons/Day

Time (min.)	Drop in Water Column (ft) (ft)	(cm)
0-5	0.156	4.7625
5-10	0.135	4.13
10-15	0.146	4.45
15-20	**0.141**	**4.29**
20-25	**0.141**	**4.29**
25-30	**0.141**	**4.29**
Avg.	**0.141**	

Time (min) = 5

Qualitative Classifications(1):

Very Slow	<0.06 iph
Slow	.06-0.2 iph
Mod. Slow	0.2-0.6 iph
Moderate	0.6-2.0 iph
Mod. Rapid	2.0-6.0 iph
Rapid	6.0-20 iph
Very Rapid	>20 iph

(1): Soil Conservation Service (USDA)

Cross Sectional Area = **0.11** ft^2
Length of Drop in Water Column = **38.84** ft/day

K_{sat} = 27.48 **Gallons/Day/ft^2**

Inches/Hour = **1.837**

Note: Ksat calculations are based on average drop in Water Column (ft) after equilibrium is reached.

PROJECT:	Waccamaw Elementary Athletic Fields Georgetown, South Carolina 1363-20-017	**HAND AUGER BORING LOG: HA-6**

DATE STARTED:	**4/1/20**	DATE FINISHED:	**4/1/20**	NOTES: Elevation unknown.
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**K. Fugate**	
WATER LEVEL:	**Not encountered.**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		POORLY GRADED SAND (SP) - Mostly fine to medium sand, trace fines, trace roots, light to dark grey, moist.		
1 2		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, brown, moist. - - - - Tan.		
		CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange and tan, wet.		
3		**POORLY GRADED SAND WITH CLAY (SP-SC)** - Mostly fine to medium sand, few low to medium plasticity fines, tan, wet.		
4		**POORLY GRADED SAND (SP)** - Mostly fine to coarse sand, trace fines, brown, wet.		

Boring terminated at 4 ft

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.

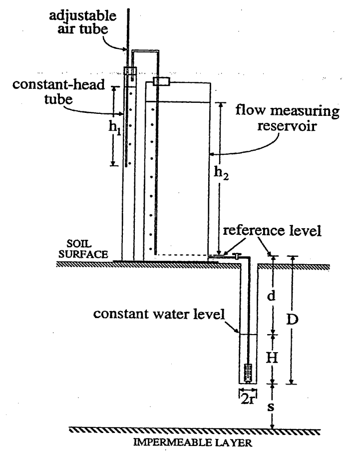
S&ME
"IN-SITU" CONSTANT HEAD PERMEAMETER

Date: 5/1/2020
Location: HA-6
Client: Georgetown County
Project Name: Waccamaw Elementary Atheletic Fields
Project #: 136320017

Hole Depth:	1.00	Feet
Hole Radius (r):	0.13	Feet
Reference Tube to Hole Bottom (D):	1.19	Feet
Water Depth in Hole (H):	0.42	Feet
CHT Tube(s) Setting (h_1):	0.60	Feet

Chamber Used:	0.11	Ft^2

Initial Water in Hole:	0.42	Feet
Final Water in Hole:	0.42	Feet



Ksat = CQ/(2PiH²)

$$K_{sat} = CQ/(2\pi H^2)$$

C = sinh⁻¹ (H/r) - [(r/H)² + 1]^{1/2} + r/H

$$C = \sinh^{-1}(H/r) - [(r/H)^2 + 1]^{1/2} + r/H$$

$\sinh^{-1}$ = inverse hyperbolic sin of a number
H = Height of water in hole (cm)
r = radius of hole (cm)
Q = Constant Flow Rate (Gal/day)
= Cross Sectional Area of Reservior x Length of Drop in Water Column over Time

r = **0.13** ft
H = **0.42** ft

C = **1.18**
Q = **24.31** Gallons/Day

Time (min.)	Drop in Water Column (ft) (ft)	(cm)
0-10	0.313	9.525
10-13	0.099	3.01625
13-16	0.099	3.02
16-19	0.109	3.33
19-22	**0.104**	**3.18**
22-25	**0.104**	**3.18**
25-28	**0.104**	**3.18**
Avg.	**0.104**	

Time (min) = 5

Qualitative Classifications(1):

Very Slow	<0.06 iph
Slow	.06-0.2 iph
Mod. Slow	0.2-0.6 iph
Moderate	0.6-2.0 iph
Mod. Rapid	2.0-6.0 iph
Rapid	6.0-20 iph
Very Rapid	>20 iph

(1): Soil Conservation Service (USDA)

Cross Sectional Area = **0.11** ft^2
Length of Drop in Water Column = **28.75** ft/day

K_{sat} = 25.91 Gallons/Day/ft^2

Inches/Hour = **1.732**

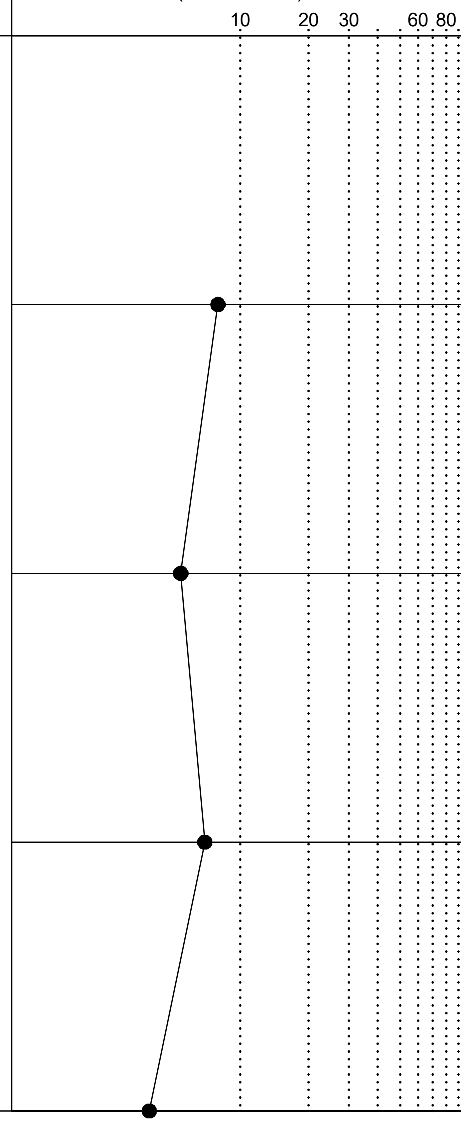
Note: Ksat calculations are based on average drop in Water Column (ft) after equilibrium is reached.

PROJECT:	Waccamaw Elementary Athletic Fields Georgetown, South Carolina 1363-20-017	**HAND AUGER BORING LOG: HA-7**

				NOTES:
DATE STARTED:	**4/1/20**	DATE FINISHED:	**4/1/20**	Elevation unknown.
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**K. Fugate**	
WATER LEVEL:	**Not encountered.**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DCP VALUE
		TOPSOIL - Approximately 6 inches thick.			
		POORLY GRADED SAND (SP) - Mostly fine to medium sand, trace fines, grey, wet, loose to very loose.			
1					8
2		---- Tan, wet.			6
3					7
4		Boring terminated at 4 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.

PROJECT:	**Waccamaw Elementary Athletic Fields** **Georgetown, South Carolina** **1363-20-017**	**HAND AUGER BORING LOG: HA-8**

DATE STARTED: **4/1/20**	DATE FINISHED: **4/1/20**	NOTES: Elevation unknown.
SAMPLING METHOD: **Hand Auger**	PERFORMED BY: **K. Fugate**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
0 – 0.3		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, trace roots, light to dark grey, moist.		
0.3 – 1.75		**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, trace fines, light brown, moist.		
1.75 – 2.75		**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, orange, wet.		
2.75 – 4		**POORLY GRADED SAND (SP)** - Mostly fine to coarse sand, trace fines, brown, wet.		

Boring terminated at 4 ft

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.

Appendix III

♦ 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.

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.

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 lim*it 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. 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. 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.

The plastic limit was determined using the procedure described in Section 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.

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 D 1557, *"Standard Test Method for 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 ¾ 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 ¾inch 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 Method for 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 sample 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.

Form No: TR-D2216-T265-1
Revision No. 1
Revision Date: 08/16/17

LABORATORY DETERMINATION OF WATER CONTENT

ASTM D 2216 ☑ *AASHTO T 265* ☐

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

Project #:	1363-20-017	Report Date:	5/11/2020
Project Name:	Waccamaw Elementary Athletic Fields	Test Date(s):	5/7/2020
Client Name:	Georgetown County Recreation & Community Services		
Client Address:	2030 Church St; Georgetown, SC 29440		
Sample by:	K. Fugate	Sample Date(s):	5/1/2020

Method: **A** **(1%)** ☐ **B** **(0.1%)** ☑

Balance ID.	19608	Calibration Date:	2/28/19
Oven ID.	17745	Calibration Date:	4/8/19

Boring No.	Sample No.	Sample Depth	Tare #	Tare Weight	Tare Wt.+ Wet Wt	Tare Wt. + Dry Wt	Water Weight	Percent Moisture	Note
		ft. or m.		grams	grams	grams	grams	%	
B-1 & B-2	C-1	1'-4'	TTT	83.90	217.20	201.20	16.00	13.6%	

Notes / Deviations / References

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Ron Forest, P.E.	RPF	Senior Reviewer	11-May
Technical Responsibility	*Signature*	*Position*	*Date*

Form No: TR-D1140-1
Revision No. 1
Revision Date: 8/2/17

MATERIAL FINER THAN THE #200 SIEVE

ASTM D1140

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

Project #:	1363-20-017	Report Date:	5/11/2020
Project Name:	Waccamaw Elementary Athletic Fields	Test Date(s):	5/6/2020
Client Name:	Georgetown County Recreation & Community Services		
Client Address:	2030 Church St; Georgetown, SC 29440		
Sample by:	K. Fugate	**LAB#**	150
		Sample Dates:	5/1/2020

Method; A ☐ **B** ☑ Soaked ☑ Soak Time 2 Hrs

Boring #	Sample #	Sample Depth	Tare #	Tare Weight	Tare Wt.+ Wet Wt	Tare Wt. + Dry Wt	Tare Wt. + Dry Wt. after Wash	% Passing #200
		ft. or m.		grams	grams	grams	grams	%
B-1 & B-2	C-1	1'-4'	CCC	82.20	322.00	322.00	272.60	**20.6%**

Balance ID. *19608* *Calibration Date:* *2/28/19* *#200 Sieve* *18775* *Calibration Date:* *2/28/20*

Notes / Deviations / References: ASTM D1140: Amount of Material in Soil Finer Than the No. 200 (75-um)) Sieve

Ron Forest, P.E.	RPF	Senior Reviewer	11-May
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D4318-T89-90
Revision No. 1
Revision Date: 7/26/17

LIQUID LIMIT, PLASTIC LIMIT, & PLASTIC INDEX

ASTM D 4318 ☒ AASHTO T 89 ☐ AASHTO T 90 ☐

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

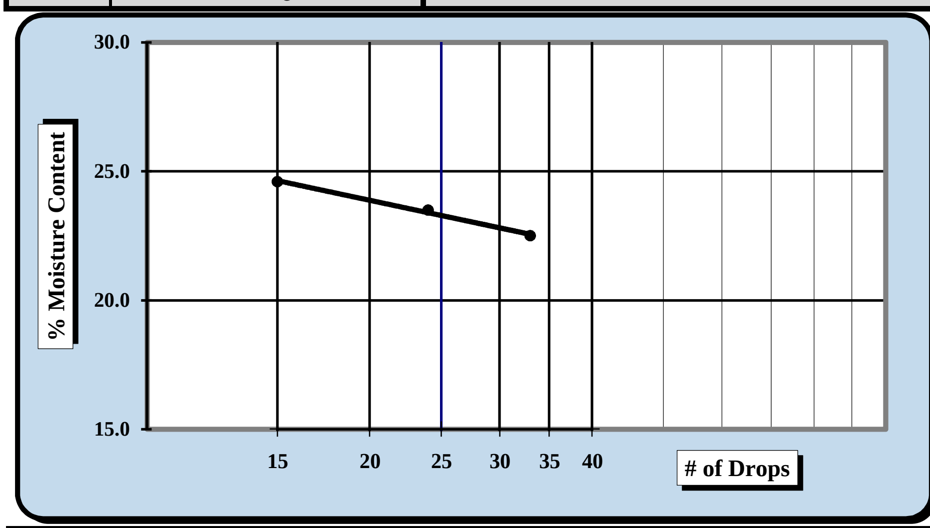
Project #:	1363-20-017	Report Date:	5/11/2020
Project Name:	Waccamaw Elementary Athletic Fields	Test Date(s)	5/7/2020
Client Name:	Georgetown County Recreation & Community Services		
Client Address:	2030 Church St; Georgetown, SC 29440		

Boring #:	B-1 & B-2	Sample #:	C-1	Sample Date:	5/1/2020
Location:	Pavements	LAB #:	150	Depth:	1'-4'

Sample Description: Brown Clayey Sand (SC)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	00401	2/28/2019	Grooving tool	11368	9/1/2018
LL Apparatus	18801	9/1/2018			
Oven	17745	4/8/2019			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	15	14	57				82	19	
A	Tare Weight	14.25	14.33	14.54				14.74	14.66	
B	Wet Soil Weight + A	31.28	31.33	31.40				21.52	21.63	
C	Dry Soil Weight + A	28.15	28.10	28.07				20.87	20.96	
D	Water Weight (B-C)	3.13	3.23	3.33				0.65	0.67	
E	Dry Soil Weight (C-A)	13.90	13.77	13.53				6.13	6.30	
F	% Moisture (D/E)*100	22.5%	23.5%	24.6%				10.6%	10.6%	
N	# OF DROPS	33	24	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = **F** * FACTOR									
Ave.	*Average*							**10.6%**		



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	**24**
Plastic Limit	**11**
Plastic Index	**13**
Group Symbol	**SC**

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

Ronald P. Forest, Jr.	RPF	Senior Engineer	5/11/2020
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D698-2
Revision No. : 1
Revision Date: 07/25/17

MOISTURE - DENSITY REPORT

Quality Assurance

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

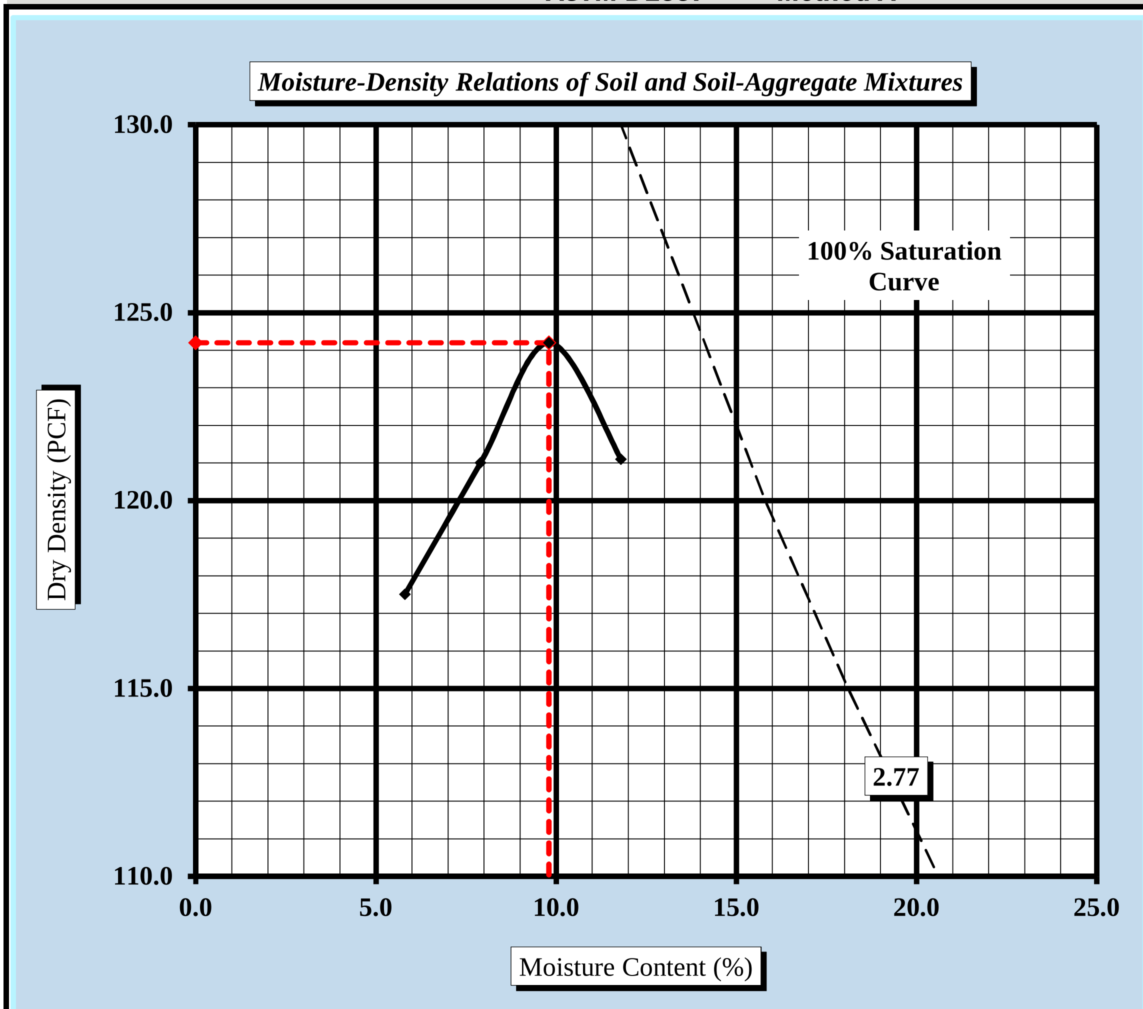
S&ME Project #:	1363-20-017	Report Date:	5/11/2020
Project Name:	Waccamaw Elementary Athletic Fields	Test Date(s):	5/4/2020
Client Name:	Georgetown County Recreation & Community Services		
Client Address:	2030 Church St; Georgetown, SC 29440		

Boring #:	B-1 & B-2	Sample #:	C-1	Sample Date:	5/1/2020
Location:	Pavements	Lab #:	150	Depth:	1'-4'

Sample Description: Brown Clayey Sand (SC)

Maximum Dry Density 124.2 PCF. Optimum Moisture Content 9.8%

ASTM D1557 - - Method A



Soil Properties	
Natural Moisture Content	13.6%
Specific Gravity of Soil	--
Liquid Limit	24
Plastic Limit	11
Plastic Index	13
% Passing	
3/4"	--
3/8"	--
#4	--
#10	--
#40	--
#60	--
#200	20.6%
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

Ronald P. Forest, Jr.	RPF	Senior Engineer	5/11/2020
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D1883-T193-3
Revision No. 2
Revision Date: 08/11/17

CBR (CALIFORNIA BEARING RATIO) OF LABORATORY COMPACTED SOIL

ASTM D 1883

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

Project #:	1363-20-017	Report Date:	5/11/2020
Project Name:	Waccamaw Elementary Athletic Fields	Test Date(s)	5/4/2020
Client Name:	Georgetown County Recreation & Community Services		
Client Address:	2030 Church St; Georgetown, SC 29440		

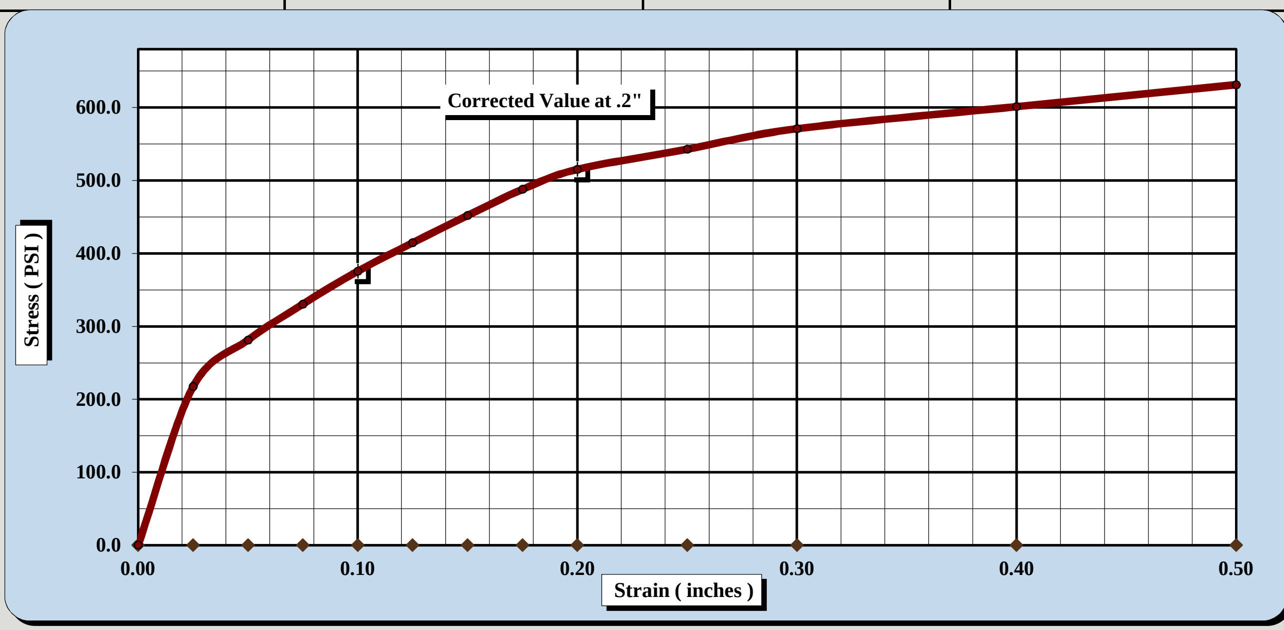
Boring #:	B-1 & B-2	Sample #:	C-1	Sample Date:	5/1/2020
Location:	Pavements	LAB #:	150	Depth:	1'-4'

Sample Description: Brown Clayey Sand (SC)

ASTM D1557 *Method A* Maximum Dry Density: 124.2 PCF Optimum Moisture Content: 9.8%

Compaction Test performed on grading complying with CBR spec. % Retained on the 3/4" sieve: 1.0%

Uncorrected CBR Values				Corrected CBR Values			
CBR at 0.1 in.	37.6	CBR at 0.2 in.	34.3	CBR at 0.1 in.	37.6	CBR at 0.2 in.	34.3



CBR Sample Preparation:

The entire gradation was used and compacted in a 6" CBR mold in accordance with *ASTM D1883, Section 6.1.1*

Before Soaking		*After Soaking*	
Compactive Effort (Blows per Layer)	25		
Initial Dry Density (PCF)	117.4	Final Dry Density (PCF)	118.8
Moisture Content of the Compacted Specimen	10.3%	Moisture Content (top 1" after soaking)	12.8%
Percent Compaction	94.5%	Percent Swell	-1.1%

Soak Time:	96 hrs.	Surcharge Weight	20.0	Surcharge Wt. per sq. Ft.	102.0
Liquid Limit	24	Plastic Index	13	Apparent Relative Density	--

Notes/Deviations/References: Liquid Limit: ASTM D 4318, Specific Gravity: ASTM D 854, Classification: ASTM D 2487

Ron Forest, P.E.	RPF	Senior Reviewer	5/11/2020
Technical Responsibility	*Signature*	*Position*	*Date*