# Report of Geotechnical Exploration
# SLED Pee Dee Regional Office
# Florence, South Carolina
# S&ME Project No. 23390074

PREPARED FOR:

**Moseley Architects**
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PREPARED BY:

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**July 11, 2023**

July 11, 2023

Moseley Architects
977 Morrison Avenue, Suite 601
Charleston, South Carolina 29406

Attention: Mr. Ben Anderson, AIA

Reference: **Report of Geotechnical Exploration**
**SLED Pee Dee Regional Office**
Florence, South Carolina
S&ME Project No. 23390074

Dear Mr. Anderson:

S&ME, Inc. has completed the geotechnical exploration for the referenced project. Our services were performed pursuant to S&ME Proposal No. 23390074, dated May 1, 2023, and accepted by Moseley Architects on May 5, 2023.

The purpose of this exploration was to evaluate subsurface conditions within the construction footprint as they relate to site preparation, earthwork, fill soil suitability, foundation support for the proposed structures, and pavement section construction for the proposed development.

This report describes our understanding of the project, presents the results of the field exploration, laboratory testing, and engineering analysis and discusses our geotechnical conclusions and recommendations based on these considerations.

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 LICENSED PROFESSIONAL ENGINEER 7/11/23 No. 28224 WILLIAM D. KANNON

William D. Kannon, P.E.
Project Engineer
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SOUTH CAROLINA S&ME, INC. No. C00473 CERTIFICATE OF AUTHORIZATION

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# Report at a Glance

Key geotechnical findings based on our current understanding of the proposed project are presented below. These findings are presented as an overview and should not be used in place of the more detailed recommendations presented in the remainder of this report.

| Category | Key Geotechnical Findings |
|---|---|
| **Site Development Challenges** | Site appears generally suitable for the proposed development. Specific geotechnical issues identified on this site that should be considered include:<br>• Control of surface water and potential perched water during wet periods of weather.<br>• Removal of 4 to 10 inches of topsoil, averaging roughly 6 inches.<br>• Removal of possible underground structures such as septic tanks, old foundations, drain lines, or utilities.<br>• Some loose soils encountered at the surface in isolated areas to depths of about 2 to 3 feet. Likelihood of surface instability near these locations that may require surface densification. While not expected to be widespread, undercutting and replacement may be required in isolated areas to depths ranging from 2 to 3 feet where surface instability is encountered.<br>• Rigorous compaction of the native soil surface to improve surface density. |
| **Subsurface Conditions** | • Loose to medium dense sandy mixtures to depths of about 2 to 6 feet.<br>• Stiff fine-grained soils to depths of about 5 to 9 feet.<br>• Soft silts encountered in Soundings C-2 and C-4 between depths of about 14 ½ to 19 feet.<br>• Medium dense to very dense sands to the maximum exploration depth of about 22 feet.<br>• Groundwater estimated at depths ranging from about 5 to 6 ½ feet in the CPT soundings. Groundwater measured at depths of 6 ½ to 7 ½ feet in the test pit excavations. Potential for shallow perched groundwater with sands overlying stiff fine-grained soils. |
| **Seismic Considerations** | Liquefaction risk during seismic shaking is low. Site Class D. Seismic Design Category C for Risk Categories I, II, and III. Seismic Design Category D for Risk Category IV.<br>• $S_{DS}$ = 0.322, $S_{D1}$ = 0.179, $PGA_M$ = 0.246g |
| **Foundation Type** | Shallow spread footings with a net allowable bearing pressure of up to 2,500 psf. Total static settlement of 1 inch or less and differential settlement of ½ inch or less under provided loads of 40-kips per column, 3.5 kip/ft walls, and 400 psf area load (slab weight + load on slab + fill load). |
| **Slab Support** | On-grade (soil supported). Modulus of subgrade reaction of 175 lbs./cu.in. |
| **Use of Site Soil as Fill** | Stratum I sandy soils within the proposed stormwater pond areas to depths of about 6 to 7 feet appear suitable for use as structural fill. Clayey soils of Stratum II are not preferable for use as structural fill, but may be considered for use with some risk of grading difficulties if they become wet or if grading takes place during wet periods. Fill soils should be compacted to at least 95 percent of the modified Proctor (ASTM D 1557) maximum dry density within +/-3% of optimum moisture. |
| **Excavation Conditions** | Hydraulic excavators should be able to excavate throughout the soil profile. Loose to medium dense sandy soils may be encountered to depths of up to 5 feet in portions of the site, underlain by dense hardpan materials to depths of about 9 feet. Perched water conditions may be encountered within the upper soil zone. |
| **Construction Dewatering** | Due to the depth of groundwater at the time of our exploration, we do not anticipate that widespread dewatering will be necessary except perhaps for deeper utility excavations. Construction dewatering, if necessary, may likely be accomplished using sumps with pumps to control groundwater. |
| **Pavements** | • *Light-Duty Asphalt (no trucks):* 2 in. of SCDOT Type C Surface HMA over 6 in. GABC.<br>• *Heavy-Duty Asphalt (with truck traffic):* 1.5 in. of SCDOT Type C Surface HMA, over 2.0 in. of SCDOT Type C Intermediate HMA, over 6 in. GABC.<br>• *Light-Duty Rigid (no trucks):* 5 in. of 4,000 psi Concrete over 6 in. GABC.<br>• *Heavy-Duty Rigid (with truck traffic):* 6 in. of 4,000 psi Concrete over 6 in. GABC. |

# 1.0 Introduction

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

A site plan showing the approximate exploration locations is included in Appendix I. The boring and sounding logs, an interpreted subsurface soil profile, discussion of the field exploration procedures, and legends of soil classification and symbols are included in Appendix II. Appendix III contains the results of the laboratory testing and our laboratory test procedures.

## 1.1 Site and Project Description

Project information was initially provided in email correspondence from Ben Anderson (Moseley Architects) to Will Kannon (S&ME) on April 27, 2023. Mr. Anderson's correspondence attached the following documents which were utilized in the preparation of our proposal:

- "Plat of Property Surveyed for Francis Marion University", prepared by Ervin Engineering, dated December 19, 2022.
- Architectural site drawings, "Site", "Floor Plan", "Front Aerial", "Back Aerial", "Entrance", "Front Elevation", prepared by Moseley Architects, undated.
- "Boring Locations", prepared by Moseley Architects, undated.

Additional information was provided by Steven Cooke (Moseley Architects) in email correspondence with Mr. Kannon on April 28, 2023. In that correspondence, Mr. Cooke provided information relating to structural loading.

### *1.1.1 Site Description*

The project site is located on Francis Marion Road, just south of its intersection with Harlan G. Hawkins Drive in Florence, South Carolina. We understand the property is owned by Francis Marion University. Based on our review of the provided topographic site plan, site elevations appear to range from about 79 to 94 feet above mean sea level (MSL), generally sloping gradually downward from east to west. A Site Vicinity Map is included in Appendix I as Figure 1.

### *1.1.2 Project Description*

Based on our review of the provided project information, we understand that site improvements will consist of a new South Carolina Law Enforcement Division (SLED) Regional Office facility. The facility will include an office building, a vehicle services shed, paved drive and parking areas, and a stormwater management pond. The office structure will be a one-story, pre-engineered metal building and will have a footprint of about 9,925 square feet (SF) in plan area. The vehicle service shed will also be a single-story pre-engineered metal building with a footprint of about 1,890 SF in plan area. Both structures are designed to be supported on shallow soil-supported foundations and concrete floor slabs. Based on email correspondence from Benjamin Whitener, AIA (Moseley) on

July 10, 2023, we understand that the Office Building will have an EOC component that will be used during natural disasters, etc., and will be Seismic Risk Category IV.

Site pavements will consist of asphalt paved drive areas and parking lots. The site stormwater pond will be about 1/4-acre in size and of unspecified depth.

### 1.2 Structural Loading Information

Based on the information provided by Mr. Cooke, we understand the structures may have maximum column loading of about 40 kips, and maximum wall loads of about 3.5 kips per linear foot. Floor slab loading is assumed not to exceed 150 pounds per square foot (psf), including the self-weight of the concrete slab but excluding the weight of the new fill embankment. Based on the grading assumptions described below, we anticipate that the weight of the new fill embankment in the building area may be up to about 250 psf, resulting in a total applied area load to the existing ground of up to 400 psf when considering the fill weight, the weight of the slab, and the load on the slab.

### 1.3 Static Settlement Tolerances

In the absence of any special settlement tolerances for the proposed structure, we assume 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 Grade Elevation Changes

Site elevations within the proposed office building area appear to generally range from about 89 to 92 feet. Site elevations within the proposed vehicle services building area appear to range from about 85 to 86 feet, and within the proposed parking lots, elevations appear to range from about 84 to 90 feet. We have not been provided with final grading information prior to preparing this proposal; however, we anticipate that cuts or fills of up to 2 feet across each building pad may be appropriate for our settlement models. If actual cuts/fill are greater that what we have assumed, then we should be notified prior to our performing analyses so we can modify our assumptions and re-calculate the estimated settlement magnitudes if appropriate.

## 2.0 Exploration Procedures

### 2.1 Field Exploration

Before visiting the site, SC-811 was contacted for clearance to dig at the site. Between the dates of May 30 through June 21, 2023, representatives of S&ME 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. We advanced one Seismic Cone Penetration Test (SCPT) sounding (C-2) to a depth of approximately 21 feet, at which depth the cone encountered refusal to advance. In conjunction with the SCPT sounding at location C-2, shear wave velocity measurements were recorded at approximate 1-meter depth intervals.
3. We advanced four additional Cone Penetration Test (CPT) soundings (C-1, and C-3 through C-5) without seismic testing to depths ranging from about 7.6 to 22 feet, where soundings C-1, C-3, and C-5 encountered refusal to further advancement.
4. We performed a hand auger boring directly adjacent to each of the SCPT/CPT sounding (designated C-1 through C-5), and seven additional hand auger borings (designated HA-1 through HA-7) within proposed parking lot areas to depths of approximately 4 feet each.
   A. Conventional Dynamic Cone Penetrometer (DCP) testing was performed in borings HA-1 through HA-7 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. DCP testing was continued to a depth of approximately 4 feet below the existing ground surface.
5. We performed test pit excavations (TP-1 and TP-2) within the proposed detention pond area to depths of about 7 to 8 feet each, at which depth the test pit excavations began to cave.
6. Small grab samples of subsurface soil materials were collected from representative subsurface strata within the borings and test pits.
7. Two composite bulk samples of the upper soils encountered within the pavement area borings and from within the detention pond test pits were collected for laboratory testing.

A brief description of the field exploration procedures performed, soil classification legends, the SCPT/CPT sounding logs and hand auger boring logs are attached in Appendix II. A Test Location Sketch is included as Figure 2 in Appendix I. The shear wave velocity profile is included as Figure 4 in Appendix I.

## 2.2 Laboratory Testing

After the recovered soil samples were brought to our laboratory, a geotechnical professional examined and/or tested each sample to estimate its distribution of grain sizes, plasticity, organic content, moisture condition, color, presence of lenses and seams, and apparent geologic origin in general accordance with ASTM D 2488, *"Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)"*. The resulting classifications are presented on the logs, included in Appendix II. Similar soils were grouped into representative strata on the logs. The strata contact lines represent approximate boundaries between soil types. The actual transitions between soil types in the field are likely more gradual in both the vertical and horizontal directions than those which are indicated on the logs.

We performed the following quantitative ASTM-standardized laboratory tests on recovered samples, to help classify the soils and formulate our conclusions and recommendations:

- Two bulk samples and four grab samples were tested in general accordance with ASTM D 2216, "*Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*", to measure the in-situ moisture content of the soil.

- Two bulk samples and four grab samples were tested in general accordance with ASTM D 1140, "*Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve*", to measure the percent clay and silt fraction.
- Two bulk samples and four grab samples were tested in general accordance with ASTM D 4318, "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*", to measure the plasticity characteristics of the material.
- Two bulk samples were tested in general accordance with ASTM D 1557, "*Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort*" to characterize the compaction characteristics of the soil.
- One specimen of the bulk sample collected in the future pavement area was recompacted to approximately 95 percent of the modified Proctor maximum dry density and tested in general accordance with ASTM D 1883, "*Standard Test Methods for California Bearing Ratio (CBR) of Laboratory-Compacted Soils*", to evaluate the subgrade support characteristics of the soils.

A summary of the laboratory procedures used to perform these tests is presented in Appendix III. The individual test results are also included in Appendix III.

# 3.0 Site and Surface Conditions

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

## 3.1 Topography

We observed the proposed development area to be relatively level to gently sloping. Based on our review of published aerial imagery, ground surface elevations within the proposed construction areas appear to range from about 84 to 92 feet. Note that it was beyond the scope of our exploration to directly measure ground surface elevations.

## 3.2 Site Surface Conditions

The site was moderately to densely wooded at the time of our exploration. At the time of our exploration, there was no standing water observed to be ponded on the ground surface; however, we observed some water standing in ditches on the site at estimated depths of about 5 to 6 feet below the surrounding ground surface elevations. We also observed what appeared to be a dilapidated residential structure near test locations C-2 and C-4. Topsoil was encountered at the ground surface at each of the test locations, and measured between 4 to 10 inches thick, averaging about 6 inches thick.

## 3.3 Local Geology

The project site is located in the Atlantic Coastal Plain physiographic province, or more specifically, the upper Coastal Plain of South Carolina. The Coastal Plain extends from the eastern limit of the Piedmont ("Fall Line") eastward to the coast and consists of a wedge-shaped deposit of ancient marine sediments of the Late Cretaceous Period and younger. Coastal Plain soils comprise interbedded layers of normally consolidated and overconsolidated limestone, gravels, sands, silts, and clays. This deposit ranges in thickness from near zero at the Fall Line to thousands of feet at the coast. In the site area, depth to crystalline metamorphic rock is likely on the order of about 800 feet.

A review of local geologic mapping indicates that the site area lies within the outcrop area of the Duplin Formation (Td), typically inter-layered terrestrial clays, silts, and sands. These materials weathered in place and have formed a mantle of clayey sands and poorly graded sands at this site which overlie less weathered soils below. The surface has been reworked by erosional processes, and the limestone residuum has been masked by deposits of loose to dense sands or stiff to very stiff clays. The upper contact of the lower sands may be irregular due to localized scouring and redeposition of the overlying clays.

# 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 boring and sounding logs should be reviewed in Appendix II.

## 4.1 Interpreted Subsurface Profile

A subsurface cross-sectional profile of the site soils is attached as Figure 3 in Appendix I to illustrate a general representation of the subsurface conditions within the proposed construction area. The cross-section orientation in plan view is shown on Figure 2. Profile A-A’ (Figure 3) depicts the subsurface conditions across the site, looking in a northerly direction.

The strata indicated in the profile are characterized in the following section. Note that the profile is not to scale and was prepared for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions may vary between test locations.

Soils presented on the profile were grouped into several general strata and substrata based on estimated physical properties derived from the borings, soundings, and the recovered samples. The strata encountered are labeled I through IIIA on the soil profile to allow their properties to be systematically described.

## 4.2 Description of Subsurface Soils

This section describes subsurface soil conditions observed at the site.

### *4.2.1 Stratum I: Upper Loose to Medium Dense Sand Mixtures*

Underlying the topsoil, Coastal Plain sandy sediments consisting of poorly graded sand (USCS Classification ”SP”), poorly graded sand with silt (SP-SM), and silty sand (SM) were encountered within the hand auger borings and test pits to depths of about 7 to 8 feet below the surface, and in the CPT soundings to depths ranging from about 2 to 6 feet.

These soils were typically various shades and combinations of brown, tan, yellow, orange, red, and white in coloration and generally moist to wet upon recovery. The DCP penetration resistance values within the upper 4 feet of these soils ranged from 5 blows per increment (bpi) to 16 bpi, and averaged about 8 bpi, indicating a generally loose relative density with some medium dense layers.

Similar soils were encountered within the CPT soundings to depths ranging from 2 to 6 feet. The tip resistance values in these soils typically ranged from 20 to 150 tsf, and averaged between 30 to 100 tsf, indicating a generally loose to medium dense relative density with some dense lenses.

Laboratory test results on the grab samples and composite bulk samples recovered from this stratum indicated natural moisture contents ranging from 6.2 to 10.8 percent. Silt/clay fines contents ranged from 16.2 to 26.5 percent passing the No. 200 sieve by weight. Atterberg limits plasticity testing indicated that the materials passing the No. 40 sieve of these samples were non-plastic. The modified Proctor maximum dry density of Bulk Sample No. 1 (taken from the detention pond area) was 123.2 pounds per cubic foot at an optimum moisture content of 8.4 percent. The modified Proctor maximum dry density of Bulk Sample No. 2 (taken from the future pavement area) was 127.3 pounds per cubic foot at an optimum moisture content of 7.3 percent. When recompacted to about 95 percent of the modified Proctor maximum dry density, the California bearing ratio (CBR) of Bulk Sample No. 2 was measured to be 34.0 percent at 0.1 inches of penetration.

### *4.2.2 Stratum II: Intermediate Very Stiff to Hard Fine-grained Soils*

Underlying the upper sand mixtures, an intermediate stratum of very stiff to hard fine-grained soils was encountered beginning at depths of 2 to 6 feet in the soundings and continuing to depths ranging from about 5 to 9 feet. Similar soils were encountered in the hand borings beginning at depths of about 1 ½ to 2 feet and continuing to the hand auger boring termination depth of about 4 feet. These upper soils were classified as clayey sands (SC). These soils were typically various shades and combinations of brown, yellow, orange, and red in coloration and generally moist upon recovery. The DCP penetration resistance values measured within these soils ranged from 10 to 16 bpi, and averaged about 13 bpi, indicating a generally medium dense relative density with some loose layers.

CPT tip resistance values within these soils ranged from about 20 to 340 tsf, but were typically within the range of about 100 to 200 tsf, indicating a generally very stiff to hard consistency, with intermittent firm to stiff layers. Shear wave velocity testing within this stratum measured velocities ranging from about 1,040 to 1,060 feet per second (fps).

Laboratory test results on the grab samples recovered from this stratum indicated natural moisture contents ranging from 11.9 to 12.5 percent. Silt/clay fines contents ranged from 36.1 to 40.7 percent passing the No. 200 sieve by weight. Atterberg limits plasticity testing indicated that the samples had liquid limits ranging from 29 to 30 percent, plastic limits ranging from 15 to 17 percent, and plasticity indices ranging from 13 to 14 percent, indicating low to medium plasticity.

### *4.2.3 Stratum III: Lower Medium Dense to Very Dense Sand Mixtures*

Underlying the fine-grained soils of Stratum II, a stratum of sand mixtures was encountered beginning at depths of about 5 to 9 feet and continuing to maximum exploration depth of about 22 feet within sounding C-5. Soundings C-1, C-2, C-3, and C-5 encountered refusal to further advancement at depths ranging from about 7.5

to 22 feet within this stratum. Sounding C-4 was terminated at a target depth of 20 feet within this stratum. The depth at which refusal was encountered is summarized in the following table:

### Table 4-1: Depth of Refusal

| Test Location | Depth of Refusal (feet) | Maximum Tip Resistance Measured (tsf) |
|---|---|---|
| C-1 | 9.3 | 400 |
| C-2 | 20.8 | 380 |
| C-3 | 7.6 | 410 |
| C-5 | 22.0 | 300 |

The CPT tip resistance of the soils of Stratum III ranged from 40 tsf to 410 tsf, but were typically in the range of about 90 to 230 tsf, indicating a generally dense relative density with some medium dense to very dense layers.

Shear wave velocity testing within this stratum measured velocities ranging from about 920 to 1,050 fps.

### *4.2.4 Stratum IIIA: Interbedded Soft Silts*

Interbedded within the Stratum III sandy soils at two of our test locations (C-2 and C-4), a layer of soft silts was encountered beginning at depths ranging from about 14 ½ to 15 feet and continuing to depths of about 17 to 19 feet. The CPT tip resistance of these soils ranged from about 5 to 15 tsf, and averaged about 7 to 10 tsf, indicating a generally soft consistency.

Shear wave velocity testing within this stratum measured a velocity of about 390 fps.

## 4.3 Subsurface Water

Subsurface water was not encountered within any of the hand auger borings at the time of drilling. Water levels were interpreted within the CPT soundings to be about 5 to 6 ½ feet below the surface. Water was encountered in the test pit excavations at depths of about 6 ½ to 7 ½ feet below the ground surface.

Based on the soil types encountered, this site may be susceptible to the development of a shallow perched water table, particularly during times of wet weather. 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.

## 4.4 Summary of Laboratory Test Results

We performed laboratory testing on four grab samples and two bulk samples to further assess the engineering index properties of the subsurface soils. The laboratory soil index test results are presented in Appendix III and are summarized in the following tables.

**Table 4-2: Summary of Laboratory Soil Index Testing Results**

| Boring/ (Sample No.) | Sample Depth Range (ft) | Natural Moisture Content (%) | Silt/Clay Fines Content (%) | Atterberg Plasticity Limits (%) | | | USCS Classification |
|---|---|---|---|---|---|---|---|
| | | | | LL | PL | PI | |
| TP-1 & TP-2 /(BULK1) | 0.5 – 5 | 10.8 | 16.2 | --- | NP* | --- | SM |
| HA-1 to HA-4 /(BULK2) | 0.5 – 2 | 6.9 | 19.1 | --- | NP* | --- | SM |
| C-1/(S-1) | 0.5 – 2 | 6.2 | 26.5 | --- | NP* | --- | SM |
| C-4/(S-1) | 0.5 – 2 | 6.5 | 20.6 | --- | NP* | --- | SM |
| HA-2/(S-2) | 2 – 4 | 11.9 | 40.7 | 29 | 15 | 14 | SC |
| HA-5/(S-3) | 3 – 4 | 12.5 | 36.1 | 30 | 17 | 13 | SC |

*NP = Non-plastic

**Table 4-3: Summary of Moisture-Density and CBR Test Results**

| Boring /(Sample No.) | Modified Proctor Maximum Dry Density (pcf) | Modified Proctor Optimum Moisture Content (%) | CBR at 95.1% Compaction (%) |
|---|---|---|---|
| TP-1 & TP-2/(BULK 1) | 123.2 | 8.4 | --- |
| HA-1 to HA-4/(BULK 2) | 127.3 | 7.3 | 34.0 |

# 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, 2023, the 2021 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.

## 5.2 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 site modified peak ground acceleration of 0.246 g.

### *5.2.1 Liquefaction Potential Index (LPI)*

To evaluate liquefaction potential, we performed analyses using the data obtained in the soundings, considering the characteristics of the soil and water levels observed in the soundings. The liquefaction analysis was performed based on the design earthquake prescribed by the 2021 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 less than 5; therefore, the risk of surface damage due to liquefaction under seismic shaking is generally low and Site Class F does not apply to this site. 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.

## 5.3 Selection of Seismic Site Class based on Shear Wave Velocity

Based upon the measured and extrapolated shear wave velocity, this site is determined to be **Site Class D**. This recommendation is provided based on the shear wave velocity measured at test sounding C-2, measured to a depth of about 21 feet, and then extrapolated to a depth of 100 feet. The average weighted shear wave velocity was measured to be 744 feet per second (fps) in the upper 21 feet. When extrapolated to a depth of 100 feet, an average shear wave velocity of 988 fps is estimated, which is greater than the minimum of 600 fps that is required for consideration of Site Class D design parameters, but less than the 1,200 fps that is required for consideration of Site Class C design parameters. See Figure 4 in Appendix I for the shear wave velocity profile.

### 5.4 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 | Seismic Site Class | $S_S$ | $S_1$ | $S_{DS}$ | $S_{D1}$ | $PGA_M$ | Seismic Design Category | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Risk Cat. I-III | Risk Cat. IV |
| 2021 IBC ASCE 7-16 | D | 0.311 | 0.113 | 0.322 | 0.179 | 0.246 | C | D |

### 5.5 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 C" as defined in section 1613.2.5 of the IBC as adopted by South Carolina. For a structure having a Risk Category classification of IV, the $S_{DS}$ and $S_{D1}$ values obtained are consistent with "Seismic Design Category D" as defined in section 1613.2.5 of the IBC as adopted by South Carolina.

## 6.0 Conclusions and Recommendations

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

When reviewing these recommendations, please note that a portion of the site appears to have been previously developed with a residential structure. Experience with previously developed sites indicates that unexpected conditions often exist. These may include active or abandoned utility lines, areas of poorly compacted fill, and other conditions. Such conditions, if encountered, can typically be addressed by on-site evaluations performed by a representative of the Geotechnical Engineer at the time of construction.

## 6.1 Demolition of Existing Structures and Utilities

The following general recommendations are provided for the demolition of underground structures and utilities in areas where they occur within the footprint of the proposed new structures:

1. Where encountered, demolish existing foundations, asphalt, or concrete within the footprint of proposed new structures, foundations, or pavements.
2. Subsurface structures (septic tanks, foundations, utilities, or drain lines) should be removed in their entirety and backfilled in accordance with Section 6.4 of this report.
3. Following demolition of the existing structures, remove or plug existing utilities that are to be permanently abandoned. If not removed or plugged, abandoned pipes may serve as conduits for subsurface erosion resulting in formation of voids below foundations, grade slabs, or pavements.
4. Reroute existing utilities that will remain in use around the proposed new structures and their foundations.
5. Areas excavated to remove any buried structures or utilities should be backfilled in accordance with Section 6.4 of this report.

## 6.2 Surface Preparation

Site preparation will include demolition of stripping of surface vegetation, removal of about 4 to 10 inches of topsoil and rootmat. The following recommendations are provided regarding site preparation and earthwork.

1. Drainage should be implemented and maintained prior to and during construction to divert water away from the construction area. Surface and subsurface water conditions that occur during construction will determine the need for and extent of drainage measures. Water levels should be maintained at least 3 feet below the working grade elevations during construction.
2. Strip surface vegetation and organic-laden or debris-laden soils where encountered and dispose of outside the building and pavement area footprints.
3. After the surface has been stripped, the existing subgrade surface should be thoroughly densified in place with a heavy vibratory roller prior to placement of any new fill.

   A. Under favorable moisture conditions and with the proper equipment, this may be able to be accomplished by densifying the soil from the working surface. However, under less favorable conditions, it may be necessary for the contractor to re-work (or remove, condition, and replace) the material, using moistening or drying techniques, in order to achieve the desired level of compaction.
   B. The densification of these soils should be performed under the observation of an S&ME representative.

4. After surface densification, but prior to placement of any new fill, have a representative of the Geotechnical Engineer observe the prepared surface for stability. This may consist of a visual observation of a proofroll, performed by the contractor, in all areas to receive fill by 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 should be a decision made at the time of construction based on the conditions observed.

A. Based on relatively low CPT tip resistances measured at test locations C-3 through C-5 in the upper 2 to 3 feet, loose sands may be encountered after stripping. We anticipate these areas may exhibit low density and instability, so undercutting and replacement of the upper 2 to 3 feet of the soils may be required if the surface densification efforts are unsuccessful in achieving a stable subgrade.
B. After removal of any unstable soils, the excavation bottoms should be thoroughly densified with a large vibratory roller.
C. Undercut soils may be reused as backfill into the excavations provided they are free of organic or other deleterious materials and meet the fill soil requirements described in Section 6.4.
D. The undercut areas should be backfilled and compacted in accordance with Section 6.4 of this report.

## 6.3 Excavation

1. Subsurface water is estimated to be encountered at a depth of approximately 5 to 6 ½ feet below the existing ground surface. If subsurface water is encountered during excavations, the water level should be maintained at least 3 feet below excavations to help maintain bottom stability. The effects of dewatering on nearby structures should be evaluated and are the responsibility of the designer of any dewatering system, which was beyond the scope of this report.
2. All excavations should be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards for Type C soils. The contractor is solely responsible for site safety. This information is provided only as a service, and under no circumstances should S&ME be assumed to be responsible for construction site or excavation safety.
3. Hydraulic excavators should be able to excavate throughout the soil profile.
4. Loose sand mixtures may be encountered at the ground surface and may continue to depths of around 2 to 3 feet in some areas of the site. Where these soils are encountered in excavation bottoms such as footings or pipe trenches, it may become necessary to over-excavate the soft/loose materials and replace with an open-graded, coarse, manufactured granitic gravel such as SCDOT No. 57 stone to create a stable excavation bottom.

## 6.4 Fill Placement and Compaction

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

1. Before beginning to place fill, sample and test each proposed fill material to determine its suitability for use, maximum dry density, optimum moisture content, and natural moisture content. It is recommended that any imported fill soils used to build up the embankment for structures or pavements meet the following minimum requirements:

   A. Natural moisture content within +/- 3 percent of the optimum moisture content for compaction as determined by ASTM D 1557.
   B. Plasticity index of 15 percent or less (ASTM D 4318).
   C. Clay/silt fines content of not greater than 35 percent by weight (ASTM D 1140).
   D. Organic content less than 5 percent by weight (ASTM D 2974).
   E. Soaked CBR value of at least 10 percent when recompacted to 95 percent of the modified Proctor maximum dry density near optimum moisture (ASTM D 1883).

2. The laboratory test results on the composite bulk sample we collected from the detention pond area indicates that the near-surface silty sands, poorly graded sands with silt, and poorly graded sands of Stratum I to depths of at least 7 to 8 feet are likely suitable to be re-used as structural fill on the site. The clayey sands of Stratum II may also be considered for use as structural fill, but are less preferred due to their higher plasticity and fines content; these clayey sands may be used with some acceptance of risk associated with grading difficulties if they are allowed to become wet or if grading occurs during wetter periods of the year.
   - A. The natural moisture content of the composite sample collected from the upper 5 feet of the pond area soils was 10.8 percent, compared to an optimum moisture content for compaction of 8.4 percent. This indicates that, where measured, the in-situ soils may be about 2.4 percent wet of their optimum moisture content for compaction, indicating that some moisture conditioning of these materials may be required prior to re-compacting them as fill.
   - B. The natural moisture content of the composite sample collected from the upper 2 feet of the future pavement area soils was 6.9 percent, compared to an optimum moisture content for compaction of 7.3 percent. This indicates that, where measured, these in-situ soils may be about 0.4 percent dry of their optimum moisture content for compaction.
   - C. Soils borrowed from below the water table may be significantly wet of the optimum moisture content for compaction and will likely need to be rigorously worked and dried.

3. Where fill soil is required, structural fill should be compacted throughout to at least **95 percent** of the modified Proctor maximum dry density (ASTM D 1557).
   - A. Compacted soils must not exhibit pumping or rutting under equipment traffic.
   - B. Loose lifts of fill should be no more than 12 inches in thickness prior to compaction, reduced to no more than 6 inches when using small walk-behind or portable compaction equipment.
   - C. Structural fill should extend at least 5 feet beyond the edge of pavements or structures before either sloping or being allowed to exhibit a lower level of compaction.

4. Fill placement should be observed 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 per each 300 cubic feet of fill placed in confined areas such as isolated undercuts and in trenches, with a minimum of 1 test per lift.

## 6.5 Shallow Foundations

The soil profile encountered appears generally suitable to support the proposed buildings with shallow foundations considering static loading conditions and the assumed maximum wall loads. The design engineer needs to confirm that the assumed maximum loads and fill height assumptions are correct; if actual loads are higher than assumed or if more than 2 feet of fill is added to the site, 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 buildings:

1. Provided that the recommendations in Sections 6.1 through 6.4 of this report are implemented, the proposed structures may be supported on shallow foundations using isolated spread footings and slab-on-grade construction as planned. A net available bearing pressure of up to **2,500 psf** may be used for design of individual spread footings and wall footings that are extended to bear within densified native Coastal Plain deposits or within structural fill compacted as recommended in Section 6.4 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 1605.2, unless otherwise restricted by design code provisions.
3. Dynamic Cone Penetrometer (DCP) testing should be performed by a representative of the Geotechnical Engineer at representative locations within the open foundation excavations in order to evaluate the available bearing capacity of the soils.

    A. In cases where the DCP testing indicates insufficient bearing capacity for the applied bearing pressure, the foundation bearing soils may need to be over-excavated and replaced with stronger material.
    B. Where soft or loose soils are encountered within the footing excavations and are removed, we recommend that an open-graded, coarse, washed, crushed, manufactured granitic gravel meeting the gradation requirements of SCDOT No. 57 or No. 67 stone be used as the backfill material to replace any over-excavated soils. The depth of the recommended over-excavation will need to be a case-by-case decision made at the time of construction, by the Geotechnical Engineer, based upon the results of the DCP testing, but is likely to be on the order of 1 to 3 feet below design bearing grade.
    C. Also, have the Geotechnical Engineer's representative observe any undercut areas in footings prior to backfilling, in order to confirm that the poor soils have been removed and that the exposed subgrade is suitable for support of footings or backfill.
    D. 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. Also, foundations which are extended to bear within loose sands are likely to require densification of the bearing surfaces after excavation and prior to footing construction. This process may also involve moisture-conditioning of the bearing soils.

4. Even if smaller dimensions are theoretically allowable from a bearing pressure consideration, the minimum column footing width should be 30 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.
5. 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.

6. The following discussion is provided regarding the estimated magnitude of post-construction settlements under static loading.
   - A. Based on an assumed maximum column load of 40 kips, and considering a uniformly applied area load (slab loading + slab self-weight + fill) of 400 psf, and a 2,500 psf shallow foundation bearing pressure, the estimated total static settlement of an individual spread footing measuring roughly 4 feet by 4 feet in plan area will likely be 1 inch or less.
   - B. Based on an assumed wall load of 3.5 kips per linear foot, and considering a uniformly applied area load of 400 psf, and a 2,500 psf shallow foundation bearing pressure, the estimated static post-construction settlement of an individual wall strip footing at least 1.5 feet wide will likely be 1 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.

## 6.6 Grade Slab Support and Construction

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

1. Soils similar to those penetrated by the borings should provide adequate support to proposed soil-supported grade slabs, assuming preparation and compaction of the subgrade as recommended above. Soil-supported thin slabs on grade may be designed using a modulus of subgrade reaction (k) of 175 pci provided that the subgrade is prepared as recommended in this report.
2. Structural design should consider incorporating the 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 silty sand subgrade and the floor slab in finished spaces.
   - A. Granular materials used as the underslab layer may consist of clean sandy soils meeting USCS Classification SP or SW and having a silt-clay fines content of 5 percent or less by weight, or, a crushed, well-graded gravel blend meeting the requirements of Macadam Graded Aggregate Base Course (GABC) as described in Section 305 of the South Carolina Department of Transportation (SCDOT) Standard Specifications for Highway Construction, 2007 edition, or an open-graded, manufactured washed gravel meeting the gradation requirements of SCDOT No. 57 or No. 67 stone.
   - B. 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 traverse with vehicles. If GABC is used, then either a pump truck or direct discharge from concrete batch trucks may be appropriate depending upon the circumstances.
   - C. If sand or GABC is used, this underslab layer should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) as determined by field density testing.

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.

## 6.7 Lateral Earth Pressures

The lateral earth pressure coefficients given below may be used to design below grade earth retaining structures.

The values given in Table 6-1 assume placement and compaction of backfill around and behind these structures in accordance with Section 6.4 of this report. These earth pressures were estimated under the assumption that materials similar to the on-site Stratum I silty or clayey sands would likely be used as the backfill material.

### Table 6-1: Lateral Earth Pressure Coefficients

| Support Condition | Angle of Internal Friction (φ') | Moist Unit Weight (γ) (pcf) | Buoyant Unit Weight (γ) (pcf) | Drained Static Earth Pressure Coefficient (K) | Drained Seismic Earth Pressure Coefficient (K) $PGA_M$=0.246g |
|---|---|---|---|---|---|
| Active ($K_a$) | 30 | 120 | 58 | 0.33 | 0.40 |
| At-Rest ($K_o$) | 30 | 120 | 58 | 0.50 | 0.60 |
| Passive ($K_p$) | 30 | 120 | 58 | 3.00 | 2.81 |

A. The above values represent a fully-drained soil condition at or near the optimum moisture content. Where backfill soils are not fully drained, the lateral soil pressure must consider hydrostatic forces below the water level, and submerged soil unit weight.
B. A coefficient of sliding friction (tan δ) of 0.36 may be used in computation of the lateral sliding resistance.

## 6.8 Pavement Design and Construction

We understand that site pavements may consist of both flexible (asphalt) and rigid (concrete) pavements. Based upon the assumption that the direct pavement support soils will consist of compacted fill or densified native sand mixtures similar to the bulk sampled soil that we tested in our laboratory, we estimate that an average combined California Bearing Ratio (CBR) value of at least 10 percent will be available for pavement support. This results in a resilient modulus of at least 11,153 psi available for flexible pavement design. This assumes that any fill materials used in the upper 2 feet beneath pavements will have a soaked CBR value of at least 10 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 local practice for typical pavement section thicknesses and assumed traffic loading parameters. These pavement section components are provided in Table 6-2 below. The design civil engineer should confirm our traffic assumptions.

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 0.9 was assigned, based upon the assumption that the sub-base soils will consist of native silty sands or 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 will be unreinforced, we used an average load transfer coefficient of 3.8. 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 0.9 was assigned.

Traffic frequency and loading data was not provided prior to our issuing this report; therefore, our evaluation for the light duty pavements at this site considered a lifetime traffic demand of roughly 40,000 ESALs, and evaluation for the heavy-duty pavements at this site considered a lifetime traffic demand of roughly 200,000 ESALs. If the actual traffic loadings are greater that the assumed, then thicker pavement sections may be required. Based on the demand conditions evaluated, we estimate that pavement sections shown in Table 6-2 below are appropriate for the corresponding traffic loading scenarios with reasonable factors of safety.

**Table 6-2: Recommended Minimum Pavement Sections(a)**

| Pavement Type | Theoretical Available Traffic Capacity (ESALs) | HMA Surface Course Type C (inches) | HMA Intermediate Course Type C (inches) | 4,000 psi Unreinforced PCC (inches) | SCDOT Section 305 Graded Aggregate Base Course [GABC] (inches) |
|---|---|---|---|---|---|
| Light-Duty Flexible HMA (no trucks) | 42,000 | 2.0 | --- | --- | 6.0 |
| Heavy-Duty Flexible HMA (with truck traffic) | 230,000 | 1.5 | 2.0 | --- | 6.0 |
| Light-Duty Rigid PCC (no trucks) | 91,000 | --- | --- | 5.0 | 6.0 |
| Heavy-Duty Rigid PCC (with truck traffic) | 221,000 | --- | --- | 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.

S&ME did not factor the traffic by direction or by lane, nor did we make any reduction in the ESAL loading to account for two or more points of access or egress.

### 6.8.1 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, which was 10 percent.

2. All fill placed in pavement areas should be compacted as recommended in Section 6.4 "Fill Placement and Compaction".

### 6.8.2 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 at final soil subgrade (FSG) elevation by the contractor under the observation of a representative 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). The base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC-T-140).
   A. The GABC should consist of either Macadam or Marine Limestone base course as defined in Section 305 of the above-referenced specification.
   B. Do not allow the substitution of Coquina shell type base course for the specified GABC material.
   C. It is not recommended to allow the substitution of "commercial base" (i.e., non-SCDOT compliant) base course materials.
3. After placement of base course stone and shortly prior to paving, the surface should be methodically proofrolled at final base grade (FBG) elevation by the contractor under the observation of a representative 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.
4. 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.
5. Construct the surface course HMA in accordance with the specifications of Sections 401 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.

8. For rigid pavements, we recommend air-entrained ASTM C 94 Portland cement concrete that will achieve a minimum compressive strength of at least 4,000 psi at 28 days after placement, as determined by ASTM C 39. We also recommend that the pavement concrete be constructed in a manner which at least meets the minimum standards recommended by the American Concrete Institute (ACI).
9. We recommend that at least 1 set of 5 cylinder specimens be cast by S&ME per every 50 cubic yards of concrete placed or at least once per placement event in order to measure achievement of the design compressive strength. We also recommend that S&ME be present on site to observe and test the concrete placement.

# 7.0 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations 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 site.

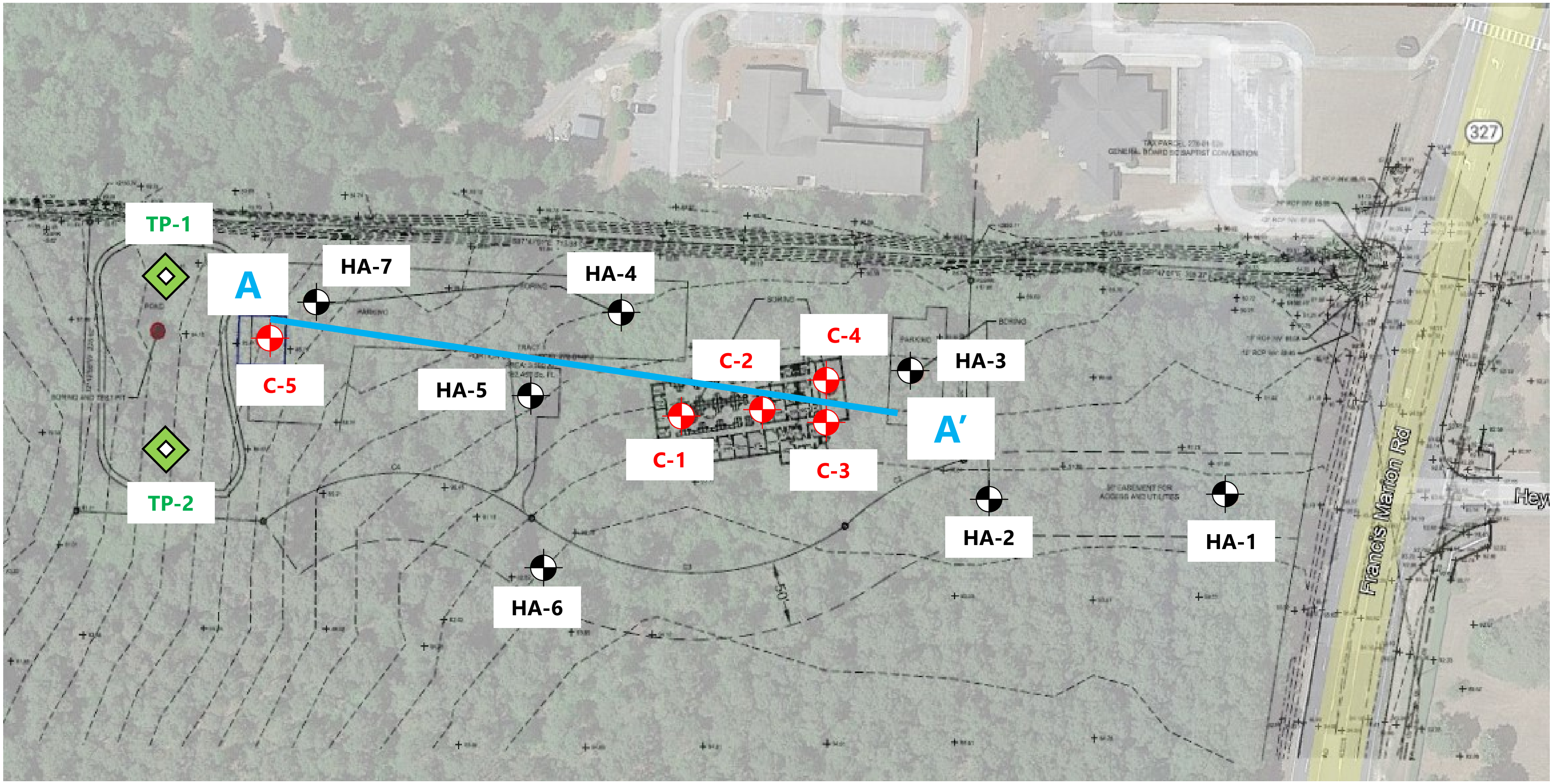
Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

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 testing of earthwork and foundation construction activities.

# Appendices

# Appendix I – Figures


| | | | |
|---|---|---|---|
| | **Site Vicinity Map** | SCALE: NTS | FIGURE NO. 1 |
| | SLED Pee Dee Regional Office<br>Florence, South Carolina | DATE: 6-23-2023<br>PROJECT NO. 23390074 | |




**LEGEND**

= Approximate CPT Test Location

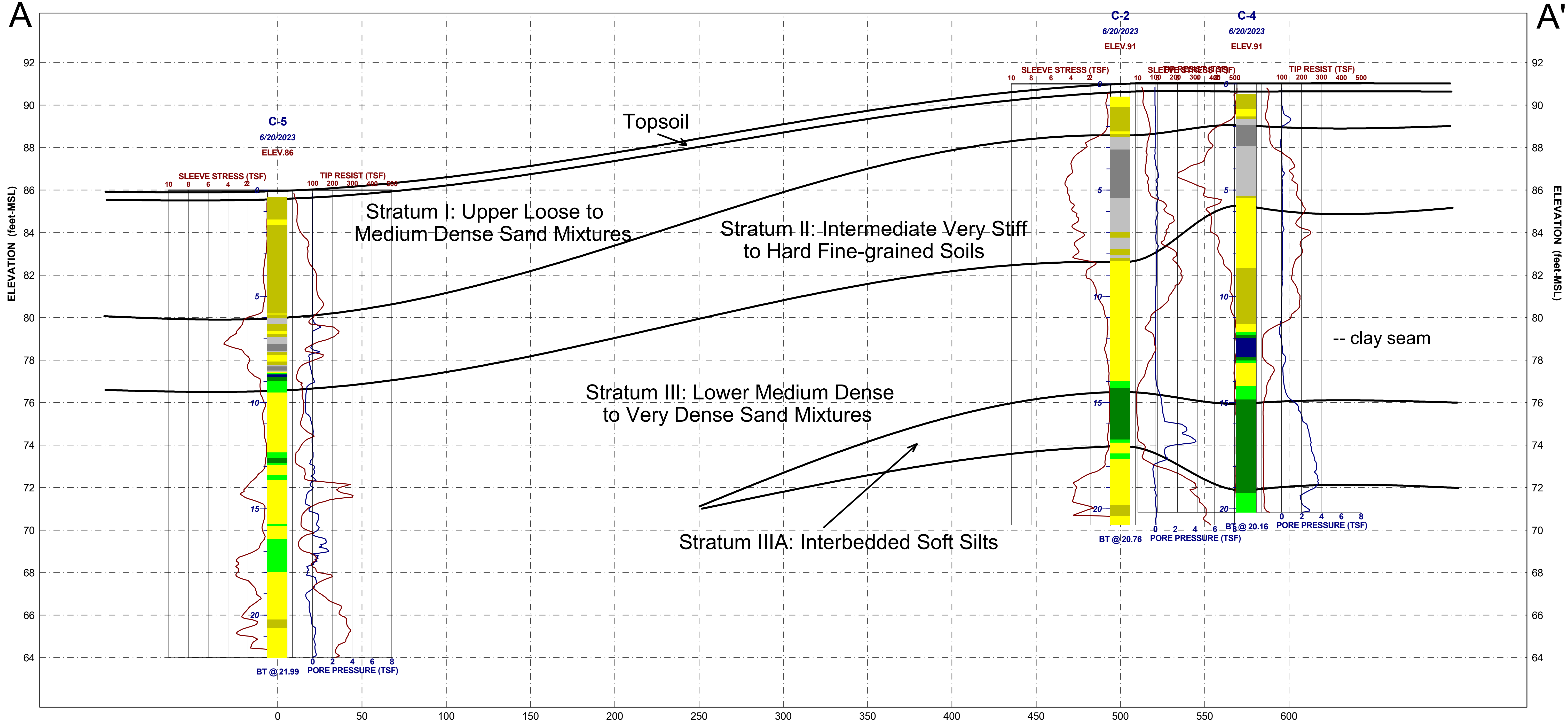
= Approximate Hand Auger Test Location

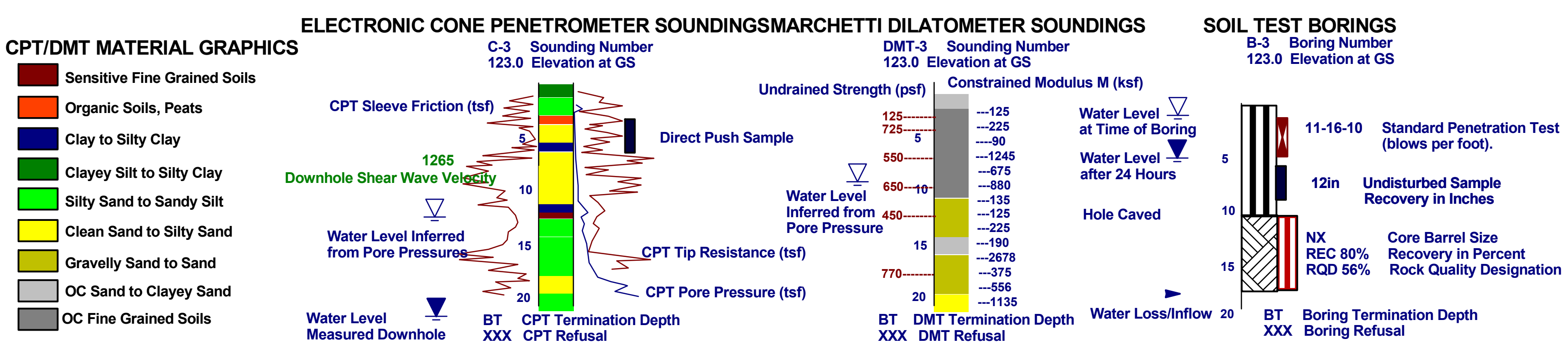
= Approximate Test Pit Location

= Subsurface Profile

NORTH

| | | | |
|---|---|---|---|
| | Test Location Sketch | SCALE: NTS | FIGURE NO. 2 |
| | SLED Pee Dee Regional Office<br>Florence, South Carolina | DATE: 6-23-2023<br>PROJECT NO. 23390074 | |




LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

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

**FIGURE 3: SUBSURFACE PROFILE**

PROJECT: SLED Pee Dee Regional Office

LOCATION: Florence, South Carolina

JOB NO: 23390074

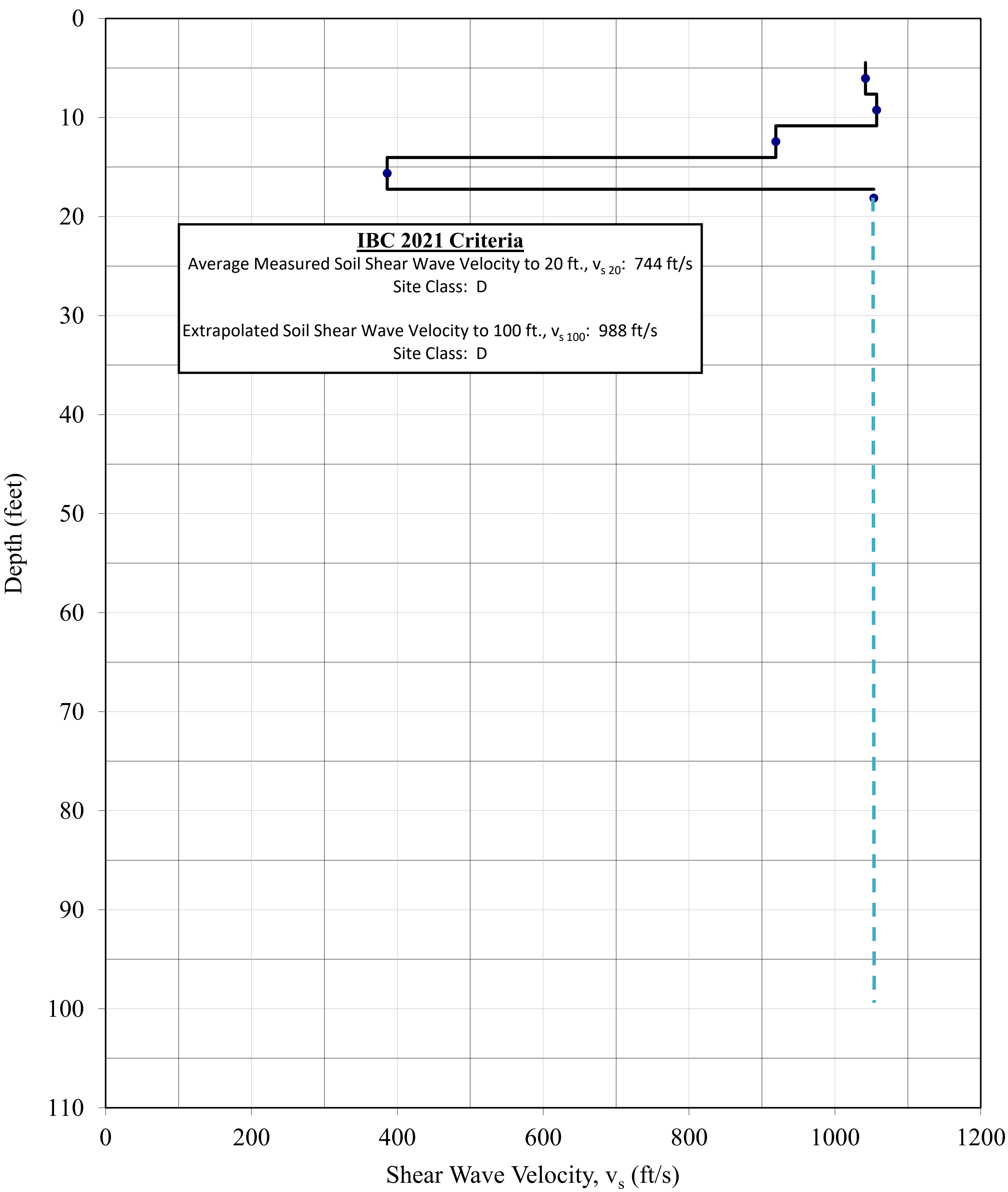
DATE: 6/23/23

## FIGURE 4: Shear Wave Velocity Calculations

*SLED Pee Dee Regional Office*
Florence, SC

Sounding ID: **C-2**
Date: 06/20/23

Project Number: **23390074**




* Site Class based on 2021 International Building Code - *Table 1613.5.2 - SITE CLASS DEFINITIONS*

# Appendix II – Exploration Procedures and Data

## 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 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.

## Refusal to CPT Push

Refusal to the cone penetrometer equipment occurred when the reaction weight of the CPT rig was exceeded by the thrust required to push the conical tip further into the ground. At that point the rig tended to lift off the ground. Refusal may have resulted from encountering hard cemented or indurated soils, soft weathered rock, coarse gravel, cobbles or boulders, thin rock seams, or the upper surface of sound continuous rock. Where fills are present, refusal to the CPT rig may also have resulted from encountering buried debris, building materials, or objects.

## 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 with Dynamic Cone Penetrometer

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 glass jars or plastic bags and later transported to the laboratory. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. At 1 foot intervals, the augers were withdrawn and soil consistency was 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 and ability to support foundations.

## Hand Auger Borings without Dynamic Cone Penetrometer

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 glass jars or plastic bags and later transported to the laboratory. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers.

## Excavated Test Pits

Test pits were excavated by a subcontractor of S&ME to obtain information about soil conditions of the material to be used as fill. Test pits allow closer observation of the soil composition with depth and give an indication of excavation difficulty with a specific type of equipment during construction. A field engineer was present to observe the soil strata exposed in each pit, estimate the relative ease of excavation, the amount of subsurface water entering the pits, and the maximum depth the pits were excavated. After completion of excavation, the pits were immediately backfilled with the spoil material; however, since the pits were narrow, deep excavations, very limited compactive effort could be applied to the backfill. Backfill was bucket-tamped during placement. The backfill was heaped up slightly above the level of the ground surface, but the client should recognize that some ground settlements may occur at the surface in the vicinity of the test pits.

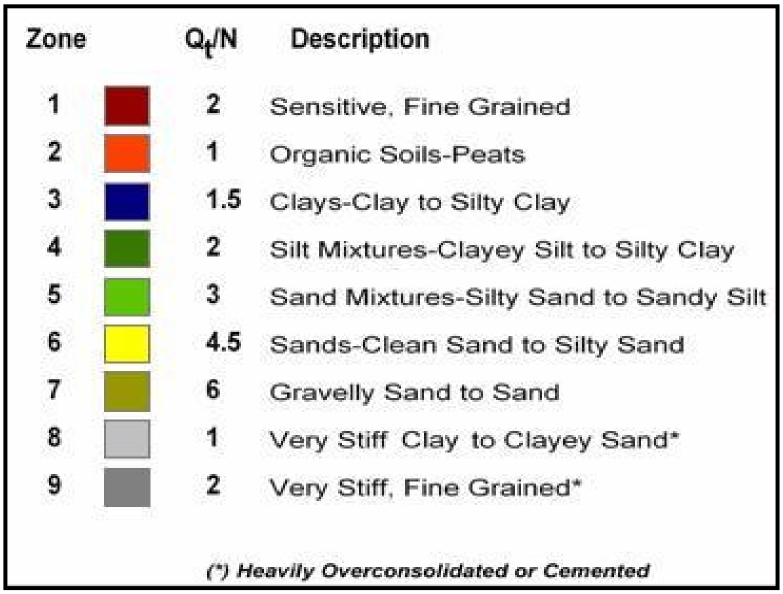
## Water Level Measurement

Subsurface water levels in each sounding were measured via pore water pressure readings and corresponding depths from the existing grade. Subsurface water was measured from existing ground surface in the hand auger borings and test pits using a tape measure, where encountered.

## Backfilling of Boreholes

Upon completion of the boreholes and measurement of the water level in the hole, each boring was backfilled with natural soil cuttings 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 | |

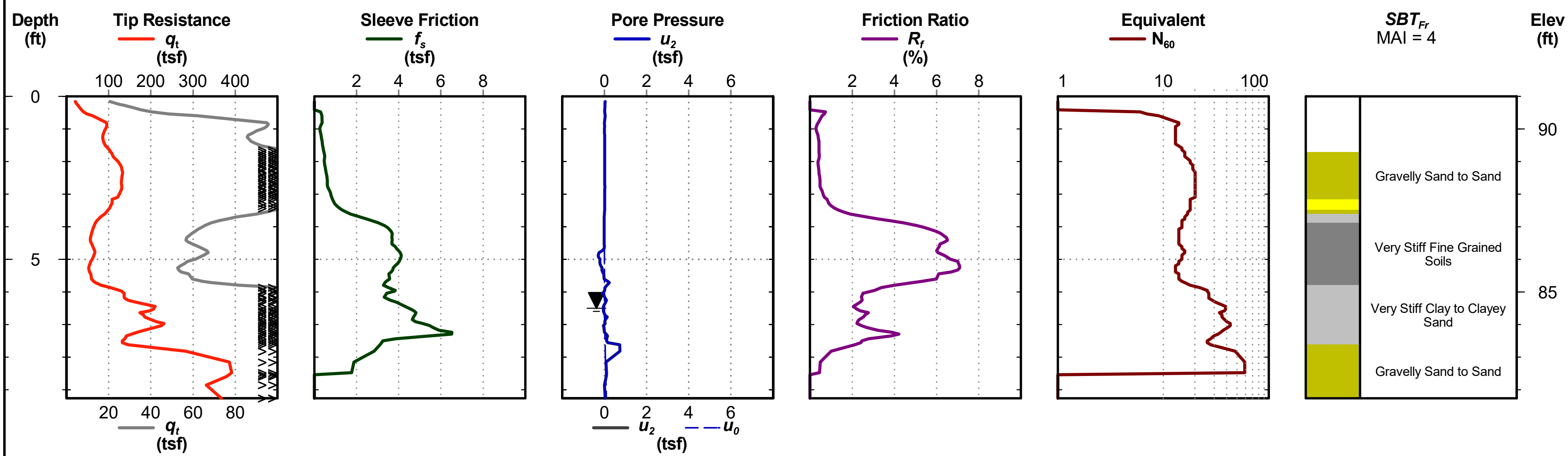
Soil behavior type is based on empirical data and may not be representative of soil classification based on plasticity and grain size distribution.

| 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-1

**SLED Pee Dee Regional Office**
**Florence, South Carolina**
**S&ME Project No: 23390074**

**Date:** Jun. 20, 2023
**Estimated Water Depth:** 6.5 ft
**Rig/Operator:** Daniel Coffee | Luke Greene
**Total Depth:** 9.3 ft
**Termination Criteria:** Maximum Reaction Force
**Cone Size:** 1.75




## Cone Penetration Test



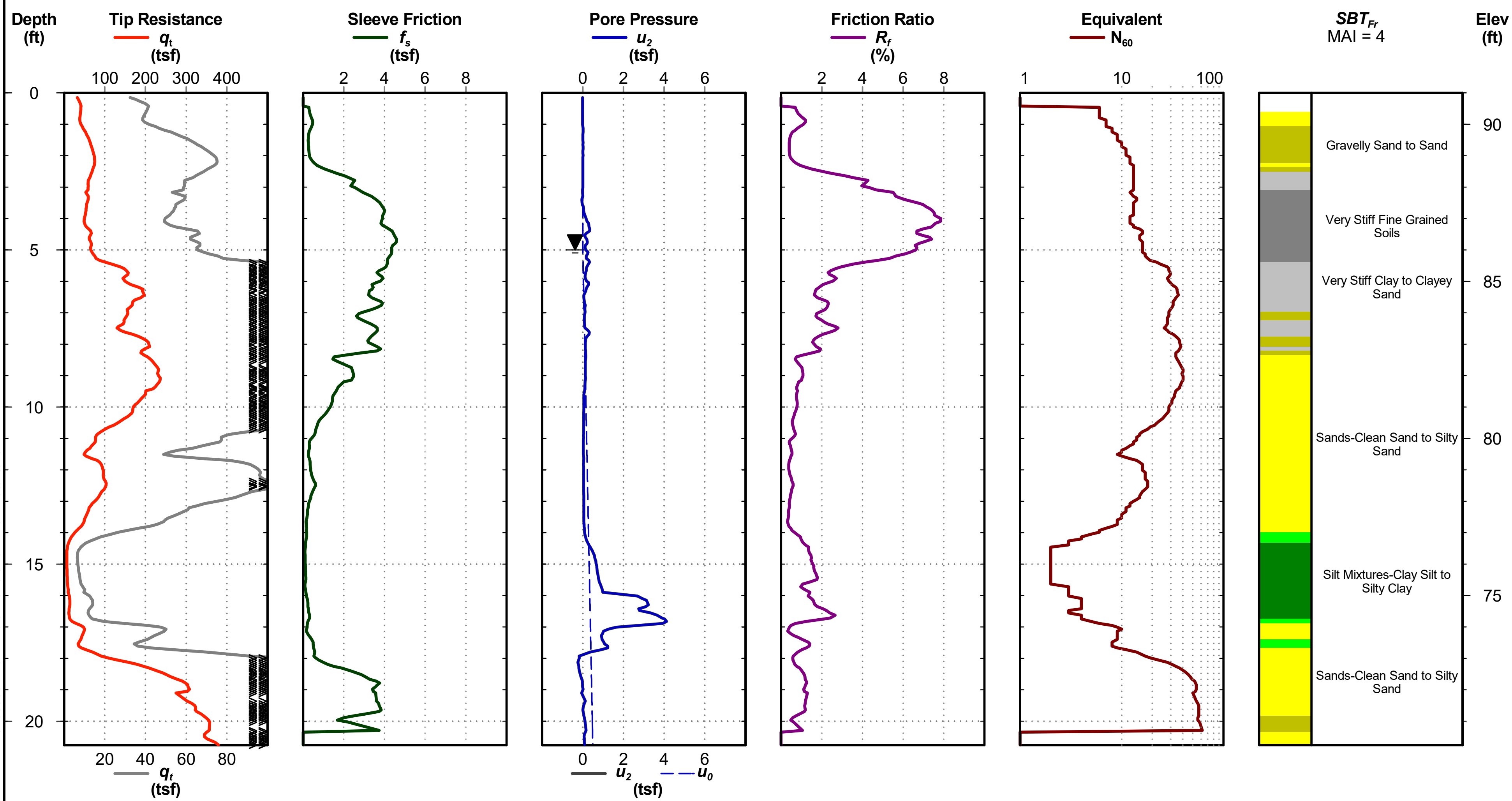
Electronic Filename: C-02.DAT

# SLED Pee Dee Regional Office
# Florence, South Carolina
# S&ME Project No: 23390074

## Sounding ID: C-2

**Date:** Jun. 20, 2023
**Estimated Water Depth:** 5 ft
**Rig/Operator:** Daniel Coffee | Luke Greene

**Total Depth:** 20.8 ft
**Termination Criteria:** Target Depth
**Cone Size:** 1.75




CPT REPORT - STANDARD - SBT FR \ 23390074 CPT LOGS.GPJ \ S&ME.GDT \ 6/23/23

## *Cone Penetration Test*

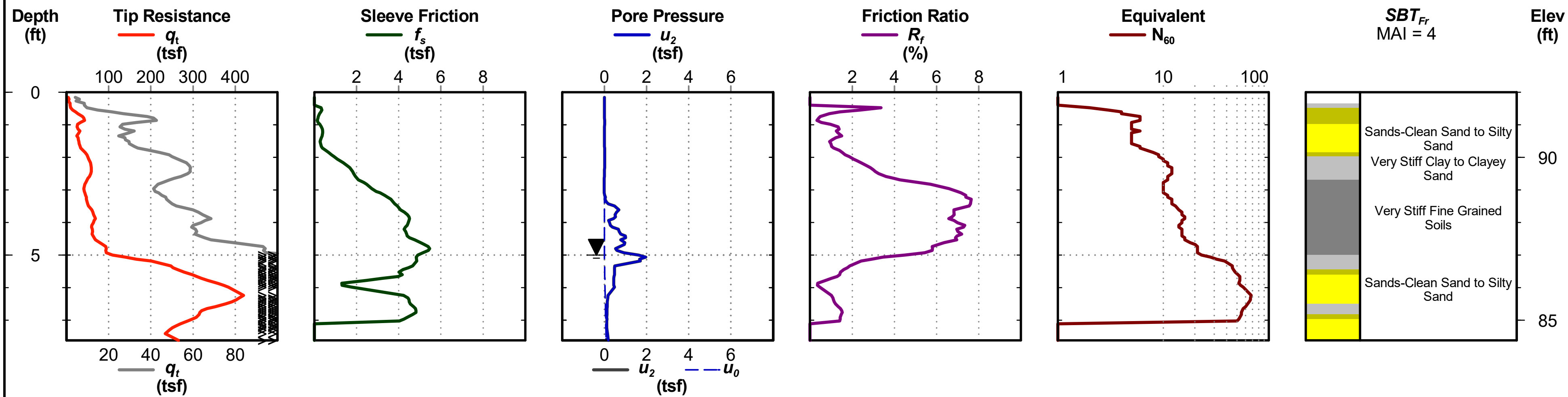


*Electronic Filename: C-02.DAT*

# Sounding ID: C-3

**SLED Pee Dee Regional Office**
**Florence, South Carolina**
**S&ME Project No: 23390074**

| | | | |
|---|---|---|---|
| **Date:** | Jun. 20, 2023 | **Total Depth:** | 7.6 ft |
| **Estimated Water Depth:** | 5 ft | **Termination Criteria:** | Maximum Reaction Force |
| **Rig/Operator:** | Daniel Coffee \| Luke Greene | **Cone Size:** | 1.75 |




# Cone Penetration Test



Electronic Filename: C-03.DAT

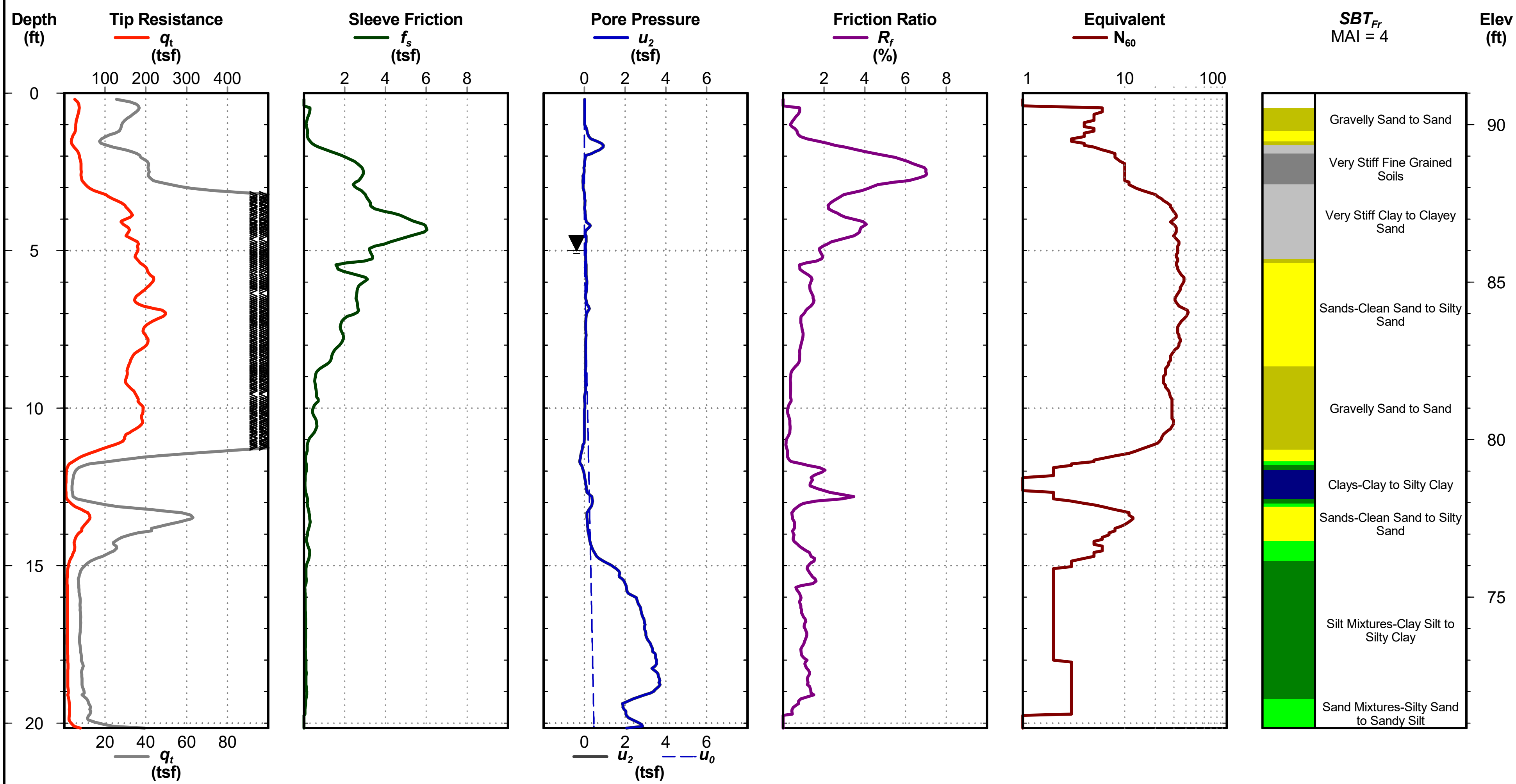
CPT REPORT - STANDARD - SBT FR \ 23390074 CPT LOGS.GPJ \ S&ME.GDT \ 6/23/23

# Sounding ID: C-4

**SLED Pee Dee Regional Office**
**Florence, South Carolina**
**S&ME Project No: 23390074**

**Date:** Jun. 20, 2023
**Estimated Water Depth:** 5 ft
**Rig/Operator:** Daniel Coffee | Luke Greene

**Total Depth:** 20.2 ft
**Termination Criteria:** Target Depth
**Cone Size:** 1.75




CPT REPORT - STANDARD - SBT FR \ 23390074 CPT LOGS.GPJ \ S&ME.GDT \ 6/23/23

## Cone Penetration Test

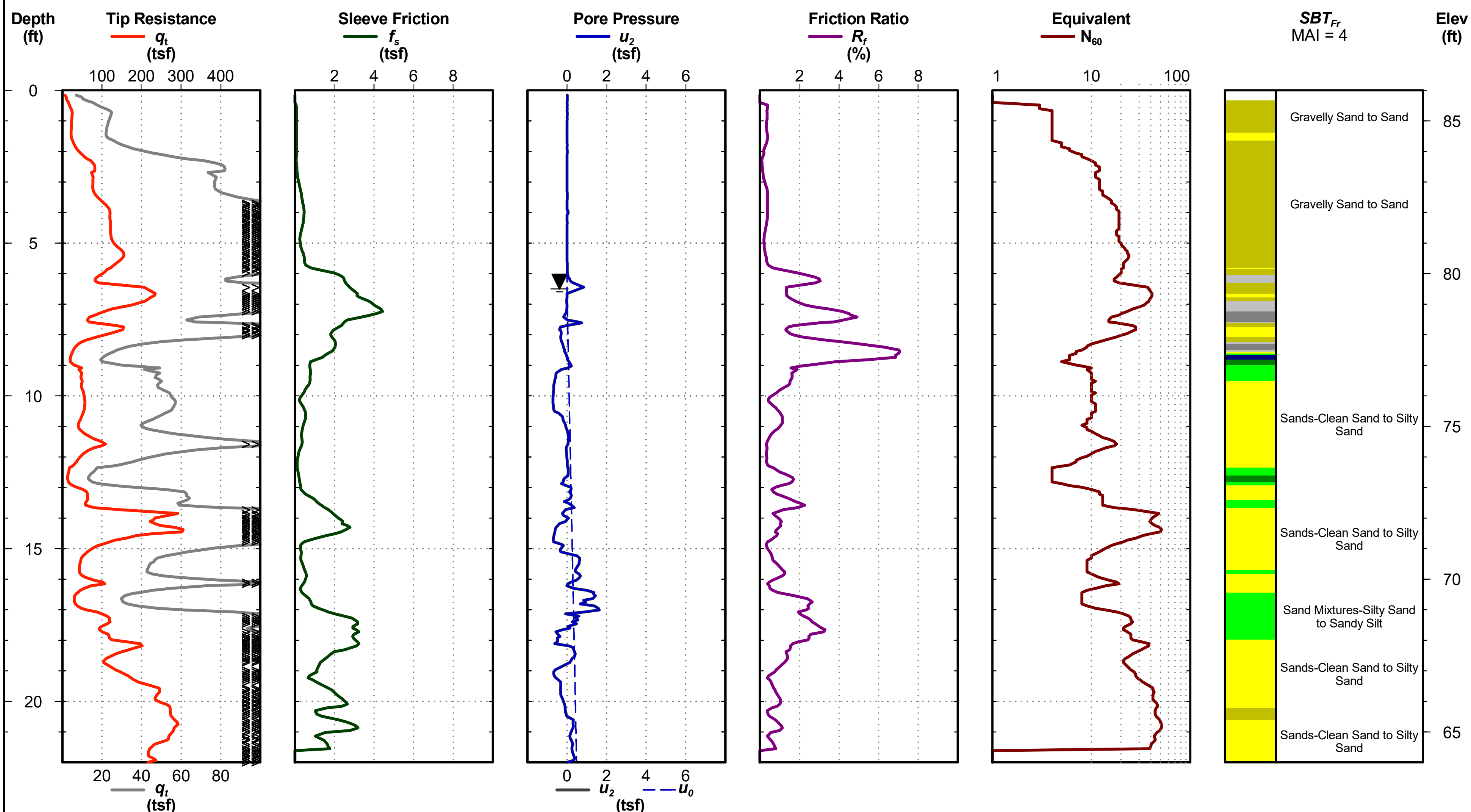


Electronic Filename: C-04.DAT

# Sounding ID: C-5

SLED Pee Dee Regional Office
Florence, South Carolina
S&ME Project No: 23390074

**Date:** Jun. 20, 2023
**Estimated Water Depth:** 6.5 ft
**Rig/Operator:** Daniel Coffee | Luke Greene
**Total Depth:** 22.0 ft
**Termination Criteria:** Maximum Reaction Force
**Cone Size:** 1.75




# Cone Penetration Test



Electronic Filename: C-05.DAT

# LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

## SOIL TYPES

(Shown in Graphic Log)

- Fill
- Asphalt
- Concrete
- Topsoil
- Gravel
- Sand
- Silt
- Clay
- Organic
- Silty Sand
- Clayey Sand
- Sandy Silt
- Clayey Silt
- Sandy Clay
- Silty Clay
- Partially Weathered Rock
- Cored Rock

## WATER LEVELS

(Shown in Water Level Column)

▽ = Water Level At Termination of Boring
▼ = Water Level Taken After 24 Hours
◄ = Loss of Drilling Water
HC = Hole Cave

## CONSISTENCY OF COHESIVE SOILS

| CONSISTENCY | STD. PENETRATION RESISTANCE BLOWS/FOOT |
|---|---|
| Very Soft | 0 to 2 |
| Soft | 3 to 4 |
| Firm | 5 to 8 |
| Stiff | 9 to 15 |
| Very Stiff | 16 to 30 |
| Hard | 31 to 50 |
| Very Hard | Over 50 |

## RELATIVE DENSITY OF COHESIONLESS SOILS

| RELATIVE DENSITY | STD. PENETRATION RESISTANCE BLOWS/FOOT |
|---|---|
| Very Loose | 0 to 4 |
| Loose | 5 to 10 |
| Medium Dense | 11 to 30 |
| Dense | 31 to 50 |
| Very Dense | Over 50 |

## SAMPLER TYPES

(Shown in Samples Column)

- Shelby Tube
- Split Spoon
- Rock Core
- No Recovery

## TERMS

**Standard Penetration Resistance** - The Number of Blows of 140 lb. Hammer Falling 30 in. Required to Drive 1.4 in. I.D. Split Spoon Sampler 1 Foot. As Specified in ASTM D-1586.

**REC** - Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100%.

**RQD** - Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks excluded) Divided by the Total Length of the Core Run Times 100%.

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | **HAND AUGER BORING LOG: C-1** |
|---|---|

| | | |
|---|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** | NOTES: |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **Not encountered** | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 5 inches | | |
| 1 | | **SILTY SAND (SM)** - Brown to yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 2<br>3<br>4 | | **CLAYEY SAND (SC)** - Orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | |

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.

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | HAND AUGER BORING LOG: C-2 |
|---|---|
| DATE STARTED: **6/6/23** DATE FINISHED: **6/6/23** | NOTES: |
| SAMPLING METHOD: **Hand auger** PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **Not encountered** | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 6 inches | | |
| 1 | | **SILTY SAND (SM)** - Yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 2<br>3<br>4 | | **CLAYEY SAND (SC)** - Orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | |
| | | 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.

| PROJECT: | **SLED Pee Dee Regional Office**<br>**Florence, South Carolina**<br>**23390074** | **HAND AUGER BORING LOG: C-3** |
|---|---|---|

| | | | |
|---|---|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** | NOTES: | |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** | | |
| **WATER LEVEL:** **Not encountered** | | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 4 inches | | |
| 1 | | **SILTY SAND (SM)** - Yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 2 | | **CLAYEY SAND (SC)** - Yellowish brown to orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | |
| 3 | | | | |
| 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.

| PROJECT: | SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | HAND AUGER BORING LOG: C-4 |
|---|---|---|

| | | | | |
|---|---|---|---|---|
| DATE STARTED: | **6/6/23** | DATE FINISHED: | **6/6/23** | NOTES: |
| SAMPLING METHOD: | **Hand auger** | PERFORMED BY: | **W. Kannon** | |
| **WATER LEVEL:** | **Not encountered** | | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 5 inches | | |
| 1 | | **SILTY SAND (SM)** - Yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 2 | | **CLAYEY SAND (SC)** - Orange and red, mostly fine to medium sand, some low to medium plasticity fines, moist. | | |
| 3 | | | | |
| 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.**

| PROJECT: | SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | **HAND AUGER BORING LOG: C-5** |
|---|---|---|

| | | |
|---|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** | NOTES: |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **Not encountered** | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 6 inches | | |
| | | **SILTY SAND (SM)** - Brown to yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 1 | | | | |
| 2 | | | | |
| 3 | | - - - Orange | | |
| 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.

PROJECT: **SLED Pee Dee Regional Office**
**Florence, South Carolina**
**23390074**

# HAND AUGER BORING LOG: HA-1

DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23**

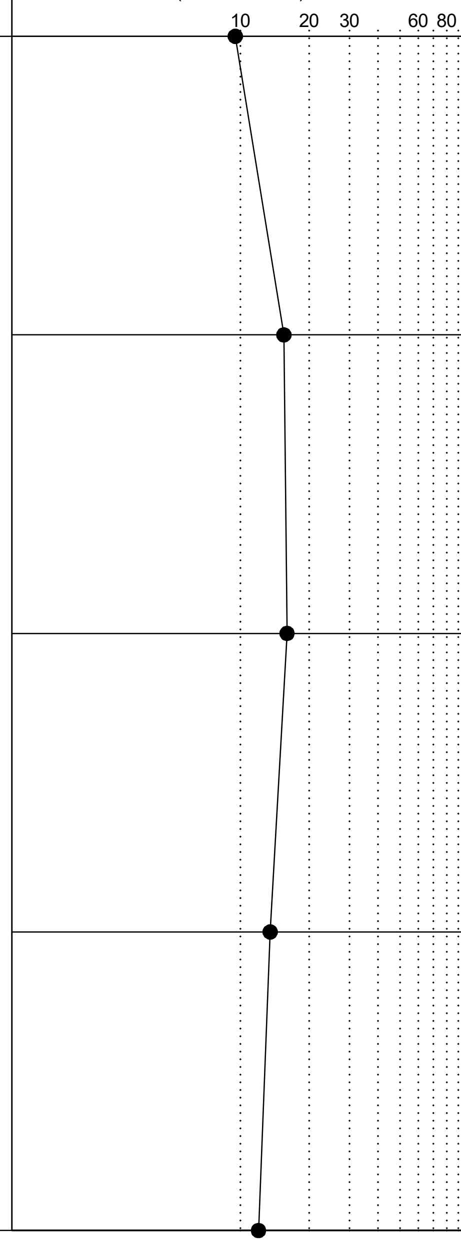
NOTES:

SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon**

**WATER LEVEL:** **Not encountered**

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DCP VALUE |
|---|---|---|---|---|---|
| 0 | | **TOPSOIL** - 4 inches | | | 10 |
| | | **SILTY SAND (SM)** - Loose, yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | |
| 1 | | - - - Medium dense | | | 16 |
| | | **CLAYEY SAND (SC)** - Medium dense, orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | |
| 2 | | | | | 16 |
| 3 | | | | | 14 |
| 4 | | Boring terminated at 4 ft | | | 12 |

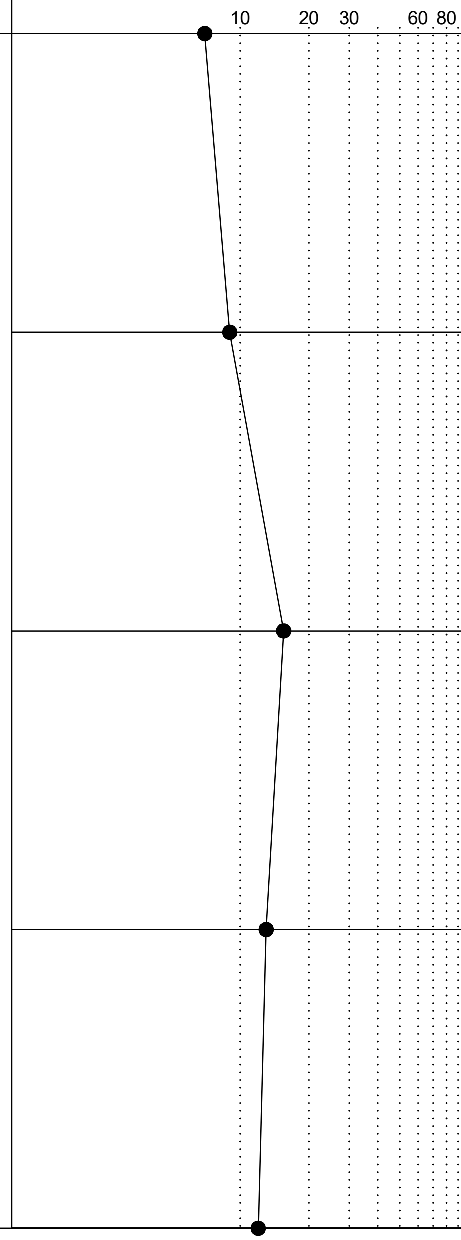
DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)




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: | SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | HAND AUGER BORING LOG: HA-2 |
|---|---|---|

| | | |
|---|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** | NOTES: |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **Not encountered** | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.) | DCP VALUE |
|---|---|---|---|---|---|---|
| | | **TOPSOIL** - 5 inches | | |  | 7 |
| | | **SILTY SAND (SM)** - Loose, yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | | |
| 1 | | | | | | 9 |
| | | **CLAYEY SAND (SC)** - Medium dense, orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | | |
| 2 | | | | | | 16 |
| 3 | | | | | | 13 |
| 4 | | Boring terminated at 4 ft | | | | 12 |



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: **SLED Pee Dee Regional Office**
**Florence, South Carolina**
**23390074**

# HAND AUGER BORING LOG: HA-3

DATE STARTED: **6/6/23** DATE FINISHED: **6/6/23**

NOTES:

SAMPLING METHOD: **Hand auger** PERFORMED BY: **W. Kannon**

**WATER LEVEL:** **Not encountered**

| Depth (feet) | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DCP VALUE (blows/1.75 in.) |
|---|---|---|---|---|
| 0 | **TOPSOIL** - 4 inches | | | 7 |
| | **SILTY SAND (SM)** - Loose, yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | |
| 1 | | | | 10 |
| | **CLAYEY SAND (SC)** - Medium dense, orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | |
| 2 | | | | 16 |
| 3 | | | | 16 |
| | --- Loose, orange and red | | | |
| 4 | Boring terminated at 4 ft | | | 10 |

DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.) — axis: 10, 20, 30, 60, 80

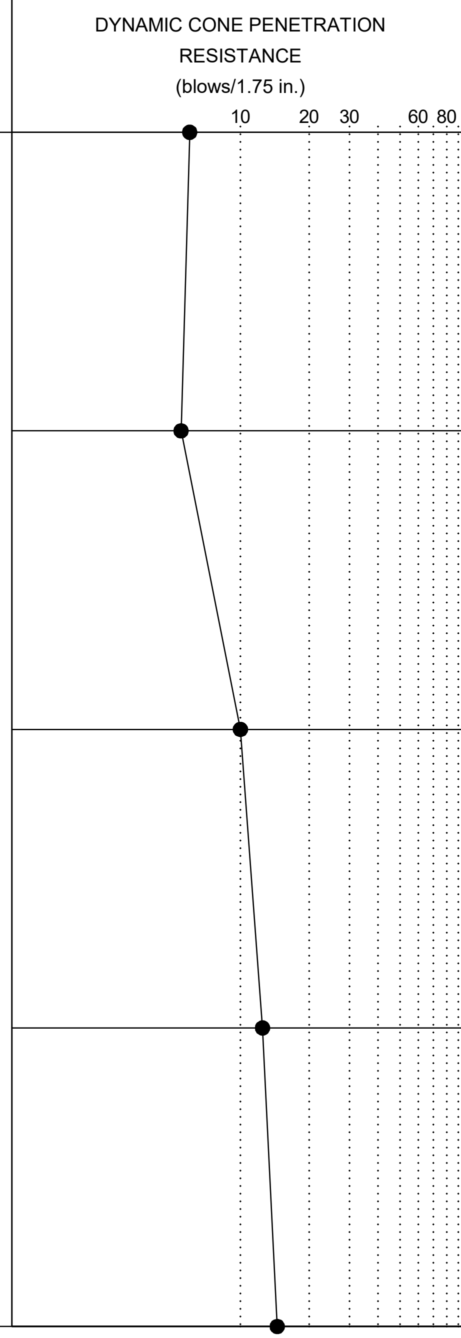
**DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.**

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | **HAND AUGER BORING LOG: HA-4** |
|---|---|

| | |
|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** |
| **WATER LEVEL:** **Not encountered** | |

NOTES:

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DCP VALUE |
|---|---|---|---|---|---|
| 0 | | **TOPSOIL** - 6 inches | | | 6 |
| 0.5 | | **SILTY SAND (SM)** - Loose, yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | |
| 1 | | | | | 6 |
| 2 | | **CLAYEY SAND (SC)** - Loose, yellowish brown, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | 10 |
| 3 | | - - - Medium dense, orange | | | 13 |
| 4 | | Boring terminated at 4 ft | | | 15 |

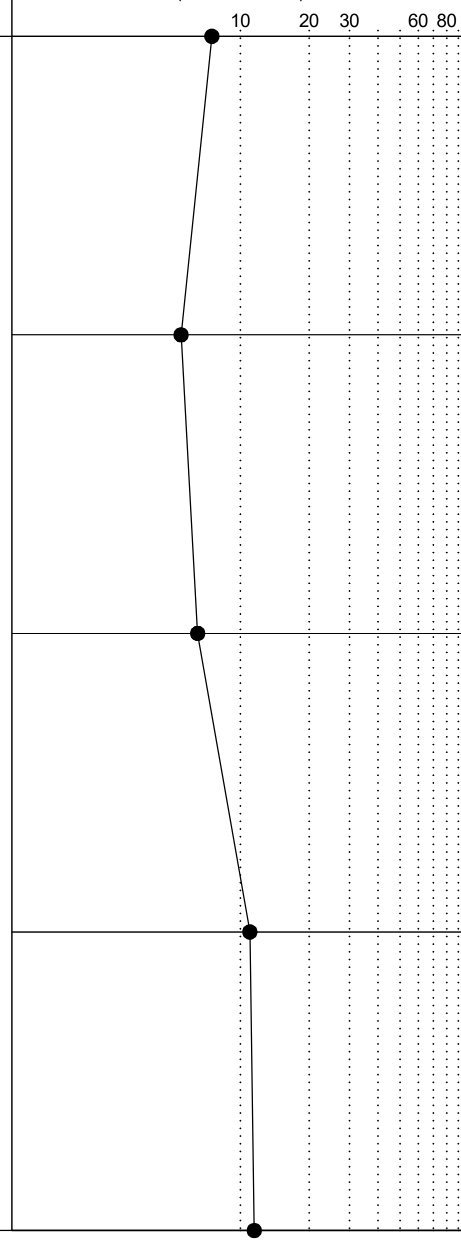



DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | **HAND AUGER BORING LOG: HA-5** |
|---|---|

| | | |
|---|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** | NOTES: |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **Not encountered** | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.) | DCP VALUE |
|---|---|---|---|---|---|---|
| 0 | | **TOPSOIL** - 5 inches | | | | 8 |
| | | **SILTY SAND (SM)** - Loose, brown to yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | | |
| 1 | | | | | | 6 |
| 2 | | | | | | 7 |
| | | **CLAYEY SAND (SC)** - Medium dense, yellowish brown to orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | | |
| 3 | | | | | | 11 |
| 4 | | Boring terminated at 4 ft | | | | 12 |



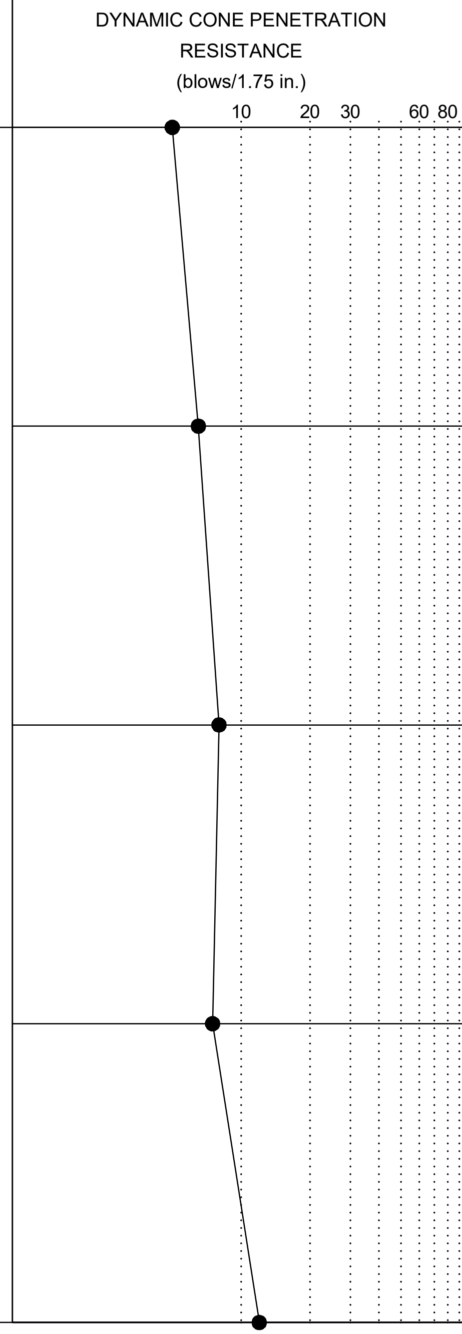

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

# HAND AUGER BORING LOG: HA-6

| | |
|---|---|
| PROJECT: | **SLED Pee Dee Regional Office**<br>**Florence, South Carolina**<br>**23390074** |
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** |
| **WATER LEVEL:** | **Not encountered** |

NOTES:

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DCP VALUE |
|---|---|---|---|---|---|
| 0 | | **TOPSOIL** - 4 inches | | | 5 |
| | | **SILTY SAND (SM)** - Loose, brown to yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | |
| 1 | | | | | 7 |
| 2 | | | | | 8 |
| 3 | | | | | 8 |
| | | **CLAYEY SAND (SC)** - Medium dense, orange, mostly fine to medium sand, some low to medium plasticity fines, moist. | | | |
| 4 | | Boring terminated at 4 ft | | | 12 |



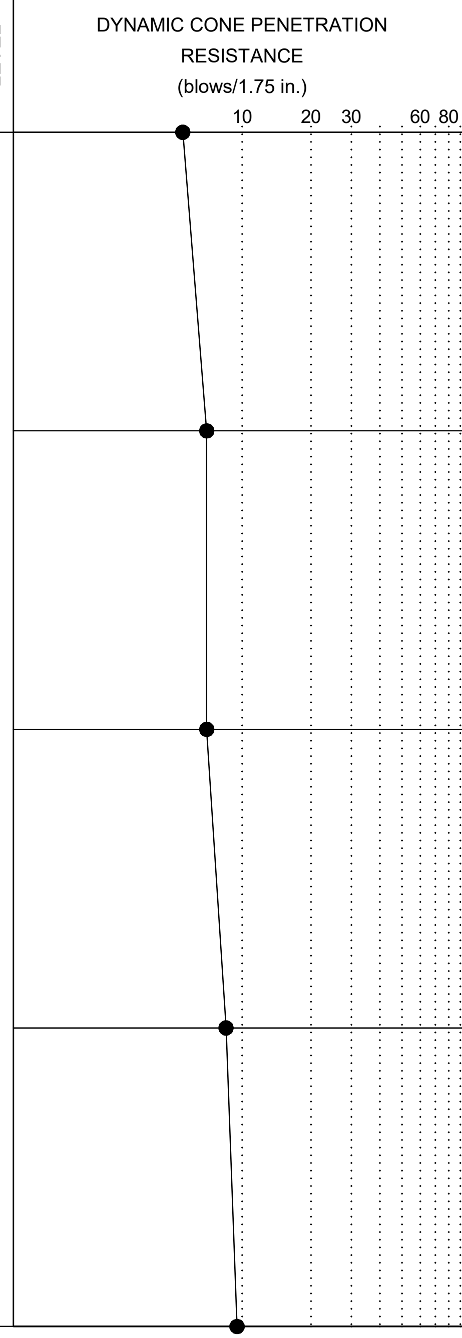

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | **HAND AUGER BORING LOG: HA-7** |
|---|---|

| | |
|---|---|
| DATE STARTED: **6/6/23** | DATE FINISHED: **6/6/23** |
| SAMPLING METHOD: **Hand auger** | PERFORMED BY: **W. Kannon** |
| **WATER LEVEL:** **Not encountered** | |

NOTES:

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL | DCP VALUE |
|---|---|---|---|---|---|
| 0 | | **TOPSOIL** - 5 inches | | | 6 |
| | | **SILTY SAND (SM)** - Loose, brown to yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | | |
| 1 | | | | | 7 |
| 2 | | | | | 7 |
| 3 | | - - - Orange | | | 9 |
| 4 | | Boring terminated at 4 ft | | | 10 |




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: | **SLED Pee Dee Regional Office**<br>**Florence, South Carolina**<br>**23390074** | **TEST PIT LOG: TP-1** |
|---|---|---|

| | | |
|---|---|---|
| DATE STARTED: **5/26/23** | DATE FINISHED: **5/26/23** | NOTES: |
| SAMPLING METHOD: **Test pit** | PERFORMED BY: **W. Kannon** | |
| **WATER LEVEL:** **TOB = 6.5 ft** | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 10 inches | | |
| 1<br>2 | | **SILTY SAND (SM)** - Yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 3<br>4 | | **POORLY GRADED SAND WITH SILT (SP-SM)** - Tan and brown, mostly fine to medium sand, few non-plastic to low plasticity fines, moist. | | |
| 5 | | **SILTY SAND (SM)** - Red, orange, and tan, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 6 | | **POORLY GRADED SAND (SP)** - White and gray, mostly fine to medium sand, trace non-plastic fines, moist to wet. | | ∇ |
| 7 | | Boring terminated at 7 ft | | |

| PROJECT: SLED Pee Dee Regional Office<br>Florence, South Carolina<br>23390074 | | TEST PIT LOG: TP-2 |
|---|---|---|
| DATE STARTED: 5/26/23 | DATE FINISHED: 5/26/23 | NOTES: |
| SAMPLING METHOD: Test pit | PERFORMED BY: W. Kannon | |
| **WATER LEVEL:** TOB = 7.5 ft | | |

| Depth (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION (feet) | WATER LEVEL |
|---|---|---|---|---|
| | | **TOPSOIL** - 8 inches | | |
| 1 | | **SILTY SAND (SM)** - Yellowish brown, mostly fine to medium sand, some non-plastic to low plasticity fines, moist. | | |
| 2 | | | | |
| 3 | | - - - Orange | | |
| 4 | | | | |
| 5 | | - - - Orange, brown, and white | | |
| 6 | | | | |
| | | - - - Orange and light gray | | |
| 7 | | **POORLY GRADED SAND WITH SILT (SP-SM)** - Tan and gray, mostly fine to medium sand, few non-plastic to low plasticity fines, moist to wet. | | ∇ |
| 8 | | Boring terminated at 8 ft | | |

# Appendix III – Laboratory Procedures and Data

# 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 B, using a hexametaphosphate solution to pre-soak the specimen for at least 2 hours, was used to prepare the sample.  The sample is then washed through the No. 200 sieve the percentage by weight of material washed 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. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 6-29-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s): | 6-5-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |
| Sample by: | W. Kannon | Sample Date(s): | 5-30-23 |
| Sampling Method: | Grab | Lab id: | 5825 |

**Method:** **A** (1%) ☐ **B** (0.1%) ☑ — Balance ID. 24996, Calibration Date: 11-16-22; Oven ID. 24457, Calibration Date: 11-15-22

| 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 | % | |
| TP-1 & TP-2 | BULK 1 | 0 to 5 ft | Vee | 88.9 | 500.1 | 460.0 | 40.1 | 10.8% | |
| HA-1 to HA-4 | BULK 2 | 0.5 to 2 ft | Lois | 91.6 | 554.6 | 524.7 | 29.9 | 6.9% | |
| C-1 | 1 | 0.5 to 2 ft | Eve | 76.5 | 468.0 | 445.2 | 22.8 | 6.2% | |
| C-4 | 1 | 0.5 to 2 ft | Ava | 91.0 | 366.4 | 349.7 | 16.7 | 6.5% | |
| HA-2 | 2 | 2 to 4 ft | Ann | 97.7 | 575.6 | 524.6 | 51.0 | 11.9% | |
| HA-5 | 3 | 3 to 4 ft | Lois | 91.6 | 500.7 | 455.1 | 45.6 | 12.5% | |

*Notes / Deviations / References*

| Jason Colvin | J. Colvin | Laboratory Manager | 6-29-23 |
|---|---|---|---|
| *Technician Name* | *Signature* | *Certification Type / No.* | *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. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 6-29-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s): | 6-6-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |
| Sample by: | W. Kannon | Sample Dates: | 5-30-23 |
| Sampling Method: | Grab | Lab id: | 5825 |

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

| 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 | % |
| TP-1 & TP-2 | BULK 1 | 0 to 5 ft | 108 | 191.7 | - | 286.0 | 270.7 | **16.2%** |
| HA-1 to HA-4 | BULK 2 | 0.5 to 2 ft | 909 | 310.5 | - | 452.8 | 425.6 | **19.1%** |
| C-1 | 1 | 0.5 to 2 ft | 706 | 294.80 | - | 439.70 | 401.30 | **26.5%** |
| C-4 | 1 | 0.5 to 2 ft | 703 | 293.70 | - | 398.00 | 376.50 | **20.6%** |
| HA-2 | 2 | 2 to 4 ft | 705 | 297.50 | - | 426.40 | 373.90 | **40.7%** |
| HA-5 | 3 | 3 to 4 ft | 23 | 143.40 | - | 260.80 | 218.40 | **36.1%** |

*Balance ID.* *24496* *Calibration Date:* *11-16-22* *#200 Sieve* *34451* *Calibration Date:* *1-10-23*

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

| Jason Colvin | | Laboratory Manager | 6-29-23 |
|---|---|---|---|
| *Technician Name* | *Signature* | *Certification Type/No.* | *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. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

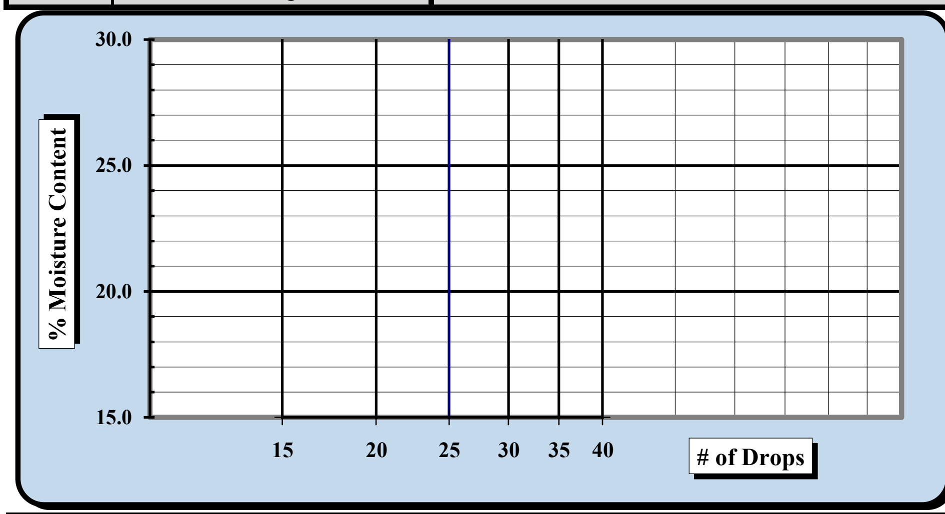
| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 7-5-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 7-3-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | C-1 | Sample #: Lab id 5825 | | Sample Date: | 5-30-23 |
| Location: | Building Pad | | | Depth: | 0.5 to 2 ft |

Sample Description: Brown Silty Sand (SM)

| *Type and Specification* | *S&ME ID #* | *Cal Date:* | *Type and Specification* | *S&ME ID #* | *Cal Date:* |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| *Pan #* | | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Tare #: | | | | | | | | | |
| A | Tare Weight | | | | | | | | NP | |
| B | Wet Soil Weight + A | | | | | | | | | |
| C | Dry Soil Weight + A | | | | | | | | | |
| D | Water Weight (B-C) | | | | | | | | | |
| E | Dry Soil Weight (C-A) | | | | | | | | #VALUE! | |
| **F** | % Moisture (D/E)*100 | | | | | | | | #VALUE! | |
| N | # OF DROPS | | | | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | | | |




One Point Liquid Limit

| N | Factor | N | Factor |
|---|---|---|---|
| 20 | 0.974 | 26 | 1.005 |
| 21 | 0.979 | 27 | 1.009 |
| 22 | 0.985 | 28 | 1.014 |
| 23 | 0.99 | 29 | 1.018 |
| 24 | 0.995 | 30 | 1.022 |
| 25 | 1.000 | | |

| | |
|---|---|
| NP, Non-Plastic | ☒ |
| Liquid Limit | -- |
| Plastic Limit | **N/P** |
| Plastic Index | -- |
| Group Symbol | **ML** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 26.5%

*Notes / Deviations / References:* Soils are non-plastic.

| Jason Colvin | 7-5-23 | | 7-5-23 |
|---|---|---|---|
| *Technician Name* | *Date* | *Technical Responsibility* | *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

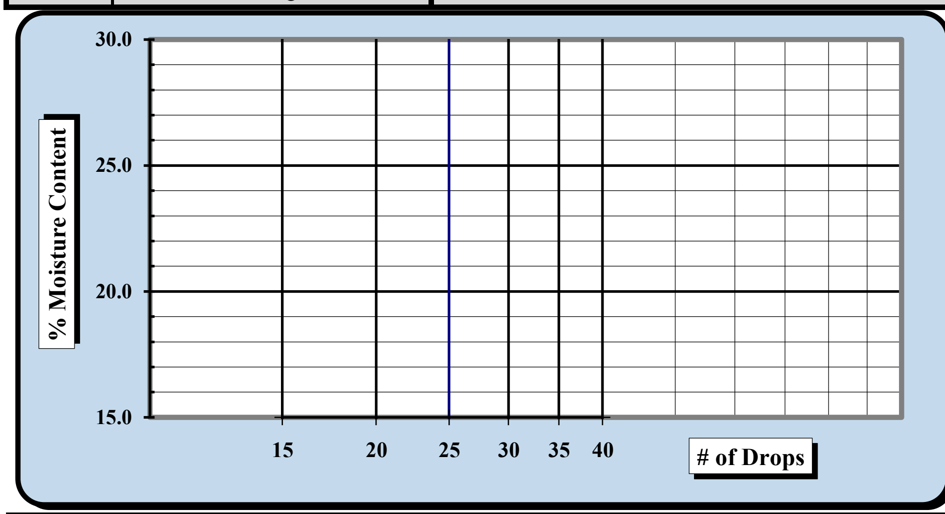
S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 7-5-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 7-3-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | C-4 | Sample #: Lab id 5825 | | Sample Date: | 5-30-23 |
| Location: | Building Pad | | | Depth: | 0.5 to 2 ft |
| Sample Description: | Brown Silty Sand (SM) | | | | |

| *Type and Specification* | *S&ME ID #* | *Cal Date:* | *Type and Specification* | *S&ME ID #* | *Cal Date:* |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| *Pan #* | | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Tare #: | | | | | | | | | |
| A | Tare Weight | | | | | | | | NP | |
| B | Wet Soil Weight + A | | | | | | | | | |
| C | Dry Soil Weight + A | | | | | | | | | |
| D | Water Weight (B-C) | | | | | | | | | |
| E | Dry Soil Weight (C-A) | | | | | | | | #VALUE! | |
| **F** | % Moisture (D/E)*100 | | | | | | | | #VALUE! | |
| N | # OF DROPS | | | | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | | | |




One Point Liquid Limit

| N | Factor | N | Factor |
|---|---|---|---|
| 20 | 0.974 | 26 | 1.005 |
| 21 | 0.979 | 27 | 1.009 |
| 22 | 0.985 | 28 | 1.014 |
| 23 | 0.99 | 29 | 1.018 |
| 24 | 0.995 | 30 | 1.022 |
| 25 | 1.000 | | |

| | |
|---|---|
| NP, Non-Plastic | ☒ |
| Liquid Limit | -- |
| Plastic Limit | **N/P** |
| Plastic Index | -- |
| Group Symbol | **ML** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 20.6%

*Notes / Deviations / References:* Soils are non-plastic.

| Jason Colvin | 7-5-23 | | 7-5-23 |
|---|---|---|---|
| *Technician Name* | *Date* | *Technical Responsibility* | *Date* |

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# LIQUID LIMIT, PLASTIC LIMIT, & PLASTIC INDEX

ASTM D 4318 ☒ AASHTO T 89 ☐ AASHTO T 90

S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

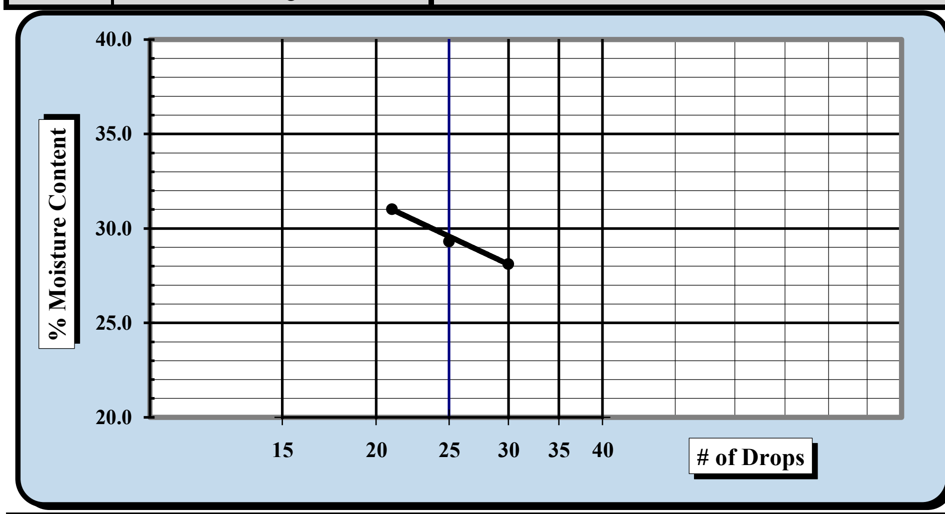
| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 7-5-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 7-3-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | |
|---|---|---|---|
| Boring #: HA-2 | Sample #: Lab id 5825 | Sample Date: | 5-30-23 |
| Location: Parking Lot | | Depth: | 2 to 4 ft |

Sample Description: Orange Clayey Sand (SC)

| *Type and Specification* | *S&ME ID #* | *Cal Date:* | *Type and Specification* | *S&ME ID #* | *Cal Date:* |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| *Pan #* | | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Tare #: | #14 | A | F | | | | J | B | |
| A | Tare Weight | 13.88 | 12.27 | 12.51 | | | | 12.33 | 12.16 | |
| B | Wet Soil Weight + A | 28.58 | 25.39 | 27.84 | | | | 19.17 | 18.51 | |
| C | Dry Soil Weight + A | 25.10 | 22.42 | 24.48 | | | | 18.26 | 17.66 | |
| D | Water Weight (B-C) | 3.48 | 2.97 | 3.36 | | | | 0.91 | 0.85 | |
| E | Dry Soil Weight (C-A) | 11.22 | 10.15 | 11.97 | | | | 5.93 | 5.50 | |
| **F** | % Moisture (D/E)*100 | 31.0% | 29.3% | 28.1% | | | | 15.3% | 15.5% | |
| N | # OF DROPS | 21 | 25 | 30 | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | **15.4%** | | |




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 | **29** |
| Plastic Limit | **15** |
| Plastic Index | **14** |
| Group Symbol | **CL** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 40.7%

*Notes / Deviations / References:*

| | | | |
|---|---|---|---|
| Jason Colvin | 7-5-23 | | 7-5-23 |
| *Technician Name* | *Date* | *Technical Responsibility* | *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

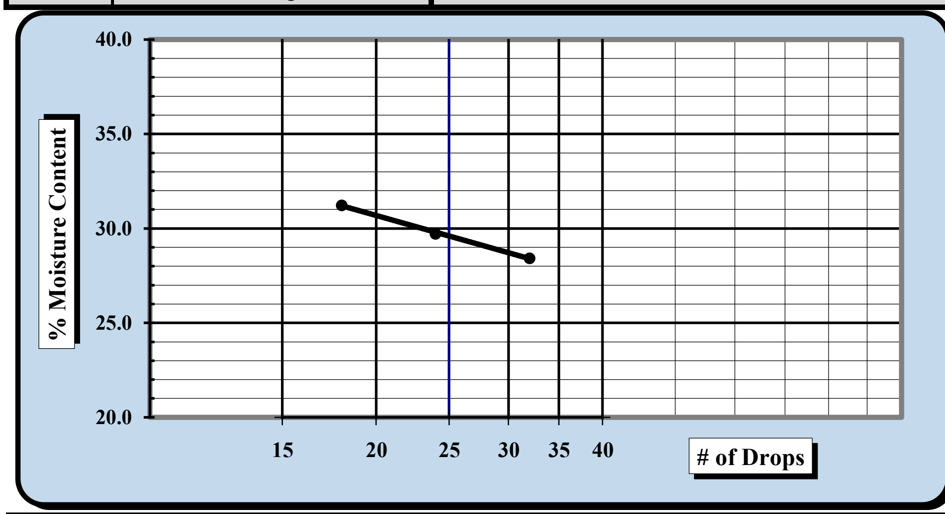
S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 7-5-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 7-3-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | HA-5 | Sample #: Lab id 5825 | | Sample Date: | 5-30-23 |
| Location: | Parking Lot | | | Depth: | 3 to 4 ft |
| Sample Description: | Orange Clayey Sand (SC) | | | | |

| Type and Specification | S&ME ID # | Cal Date: | Type and Specification | S&ME ID # | Cal Date: |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| Pan # | | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Tare #: | 50 | C | 54 | | | | 52 | G | |
| A | Tare Weight | 15.17 | 12.14 | 14.94 | | | | 15.34 | 12.34 | |
| B | Wet Soil Weight + A | 28.87 | 25.08 | 28.88 | | | | 21.98 | 18.89 | |
| C | Dry Soil Weight + A | 25.61 | 22.12 | 25.80 | | | | 21.04 | 17.96 | |
| D | Water Weight (B-C) | 3.26 | 2.96 | 3.08 | | | | 0.94 | 0.93 | |
| E | Dry Soil Weight (C-A) | 10.44 | 9.98 | 10.86 | | | | 5.70 | 5.62 | |
| **F** | % Moisture (D/E)*100 | 31.2% | 29.7% | 28.4% | | | | 16.5% | 16.5% | |
| N | # OF DROPS | 18 | 24 | 32 | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | **16.5%** | | |




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 | **30** |
| Plastic Limit | **17** |
| Plastic Index | **13** |
| Group Symbol | **CL** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 36.1%

*Notes / Deviations / References:*

| | | | |
|---|---|---|---|
| Jason Colvin | 7-5-23 | | 7-5-23 |
| *Technician Name* | *Date* | *Technical Responsibility* | *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. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

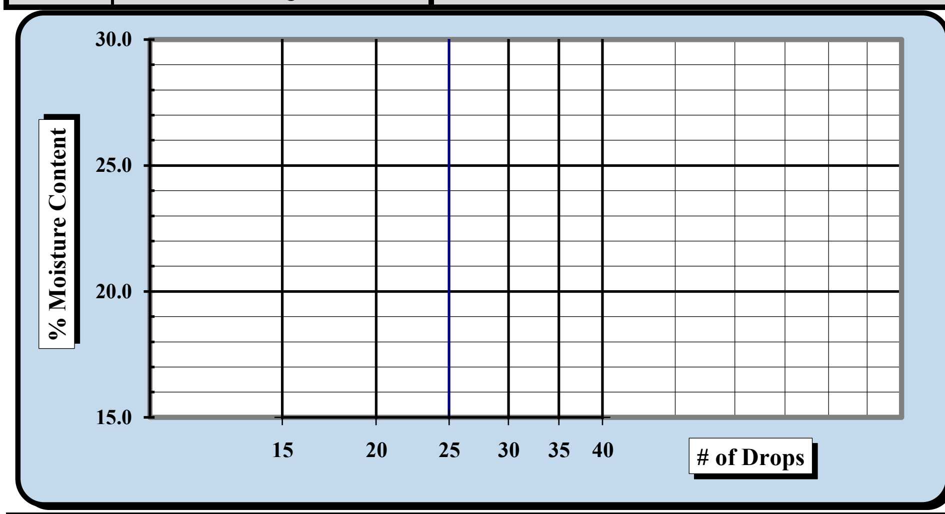
| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 6-13-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 6-6-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | TP-1 & TP-2 | Sample #: BULK 1 | | Sample Date: | 5-30-23 |
| Location: | Detention Pond | | | Depth: | 0 to 5 ft |

Sample Description: Brown Silty Sand (SM)

| *Type and Specification* | *S&ME ID #* | *Cal Date:* | *Type and Specification* | *S&ME ID #* | *Cal Date:* |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| *Pan #* | Tare #: | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| A | Tare Weight | | | | | | | | NP | |
| B | Wet Soil Weight + A | | | | | | | | | |
| C | Dry Soil Weight + A | | | | | | | | | |
| D | Water Weight (B-C) | | | | | | | | | |
| E | Dry Soil Weight (C-A) | | | | | | | | #VALUE! | |
| **F** | % Moisture (D/E)*100 | | | | | | | | #VALUE! | |
| N | # OF DROPS | | | | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | | | |




One Point Liquid Limit

| N | Factor | N | Factor |
|---|---|---|---|
| 20 | 0.974 | 26 | 1.005 |
| 21 | 0.979 | 27 | 1.009 |
| 22 | 0.985 | 28 | 1.014 |
| 23 | 0.99 | 29 | 1.018 |
| 24 | 0.995 | 30 | 1.022 |
| 25 | 1.000 | | |

| | |
|---|---|
| NP, Non-Plastic | ☒ |
| Liquid Limit | -- |
| Plastic Limit | **N/P** |
| Plastic Index | -- |
| Group Symbol | **ML** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 16.2%

*Notes / Deviations / References:* Soils are non-plastic.

| | | | |
|---|---|---|---|
| Jason Colvin | 6-13-23 | | 6-13-23 |
| *Technician Name* | *Date* | *Technical Responsibility* | *Date* |

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

# MOISTURE - DENSITY REPORT

*Quality Assurance*

S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

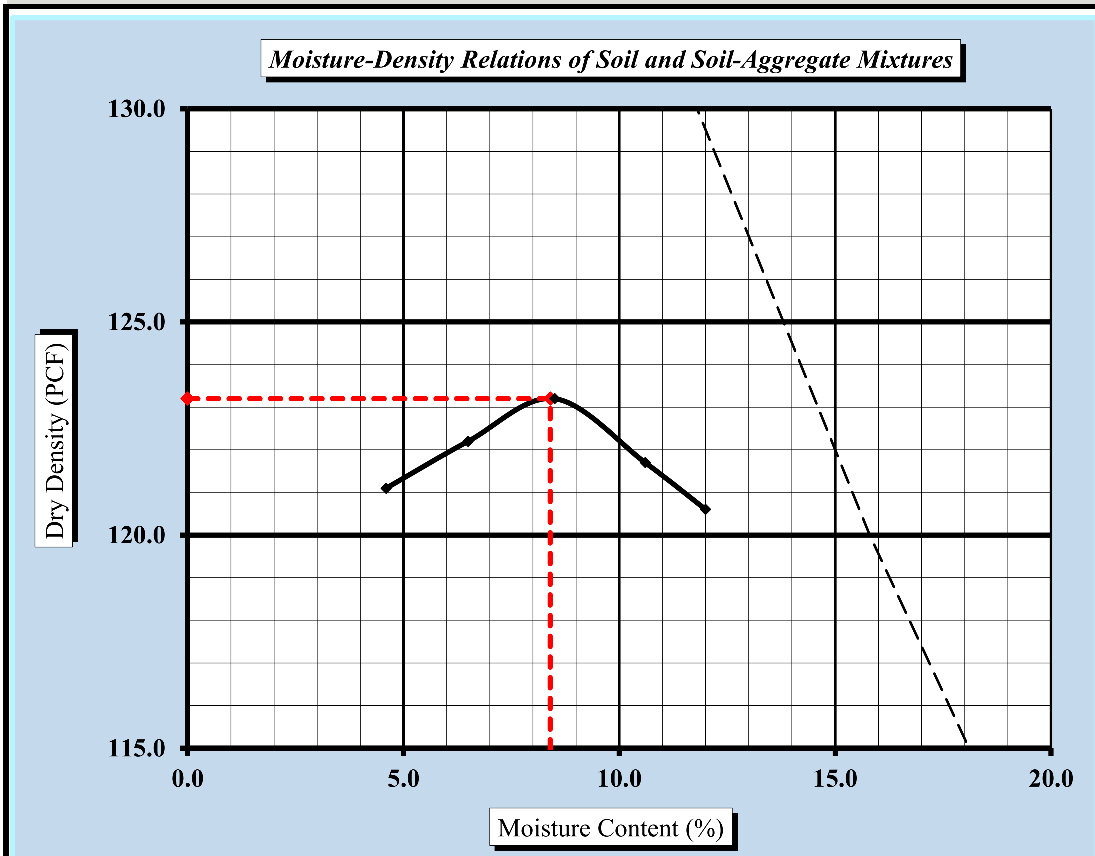
| | | | |
|---|---|---|---|
| S&ME Project #: | 23390074 | Report Date: | 6-7-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s): | 6-6-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | TP-1 & TP-2 | Sample #: | BULK 1 | Sample Date: | 5/30/2023 |
| Location: | Detention Pond | Offset: | N/A | Depth: | 0 to 5 ft |

Sample Description: Brown Silty Sand (SM)

Maximum Dry Density 123.2 PCF. Optimum Moisture Content 8.4%

***ASTM D1557 - - Method A***




| **Soil Properties** | |
|---|---|
| Natural Moisture Content | 10.8% |
| Spec. Gravity of Soil | N/A |
| Liquid Limit | N/A |
| Plastic Limit | N/A |
| Plastic Index | N/A |
| **% Passing** | |
| #4 | N/A |
| #10 | N/A |
| #30 | N/A |
| #40 | N/A |
| #60 | N/A |
| #100 | N/A |
| #200 | 16.2% |
| **Oversize Fraction** | |
| Bulk Gravity | |
| % Moisture | |
| % Oversize | #DIV/0! |
| 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:*

| Jason Colvin | | Laboratory Manager | 6-7-23 |
|---|---|---|---|
| *Technical Responsibility* | *Signature* | *Position* | *Date* |

# LIQUID LIMIT, PLASTIC LIMIT, & PLASTIC INDEX

ASTM D 4318 ☒ AASHTO T 89 ☐ AASHTO T 90

S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

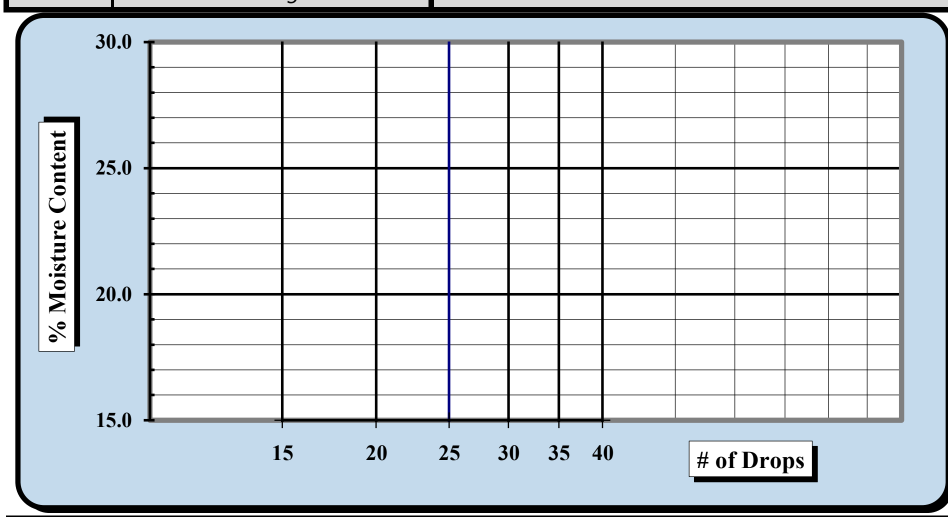
| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 6-13-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 6-6-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | |
|---|---|---|---|
| Boring #: | HA-1 to HA-4 | Sample #: BULK 2 | Sample Date: 5-30-23 |
| Location: | Roadway | | Depth: 0.5 to 2 ft |

Sample Description: Reddish brown Silty Sand (SM)

| *Type and Specification* | *S&ME ID #* | *Cal Date:* | *Type and Specification* | *S&ME ID #* | *Cal Date:* |
|---|---|---|---|---|---|
| Balance (0.01 g) | 24496 | 11/16/2022 | Grooving tool | 34452 | 9/6/2022 |
| LL Apparatus | 34453 | 9/6/2022 | | | |
| Oven | 3 | 11/15/2022 | | | |

| *Pan #* | Tare #: | Liquid Limit | | | | | | Plastic Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|
| A | Tare Weight | | | | | | | | NP | |
| B | Wet Soil Weight + A | | | | | | | | | |
| C | Dry Soil Weight + A | | | | | | | | | |
| D | Water Weight (B-C) | | | | | | | | | |
| E | Dry Soil Weight (C-A) | | | | | | | | #VALUE! | |
| **F** | % Moisture (D/E)*100 | | | | | | | | #VALUE! | |
| N | # OF DROPS | | | | | | | *Moisture Contents determined by ASTM D 2216* | | |
| LL | LL = **F** * FACTOR | | | | | | | | | |
| *Ave.* | *Average* | | | | | | | | | |




One Point Liquid Limit

| N | Factor | N | Factor |
|---|---|---|---|
| 20 | 0.974 | 26 | 1.005 |
| 21 | 0.979 | 27 | 1.009 |
| 22 | 0.985 | 28 | 1.014 |
| 23 | 0.99 | 29 | 1.018 |
| 24 | 0.995 | 30 | 1.022 |
| 25 | 1.000 | | |

| | |
|---|---|
| NP, Non-Plastic | ☒ |
| Liquid Limit | -- |
| Plastic Limit | **N/P** |
| Plastic Index | -- |
| Group Symbol | **ML** |
| Multipoint Method | ☐ |
| One-point Method | ☑ |

Wet Preparation ☑ Dry Preparation ☐ Air Dried ☑ *% passing the # 200 Sieve:* 19.1%

*Notes / Deviations / References:* Soils are non-plastic.

| | | | |
|---|---|---|---|
| Jason Colvin | 6-13-23 | | 6-13-23 |
| *Technician Name* | *Date* | *Technical Responsibility* | *Date* |

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

# MOISTURE - DENSITY REPORT

*Quality Assurance*

S&ME, Inc. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

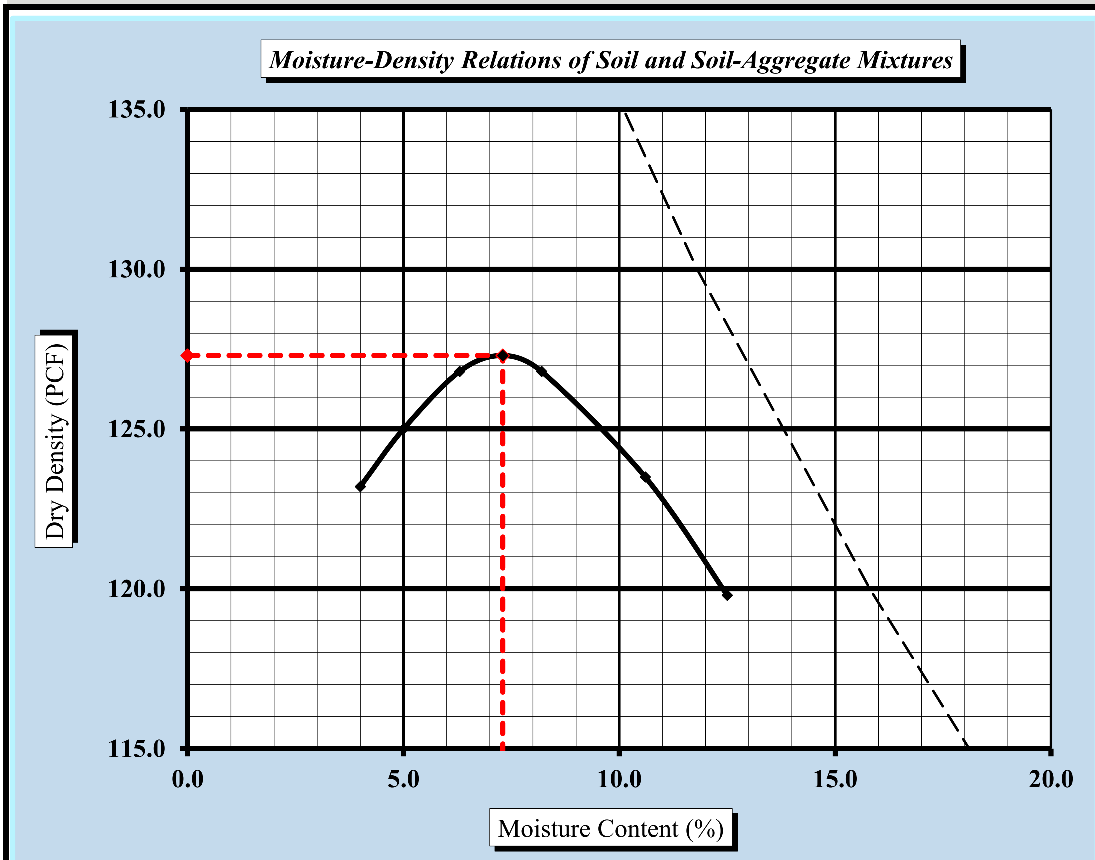
| | | | |
|---|---|---|---|
| S&ME Project #: | 23390074 | Report Date: | 6-13-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s): | 6-6-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | HA-1 to HA-4 | Sample #: | BULK 2 | Sample Date: | 5/30/2023 |
| Location: | Roadway | Offset: | N/A | Depth: | 0.5 to 2 ft |

Sample Description: Reddish brown Silty Sand (SM)

Maximum Dry Density 127.3 PCF. Optimum Moisture Content 7.3%

***ASTM D1557 - - Method A***




| ***Soil Properties*** | |
|---|---|
| *Natural Moisture Content* | 6.9 |
| *Spec. Gravity of Soil* | N/A |
| *Liquid Limit* | N/A |
| *Plastic Limit* | N/P |
| *Plastic Index* | N/A |
| ***% Passing*** | |
| *#4* | N/A |
| *#10* | N/A |
| *#30* | N/A |
| *#40* | N/A |
| *#60* | N/A |
| *#100* | N/A |
| *#200* | 19.1% |
| ***Oversize Fraction*** | |
| *Bulk Gravity* | |
| *% Moisture* | |
| *% Oversize* | #DIV/0! |
| *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:*

| Jason Colvin | | Laboratory Manager | 6-13-23 |
|---|---|---|---|
| *Technical Responsibility* | *Signature* | *Position* | *Date* |

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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. - Florence: 2327 Prosperity Way, Suite 9, Florence, SC 29501

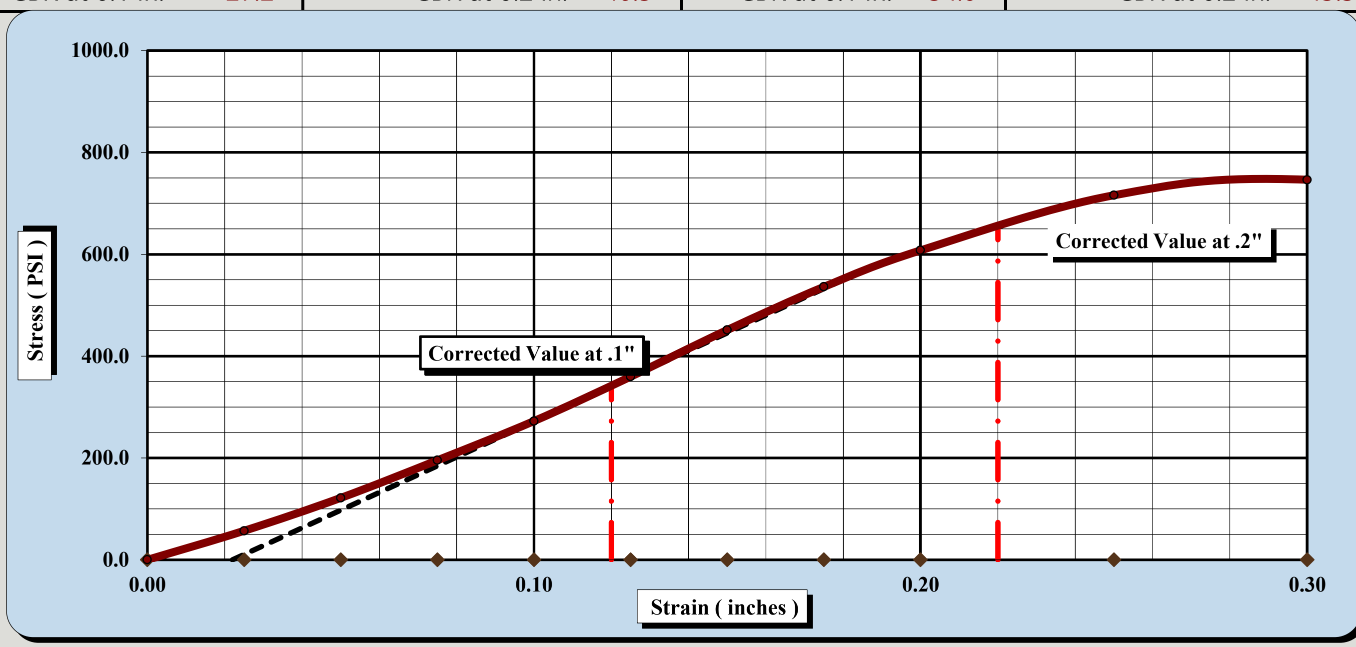
| | | | |
|---|---|---|---|
| Project #: | 23390074 | Report Date: | 6-13-23 |
| Project Name: | SLED Pee Dee Regional Office | Test Date(s) | 6-12-23 |
| Client Name: | Moseley Architects | | |
| Client Address: | 977 Morrison Ave., Suite 201, Charleston, SC 29406 | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring #: | HA-1 to HA-4 | Sample #: | BULK 2 | Sample Date: | 5-30-23 |
| Location: | Roadway | Offset: | N/A | Elevation: | 0.5 to 2 ft |

Sample Description: Reddish brown Silty Sand (SM)

*ASTM D1557* *Method A* Maximum Dry Density: 127.3 PCF Optimum Moisture Content: 7.3%
Compaction Test performed on grading complying with CBR spec. % Retained on the 3/4" sieve: 0.0%

| **Uncorrected CBR Values** | | | | **Corrected CBR Values** | | | |
|---|---|---|---|---|---|---|---|
| CBR at 0.1 in. | 27.2 | CBR at 0.2 in. | 40.5 | CBR at 0.1 in. | 34.0 | CBR at 0.2 in. | 43.3 |




CBR Sample Preparation: *Performed on the fine fraction*
*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) | 13 | | |
| Initial Dry Density (PCF) | 121.0 | Final Dry Density (PCF) | 121.0 |
| Moisture Content of the Compacted Specimen | 7.4% | Moisture Content (top 1" after soaking) | 9.8% |
| Percent Compaction | 95.1% | Percent Swell | 0.0% |

| | | | | | |
|---|---|---|---|---|---|
| Soak Time: | 96 hrs. | Surcharge Weight | 20.0 | Surcharge Wt. per sq. Ft. | 101.9 |
| Liquid Limit | N/A | Plastic Index | N/P | Apparent Relative Density | N/A |

Notes/Deviations/References:

| Jason Colvin | J. Colvin | Laboratory Manager | 6-13-23 |
|---|---|---|---|
| *Technical Responsibility* | *Signature* | *Position* | *Date* |