PART 1 - GENERAL

1.1 SOILS INVESTIGATION DATA

A. The following reports of soil borings and subsurface investigations performed by S&ME, Inc. are attached hereto. These reports are available for Contractor’s information.


END OF SECTION 003132
April 25, 2016

Coastal Carolina University
Post Office Box 261954
Conway, South Carolina 29528

Attention: Mr. Mark Avant

Reference: Brooks Stadium Expansion and Renovations
Report of Geotechnical Exploration
Conway, South Carolina
S&ME Project No. 1463-16-012

Dear Mr. Avant:

S&ME, Inc. has completed the subsurface exploration for the referenced project after receiving your written authorization to proceed March 21, 2016. Our exploration was conducted in general accordance with our Proposal No. 42-1600190, dated March 1, 2016.

The purpose of this study was to characterize the surface and subsurface soils on the proposed site, and to provide recommendations for site preparation, earthwork, foundation types, and seismic site response. This report presents the findings of our exploration along with our conclusions and recommendations.

This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations. S&ME, Inc. appreciates this opportunity to be of service to you. Please call if you have questions concerning this report or any of our services.

Sincerely,

Chelsea Jones
Staff Professional

Ronald P. Forest, Jr., P.E.
Senior Engineer
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Executive Summary

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

1. **Soil Conditions:** Our borings and soundings encountered a loose to medium dense sandy layer (Stratum I) consisting primarily of silty sands and poorly graded sands to depths ranging from about 11 to 13 feet. These soils were underlain by a layer of soft, primarily clayey soils (Stratum II), to depths of about 21 to 22 feet. This was followed by a layer of very loose to loose sands (Stratum III) which reached depths of about 31 feet. Underlying Stratum III, the Pee Dee Formation (Stratum IV) was encountered. At the top of the Pee Dee Formation a very hard, thin cap of dense to very dense clayey sands overlies stiff to very hard overconsolidated clays and silts. The cone soundings encountered refusal on the very dense cap layer at roughly 32 feet. Within the soil borings, this stratum extended to the assigned termination depths of 50 to 55 feet below the ground surface.

2. **Subsurface Water:** At the time of exploration, subsurface water was interpreted to range from depths of 4.8 to 5.75 feet within the soundings and was measured to range from 1.5 to 10.0 feet within our borings. After 24 hours, we returned to the site and re-measured stabilized water levels within the borings to range from 1.5 to 7.75 feet below the ground surface. These shallow water level measurements are likely indicative of perched water conditions, which often occur during periods of increased rainfall. Water levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence water elevations. Infiltration rates in the bottom of the existing ponds started out slow (about 0.1 to 1.0 inches per hour) and decreased to zero during the test, because the water table was essentially coincident with the bottom of the pond and the subsoils were already saturated. The infiltration rate through the unsaturated sands in the swale alongside Highway 544 was rapid, measured to be greater than 20 inches per hour. Most pond seepage is therefore expected to be lateral, through the sides of the ponds, rather than vertical through the bottom of the ponds.

3. **Liquefaction and Seismic Hazards:** Boring data indicates that this site is best described as Seismic Site Class F, due to the potential for liquefaction to occur in the subsurface loose sands during seismic shaking. Based on the apparent age and cohesion characteristics of the soils, liquefaction was determined to be a moderate risk at this site, considering the design earthquake magnitude and the anticipated ground accelerations. Up to about 5 inches of total earthquake-related settlement could occur during seismic shaking associated with the Code-level earthquake.

4. **Seismic Site Class:** A site-specific response analysis of the site was performed as part of this study, and yielded the following seismic design parameters: \( S_{DS} = 0.67g \), \( S_{D1} = 0.32g \), and Peak Ground Acceleration (PGA\(_M\)) = 0.41g. For a structure having a seismic use group classification of I, II, III, or IV, both \( S_{DS} \) and \( S_{D1} \) values obtained are consistent with Seismic Design Category D as defined in section 1613.5 of the IBC, 2012 edition.

5. **Deep Foundation System:** Shallow foundations do not appear feasible for support of this stadium structure due to the magnitude of the anticipated structural loads compared to the strength and compressibility of the soils, and because of the potential for up to 5 inches of liquefaction-related settlement during an earthquake for structures supported on shallow foundations at the surface. Further challenges may be presented by coordinating large, spread footings in the vicinity of existing stadium foundations. Driven, pre-stressed concrete piles have
been used successfully on this same project site in the past, and performance aspects are well documented, including prior load testing by S&ME, Inc. with Pile Driving Analyzer (PDA) test equipment; therefore, a deep foundation system consisting of pre-stressed concrete (PSC) driven piles is recommended. It is anticipated that with the proper equipment, PSC piles can be advanced to the desired bearing depth range, which assumes penetrating several feet into the Pee Dee Formation.

6. **Axial Pile Capacity:** 14-inch square PSC piles, advanced to a depth of about 35 feet below the existing ground surface, are estimated to have a single pile allowable axial design capacity of up to 40 tons per pile (80 tons ultimate). This pile size was used previously at this site, and PDA tests measured ultimate capacities meeting this requirement at this depth of embedment. If additional capacity is needed, it may be necessary to increase the pile size to a 16-inch square PSC pile. A 16-inch square PSC pile, advanced to a depth of about 35 feet below the existing ground surface, is estimated to have a single pile allowable axial design capacity of up to 50 tons per pile (100 tons ultimate). If it is later determined that a lower capacity may suffice, it may be feasible to reduce the pile embedment depth. If higher capacity is needed, then augered, cast-in-place reinforced concrete piles that can be drilled through the cemented lenses of the Pee Dee Formation may need to be considered. Drilled piles could also be considered if pile driving vibrations are of particular concern. Please contact us for additional recommendations if either of these situations arise.

7. **Wood Boardwalk (and other light structures):** The “wood boardwalk” shown on the site layout sketch may require less load carrying capacity. In such case, it may be appropriate to shorten the PSC piles somewhat, although it will probably still be necessary to advance the piles to the top of the Pee Dee Formation at a depth of about 31 to 32 feet in order to establish fixity at the tip due to the softness of the upper soil layers. Driven timber piles advanced to refusal on the top of the Pee Dee Formation might also be a possibility for support of the wood boardwalk, if a wooden appearance is desired. If shallow foundations are to be considered for support of the wood boardwalk (or any other light structure), then the potential for liquefaction-related settlement must be taken into account during the design process such that the structure does not collapse during seismic shaking. An applied bearing pressure limited to 2,000 psf should be used.

8. **Pavements:** For flexible pavements, we recommend the minimum pavement section for light duty asphalt (not subjected to truck traffic) consist of 2 inches of SCDOT Type C surface coarse hot mixed asphalt (HMA) over 6 inches of compacted graded aggregate base course (GABC); heavy duty asphalt (subject to truck traffic) should consist of 1.5 inches of SCDOT Type C Surface course HMA over 2 inches of SCDOT Type C Intermediate course HMA over 6 inches of compacted GABC. For heavy and light-duty rigid (concrete) pavement areas we recommend the compressive strength Portland cement concrete thickness of at least 6 inches with steel reinforcement (such as dowel baskets) at the load transfer joints, overlying a compacted graded aggregate base course thickness of 6 inches. For all pavements, the soil subgrade should be densified to at least 95 percent compaction to a depth of at least 12 inches prior to construction of the GABC layer.
1.0 Introduction

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning grading, foundation design, seismic site response, 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.

The scope of our geotechnical services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials.

A site plan showing the approximate exploration locations is included in Appendix I. The sounding logs, boring logs, infiltrometer test locations, shear wave velocity array locations, a discussion of the field exploration procedures, and the soil classification legends and symbols are included in Appendix II. The laboratory test procedures and test results are included in Appendix III, lateral pile load versus deflection curves are presented in Appendix IV, and the site-specific response analysis is presented in Appendix V.

2.0 Project and Site Description

2.1 Project Description

Project information was initially provided in an email from Mark Avant (Coastal Carolina University) to Tommy Still (S&ME) on February 18, 2016. The email included a set of site layout plans dated January 29, 2016, prepared by SMHA. We were also provided a copy of the ADC Checklist for Geotechnical Report, prepared by ADC Engineering, which outlined the requirements for the geotechnical exploration. We were given a purchase order from Coastal Carolina University to proceed on March 21, 2016.

This project includes renovation and expansion of the existing Coastal Carolina University football stadium. The provided site layout sketch shows new upper deck construction on the west stands, new lower deck construction on both the east and west stands, and new wood boardwalk construction in the south end zone. New flexible and rigid pavements are also planned for the development. Since the structure is still in the preliminary design stage, the structural building loads and exact building components are not yet available. The extent to which any of the existing facilities will be demolished is unknown at this time.

2.2 Site Description

The site is located on the Coastal Carolina University campus in Conway, South Carolina. It is located to the northeast of Highway 544 and University Boulevard. A site vicinity plan is included in Appendix I as Figure 1. There is an existing football stadium that is being renovated and expanded. The majority of the site was surfaced with concrete pavement at the time of our exploration. The remainder of the site consisted of landscaped areas, asphalt paved areas, and the existing football stadium with a synthetic turf athletic field.
3.0 Exploration Procedures

3.1 Field Exploration

Between the dates of March 10 and March 31, 2016, representatives of S&ME, Inc. visited the site on several occasions. Using the information provided, we performed the following tasks:

- We performed a site walkover, observing features of topography, ground cover, and surface soils at the project site.
- We established four cone penetration test (CPT) sounding locations, six soil penetration test (SPT) boring locations, two shear wave velocity array locations, and three double-ring infiltrometer (DRI) testing locations at the site. Locations were established using existing landmarks observed on site and the provided site layout sketch. A test location sketch is attached in Appendix I as Figure 2.
- We advanced four CPT soundings to refusal depths ranging from 31 to 34 feet, six SPT borings to depths ranging from 50 to 55 feet, and three double-ring infiltrometer test to a depth of 4 feet. We also performed two seismic shear wave velocity surface arrays.
- In conjunction with the penetration testing, split-spoon disturbed soil samples were recovered at regular depth intervals for laboratory classification and transported to our laboratory for further testing. The Standard Penetration Test (SPT) was performed at regular intervals in each boring, in general accordance with ASTM D 1586 procedures, in order to provide us with an index for estimating soil strength parameters and relative consistency of the soils encountered. In addition, one relatively undisturbed push tube (“Shelby tube”) sample was recovered from boring SPT-2 between depths of 13 and 15 feet, in general accordance with ASTM D 1587 procedures.
- The subsurface water level at each boring and infiltration test was measured with a tape at the time of exploration and after 24 hours, and the subsurface water level at each sounding location was interpreted based on pore pressure measurements at the time of exploration. Test pits and boreholes were backfilled with soil cuttings after stabilized water level measurements were obtained.

A brief description of the field tests performed during the exploration and the boring, double-ring infiltration test, and sounding logs are attached in Appendix II.

3.2 Laboratory Testing

Soil samples that we obtained were transported to our laboratory. The following laboratory tests were performed on several selected samples collected from our test locations:

- Natural moisture content measurement of four soil samples, to measure the in-situ moisture content of the soil in general accordance with ASTM D 2216, "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass”.
- Grain size analysis (without the hydrometer portion) of four soil samples, to characterize the grain size distribution of the soil in general accordance with ASTM D 422, "Standard Test Method for Particle Size Analysis of Soils”.
- Atterberg limits testing of four soil samples, to measure the soil plasticity in general accordance with ASTM D 4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils”.

April 25, 2016

The testing was performed in general accordance with ASTM or other applicable standards as summarized in the laboratory test procedures described in Appendix III. Laboratory test results are also presented in Appendix III.

4.0 Site and Surface Conditions

This section of the report describes the general site and surface conditions observed at the time of our exploration. It was beyond the scope of our exploration to survey ground elevations at our test locations.

4.1 Topography

The proposed construction area appears to be relatively level to gently sloping. Ground surface elevations were not directly surveyed, and no site specific topographic plan was made available to us; therefore, for the purpose of illustrating our subsurface cross-sectional soil profile (Figure 3), the ground surface level was set to zero.

4.2 Existing Structures & Ground Cover

At the time of our exploration, a football stadium was present on site. The majority of the site was surfaced with concrete; some areas were also surfaced with asphalt. Some grassed and landscaped areas were also present. Grasses within the landscaped areas were up to a few inches in height. Large trees were not observed at the time of our exploration. Topsoil measured approximately 4 to 6 inches in thickness in the unpaved areas. The existing football stadium contains a synthetic turf athletic field.

5.0 Subsurface Conditions

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

5.1 Regional Geology

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

Materials comprising the terraces typically consist of a strand or beach ridge deposit of clean sands at the seaward margin. Between the strand and the toe of the next inland terrace are mainly finely interlayered clays and sands termed backbarrier deposits. In most areas, the terrace deposits are sufficiently old for a
fully developed residual soil profile to have formed from the parent material, but old swamp deposits, stumps, and buried tress have in some areas been covered by the terraces and are usually not evident at the surface.

Over wide areas in Horry County, seams of poorly consolidated silts or clays occur near the base of the terrace sediments. These sediments were weathered or eroded from the underlying Pee Dee Formation and redeposited a short distance away in a low-energy environment. Under these conditions, the in-place soils often exhibit little strength and can be highly compressible.

The Pee Dee Formation consists of a thick, massive bedded, dark gray to green, calcareous clay-sand or sand-clay. Ledges of thin limestone or cemented soils are often encountered about every 6 to 8 feet in soil test borings and may range from 6 inches to 4 feet thick. In this part of the county, it is also common for the upper surface of the Pee Dee Formation to be encountered with a highly-cemented marine layer of 12 to 24 inches in thickness, which is where our deepest soundings refused at depths of about 31 to 32 feet. The Pee Dee Formation is estimated to be late Cretaceous age, about 65 million years old. This layer generally forms the bearing layer for deep foundations supporting heavy structures in the area, and is rarely penetrated fully by geotechnical borings.

### 5.2 Interpreted Subsurface Profiles

One subsurface cross-sectional profile of the site soils is attached in Appendix I as Figures 3. The cross-section orientation in plan view is shown on Figure 2. The strata indicated in the profile are characterized in the following sections. Note that the profile is not to scale. The subsurface profile was prepared for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions may vary between test locations.

Soils encountered by each of the test soundings and borings presented on the profiles were grouped into four general strata based on estimated physical properties derived from subsurface data and the recovered soil samples. The strata encountered are labeled I through IV on the soil profiles to allow their properties to be systematically described.

### 5.3 Description of Subsurface Soils

This section describes soil conditions observed at our test locations. Soil conditions may vary between test locations.

#### 5.3.1 Stratum I: Loose to Medium Dense Upper Sands

Beneath the topsoil, beginning at depths of 4 to 6 inches, a stratum of sandy soils consisting primarily of poorly-grade sand (USCS Classification “SP”) and poorly-graded sand with silt (SP-SM) was encountered to depths ranging to about 11 to 13 feet. These soils exhibited sleeve stresses ranging from 1.5 to 0.5 tsf. The tip stresses in these soils ranged from 10 to about 380 tsf, which is consistent with loose to very dense sands, but tip stresses were typically around 40 to 120 tsf, indicating typically loose to medium dense conditions. Some layers of silty sand and clayey sand were observed in the soil borings within this stratum.
These soils were typically moist to saturated, and tan to brown in color. These soils exhibited SPT N-values ranging from 4 blows per foot to 26 blows per foot (bpf), but were typically around 11 bpf, indicating a typically medium dense relative density with some very loose to loose layers.

Laboratory testing performed upon samples recovered from this stratum indicated a fines content value (percent passing the No. 200 sieve) of 9.8 percent. These soils also generally exhibited non-plastic behavior.

5.3.2 Stratum II: Very Soft Clay

Underlying Stratum I, a layer of interbedded silty sands (SM), clayey sands (SC), sandy lean clays (CL) and sandy fat clays (CH) was encountered beginning at depths ranging from 11 to 13 feet, and extending to depths ranging from about 21 to 22 feet.

These soils exhibited sleeve stresses of typically less than 0.1 to 0.25 tsf. Although the tip stresses in these soils ranged from less than 5 to 65 tsf, indicating very soft to hard consistency soils, the majority of the soils exhibited tip stresses of less than 20 tsf, which is consistent with very soft to firm clays and very loose clayey or silty sands.

These soils were typically moist to saturated, and gray in color with occasional shell fragments. These soils exhibited SPT N-values ranging from 0 to 7 blows per foot (bpf), indicating a very soft to firm consistency for the silts and clays and a very loose to loose density in the clayey and silty sands.

Laboratory testing performed upon samples recovered from this stratum indicated natural moisture contents ranging from 15.6 to 16.5 percent, and a fines content value (percent passing the No. 200 sieve) of 84.5 percent. These soils exhibited plastic behavior with a liquid limit of between 34 and 54 percent and a plastic index ranging from 18 to 37 percent. An unconfined compressive strength test performed on a sample recovered from within this stratum in boring SPT-2 at depths of 13.5 to 15 feet indicated an unconfined compressive strength for this soil of approximately 400 psf.

5.3.3 Stratum III: Very Loose to Loose Sands

Beneath Stratum II, beginning at a depth of 21 to 22 feet, a stratum of very loose to loose sandy soils was encountered to a depth of about 31 feet. These soils exhibited sleeve stresses ranging from 0.2 to 2.2 tsf. The tip stresses in these soils ranged from 20 to about 400 tsf, which classifies as loose to very dense sands, but tip stresses were most commonly around 30 tsf, indicating loose conditions. Some layers of silty sand and clayey sand were observed in the soil borings within this stratum.

These soils were typically saturated and gray in color. Shell fragments were observed at various locations within this stratum. These soils exhibited SPT N-values ranging from 3 to 12 bpf, but exhibited an average N-value of 8 bpf, indicating a typically loose relative density.

Laboratory testing was performed upon one sample recovered from this stratum indicating a natural moisture content of 13.8 percent, and a fines content value (percent passing No. 200 sieve) of 6.5 percent. Some of the clayey sand (SC) soils in this stratum exhibit moderate plasticity with a liquid limit of 38 percent and a plastic index of 24 percent.
5.3.4 Stratum IV: Pee Dee Formation

Underlying Stratum III, a stratum of overconsolidated silty sand (SM), elastic silt with sand (MH), sandy lean clay (CL), and sandy fat clay (CH) was encountered, beginning at a depth of about 31 feet, and extending to a depth of 55 feet. A thin layer or “cap” of very hard cemented sands was observed overlying the clays and silts of the Pee Dee Formation. This layer of sands and cemented soils are typical of the transition zone into the Pee Dee Formation. Soundings C-1 through C-4 reached maximum reaction force (i.e. refusal) at a depth between 31 to 35 feet on the cemented soils.

The soils underlying the cemented cap were typically moist to wet and gray in color. We were able to drill through the cemented cap with the soil boring rig, and we measured the cap to be approximately 3 to 7 inches in thickness in the soil borings (SPT-1 to SPT-6). The clays underlying the upper cemented lens exhibited SPT N-values ranging from 5 bpf to greater than 50 bpf in the cemented lenses, but was typically observed to range between 10 bpf and 20 bpf indicating a stiff to very stiff consistency.

Laboratory testing performed upon one sample recovered from this stratum indicated a natural moisture content of 11.5 percent. This sample exhibited a liquid limit of 28 percent, a plastic limit of 12 percent, and a plastic index of 16. Plasticity can be expected to vary significantly within the Pee Dee Formation.

5.4 Subsurface Water

At the time of exploration, subsurface water was interpreted to range from depths of 4.8 to 5.75 feet within the soundings and measured to range from 1.5 to 10 feet within our borings and hand auger holes. After 24 hours, we returned to the site and re-measured stabilized water levels within the borings to range from 1.5 to 7.75 feet below the ground surface. These shallow water level measurements are likely indicative of perched water conditions, which often occur during periods of increased rainfall. Water levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence water elevations.

5.5 Measured Infiltration Rates

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

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

The water level is maintained for a specific period of time, depending on the type of soil and permeability level. The volume of water needed to maintain a specified level and the time factors are recorded and converted into a specific infiltration rate.
Infiltration rates in the bottom of the existing ponds started out slow (about 0.1 to 1.0 inches per hour) and decreased to zero during the test, because the water table was essentially coincident with the bottom of the pond and the subsoils were already saturated. The infiltration rate through the unsaturated sands in the swale alongside Highway 544 was rapid. Most pond seepage is therefore expected to be lateral, through the sides of the ponds, rather than vertical through the bottom of the ponds.

The stabilized (saturated) infiltration rates measured at our test locations were approximately 0 inches per hour at locations I-1 and I-3, but at location I-2 an infiltration rate of 24.3 inches per hour was measured. A summary of the field test results is presented in Appendix II.

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


Seismic induced ground shaking at the foundation is the effect taken into account by building code seismic-resistant design provisions. Other effects, such as soil liquefaction, are not addressed explicitly in building codes but must also be considered.

6.1 Seismic Site Class

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

A site specific seismic design study was conducted for this project. Based on the site-specific seismic response analysis (SSRA), Seismic Design Category D parameters appear to be appropriate for design of this facility, for all risk categories I through IV. The following seismic design parameters are applicable to the site: $S_{DS} = 0.67g$, $S_{D1} = 0.32g$, and site class adjusted Peak Ground Acceleration ($PGA_M$) = 0.41g.

The SSRA report is attached in Appendix V and should be reviewed for additional information.

6.2 Liquefaction Potential

Liquefaction of saturated, loose, cohesionless soils occurs when they are subjected to earthquake loading that causes the pore pressures to increase and the effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results. Earthquake-induced ground surface acceleration at the site was assumed from the building code design peak ground acceleration of 0.41g.

To evaluate liquefaction potential, we performed analyses using the data obtained in the borings, considering the characteristics of the soil and water levels observed in the boring. The liquefaction analysis was performed based on the design earthquake prescribed by the 2012 edition of the International Building Code, the "simplified procedure" as presented in Youd et al. (2001), and recent

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 is between 10 and 19 with an average of 15, which indicates that the risk of liquefaction is moderate, bordering on severe, such that shallow foundation damage and damage to soil-supported grade slabs may occur in the event of the code-level earthquake.

### 6.3 Liquefaction Settlements Due to Volumetric Compression

The settlement of sands due to volumetric compression of liquefied soils depends on the induced cyclic stresses from the earthquake, the vertical effective stress at the depth of the layer being examined, and the equivalent SPT values. A rigorous evaluation of surface settlement due to earthquake motion was beyond our scope of work, but settlements were in general terms evaluated by comparing SPT values within the liquefied zone to available empirical data.

By multiplying the average estimated volumetric strain by the thickness of the liquefied zone, cumulative total settlements under the design earthquake are estimated to be on the order of about 5 inches. Settlements occurring due to volumetric strains within the liquefied soils are likely to be somewhat variable across the site; differential settlements could be as much as three-quarters of the total settlement, which in this case would be greater than 3 inches.

### 7.0 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 structure location is changed, or if conditions are encountered during construction that differ from those encountered, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

Our geotechnical exploration indicates that the site is adaptable for the proposed construction.

Shallow foundations do not appear feasible for support of the stadium structure due to the magnitude of the anticipated structural loads compared to the strength and compressibility of the soils, and because of the potential for up to 5 inches of liquefaction-related settlement during an earthquake for structures supported on shallow foundations at the surface. Further challenges may be presented by coordinating
large, spread footings in the vicinity of existing stadium foundations. Driven, pre-stressed concrete piles have been used successfully on this same project site in the past, and performance aspects are well documented, including prior load testing by S&ME, Inc. with Pile Driving Analyzer (PDA) test equipment; therefore, a deep foundation system consisting of pre-stressed concrete (PSC) driven piles is recommended. It is anticipated that with the proper equipment, PSC piles can be advanced to the desired bearing depth range, which assumes penetrating several feet into the Pee Dee Formation.

The "wood boardwalk" shown on the site layout sketch may require less load carrying capacity. In such case, it may be appropriate to shorten the PSC piles somewhat, although it will probably still be necessary to advance the piles to the top of the Pee Dee Formation at a depth of about 31 to 32 feet in order to establish fixity at the tip due to the softness of the upper soil layers. Driven timber piles advanced to refusal on the top of the Pee Dee Formation might also be a possibility for support of the wood boardwalk, if a wooden appearance is desired and if the boardwalk is lightly loaded. We have not provided timber pile capacity recommendations in this report, but if more information is desired about this alternative, please contact us. If shallow foundations are to be considered for support of the wood boardwalk (or any other light structure), then the potential for liquefaction-related settlement must be taken into account during the design process such that the structure does not collapse during seismic shaking.

The following sections present our geotechnical recommendations regarding site preparation and structural support.

7.1 Demolition of Existing Structures

The extent to which existing parts of the stadium will be demolished as part of this project is unknown at this time. However, the following general recommendations are provided for the demolition of utilities and foundations of the existing structures in areas where this occurs:

1. Remove or plug existing utilities that are to be permanently abandoned. If not removed or plugged, pipes may serve as conduits for subsurface erosion resulting in formation of voids below foundations or grade slabs. Where existing utilities are left in place and plugged in the structural footprint, it may be necessary to undercut poorly compacted backfill to provide adequate support for foundations or slabs.

2. Reroute existing utilities that will remain in use around the proposed new structural footprint.

3. The foundation types used during construction of the existing football stadium is not fully known by S&ME. We observed first-hand the construction of the Athletic Training Facility in the north end zone, and it is known to be founded on PSC piles; however, there may also be structures on this project site that are supported on shallow foundations.

   A. Completely exhume and remove all grade slabs and shallow spread footing foundations associated with the existing structure from within the demolition footprint. Backfill the holes left by removal of the footings in accordance with Section 7.3 of this report.

   B. For structures supported on deep foundations, surface grade slabs should be removed and the piles should be separated from their pile caps; the pile caps should be removed in their entirety. Soils should be excavated from around the piles to a depth of at least 1 foot below
the future building's foundation (footing or pile cap) bearing elevation. At that depth, the old piles may be cut off. The upper portion of the piles that have been cut off should be removed and disposed of; the remainder of the piles should be left in the ground. The old piles should not be extracted from the ground because this could create voids into which loose sands could collapse, resulting in an increase in the settlement or subsidence potential of the future foundations and slabs that are built over the voids.

C. Areas excavated to cut off piles or to remove buried footings or pile caps may be backfilled with clean, washed, coarse gravel such as SCDOT No. 57 or No. 67 stone, if located below the water table, in which case backfilling with layers of soil may be impractical.

As with any demolition project, there may be unexpected or unforeseen conditions encountered in the subsurface. In such cases, the Geotechnical Engineer shall be consulted.

7.2 Surface Preparation

Site preparation over most of the site will include demolition of the existing structures, pavements, and foundations, and stripping of surface vegetation, removal of trees, organic-laden topsoil, rootmat, roots, stumps, and similar organic materials. The following recommendations are provided regarding site preparation and earthwork:

1. Strip surface vegetation, organic material, or debris where encountered, and dispose of outside the future structure footprint. Organic soils containing more than about 5 percent organics should be removed from the proposed construction areas.

2. After stripping, the surface soils in these areas should be thoroughly densified at the surface under the observation of the Geotechnical Engineer (S&ME), or his representative, prior to placement of new fill. Densification of these soils can likely be accomplished with a heavy vibratory roller. This is because of the loose condition of the near-surface sands. Moisture conditioning by the addition of water or drying of soils should be expected to be required prior to densification. The surface should be densified to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) to a depth of at least 12 inches.

3. Care should be exercised to maintain the soils within plus or minus 3 percent of the optimum moisture content at the time of compaction, as determined by modified Proctor moisture-density relationship testing (ASTM D 1557). Field density tests should be performed after densification is performed to measure compaction levels. Areas not meeting compaction requirements should be reworked and retested until proper compaction is achieved.

4. It may be necessary to overexcavate and replace some soft soils that will not sufficiently densify using this procedure. Where overexcavation of soils is required, the excavation should be extended to a lateral distance of at least 5 feet beyond the limits of the structure. Backfill of the excavation should be performed using structural fill as outlined in Section 7.3 of this report.

5. Following densification, the densified surfaces of the construction area should be proofrolled by the contractor under the observation of the Geotechnical Engineer by making repeated passes with a fully-loaded tandem-axle dump truck or equipment with similar ground pressure. The proofrolling should be conducted only during dry weather to avoid deteriorating the surface. Areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to fill placement or slab construction, as determined by the Geotechnical Engineer.
6. Following implementation of the above measures, a layer of washed open-graded gravel such as SCDOT No. 57 stone may be necessary to provide additional drainage and support for pile driving equipment, or timber rafts may need to be used, depending upon the surface conditions at the time of construction.

7.3 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 fill soils used to build up the pad for the structure and pavements meet the following minimum requirements: plasticity index of 10 percent or less; clay/silt fines content of not greater than 25 percent.

   A. This may include soils from the following ASTM soil classifications: SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, and/or SC. Not all soils in these categories will comply with the plasticity and fines content requirements; therefore, the contractor should sample each fill material that they propose to use and submit it to the Geotechnical Engineer for determination of its suitability, and measurement of the maximum dry density, optimum moisture content, and natural moisture content.

   B. The results of our laboratory tests indicate that soils excavated on-site from within the upper 10 feet are likely to be suitable for re-use as structural fill; although soils borrowed from below the water table are expected to be saturated.

2. Where fill soil is required, structural fill should be compacted throughout to at least 95 percent of the soil’s modified Proctor maximum dry density, “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft$^3$ (2,700 kN-m/m$^3$))” (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 10 inches in thickness prior to compaction when using large compaction equipment.

   C. Loose fill lifts should be limited to no more than 6 inches in thickness when using small or portable compaction equipment such as walk-behind remote-controlled compactors or vibrating plate tamps.

   D. Structural fill should extend at least 5 feet beyond building foundations or from the edge of pavements before either sloping or being allowed to exhibit a lower level of compaction.

3. 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 150 cubic feet of fill placed in confined areas such as isolated undercuts and in trenches, with a minimum of 1 test per lift.
7.4 Driven Pile Foundations

As previously mentioned, shallow foundations are not recommended for the support of the football stadium structure. Therefore, it is our recommendation that this proposed structure be supported on deep foundations.

We have initially considered and evaluated two potential options for deep foundation support of the stadium, which include a driven 14-inch square PSC piles, and driven 16-inch square PSC piles. Our recommendations regarding the PSC pile alternatives are outlined below:

7.4.1 Axial Pile Capacity

The proposed structure may be supported on 14-inch square driven PSC piles bearing in the Pee Dee Formation (Stratum IV) at a depth of approximately 35 feet below the existing ground surface, or in other words, embedded approximately 3 feet into the Pee Dee Formation. Based upon our calculations, each pile is estimated to support an axial working load of up to 40 tons, subject to verification by pile load testing using Pile Driving Analyzer (PDA) equipment. This working load capacity assumes a factor of safety of 2.0 against the estimated ultimate capacity. The soil coefficients used in our axial capacity analyses were developed using published correlations relating soil skin friction and end bearing unit capacities to SPT N-value.

If additional axial capacity per pile is needed, the pile size may be increased to 16-inch square driven PSC piles also bearing at a depth of 35 feet below the existing ground surface or about 3 feet into the Pee Dee Formation. This pile is estimated to support an axial working load of up to 50 tons, subject to verification by pile load testing using Pile Driving Analyzer (PDA) equipment. This working load capacity assumes a factor of safety of 2.0 against the estimated ultimate capacity. The soil coefficients used in our axial capacity analyses were developed using published correlations relating soil skin friction and end bearing unit capacities to SPT N-value.

If it is later determined that somewhat less than 40 tons of axial working capacity per pile would suffice, then it may be feasible to reduce the embedment depth of the piles, although it will probably still be necessary to advance the piles at least to the top of the Pee Dee Formation at a depth of about 32 feet in order to establish fixity at the tip due to the softness of the upper soil layers. If higher pile capacity (>50 tons/pile) is needed, then augered, cast-in-place reinforced concrete piles that can be drilled through the cemented lenses of the Pee Dee Formation may need to be considered, in lieu of a driven pile. The nearby CCU baseball complex, for example, was supported on 16-inch diameter, 70-ton auger-cast piles bearing at 42 to 45 feet. Drilled piles could also be considered if pile driving vibrations are of concern.

Pile capacities during seismic shaking were also estimated, modeling the liquefiable soil zone and considering downdrag of the overlying unliquefied layers. Soils in the upper five feet of the soil profile were considered not to contribute to pile resistance or downdrag. Also, soils within one pile diameter above the pile tip are generally considered not to contribute to side friction capacity, and were ignored in computation of ultimate pile capacity.

The minimum recommended center-to-center pile spacing is 3 pile diameters. For center-to-center pile spacings of at least 3 pile diameters, no reduction factor needs to be applied to the individual pile capacity to account for group effects due to the type of the bearing soils.
7.4.2 Lateral Pile Deflection Analysis

Our lateral pile analyses were performed using the computer program LPILE\(^1\) and a generalized soil profile based on our subsurface exploration. This program performs a beam-column analysis of single piles subjected to given lateral and axial loading, and assuming a non-linear soil response.

Individual 14-inch square PSC piles and 16-inch square PSC piles embedded at 35 feet were analyzed. In the analysis, a series of horizontal loads was applied to the top of the pile. The single-pile analyses modeled fixed-head conditions, and the top of the pile was assumed to be flush with the ground surface.

The lateral deflection was estimated for each load, as well as the resulting bending moment at the pile head and along the pile. Plots of pile deflection, bending moment, and shear force as a function of depth along the pile are attached in Appendix IV. Table 7-1 below summarizes our lateral pile analysis for the structure, with the graphical results shown in Appendix IV.

<table>
<thead>
<tr>
<th>Case</th>
<th>Lateral Deflection (in)</th>
<th>Lateral Load (kips)</th>
<th>Maximum Bending Moment (in-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14” x 14” Static Case</td>
<td>¼</td>
<td>26.4</td>
<td>1,160</td>
</tr>
<tr>
<td></td>
<td>½</td>
<td>37.6</td>
<td>1,560</td>
</tr>
<tr>
<td></td>
<td>¾</td>
<td>44.5</td>
<td>1,700</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>47.6</td>
<td>1,720</td>
</tr>
<tr>
<td>14” x 14” Seismic Case</td>
<td>¼</td>
<td>21.1</td>
<td>900</td>
</tr>
<tr>
<td></td>
<td>½</td>
<td>30.3</td>
<td>1,300</td>
</tr>
<tr>
<td></td>
<td>¾</td>
<td>36.3</td>
<td>1,520</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>38.3</td>
<td>1,600</td>
</tr>
<tr>
<td>16” x 16” Static Case</td>
<td>¼</td>
<td>34.3</td>
<td>1,600</td>
</tr>
<tr>
<td></td>
<td>½</td>
<td>48.7</td>
<td>2,200</td>
</tr>
<tr>
<td></td>
<td>¾</td>
<td>58.4</td>
<td>2,500</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>62.6</td>
<td>2,500</td>
</tr>
<tr>
<td>16” x 16” Seismic Case</td>
<td>¼</td>
<td>25.0</td>
<td>1,160</td>
</tr>
<tr>
<td></td>
<td>½</td>
<td>35.3</td>
<td>1,700</td>
</tr>
<tr>
<td></td>
<td>¾</td>
<td>43.1</td>
<td>2,040</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>45.6</td>
<td>2,200</td>
</tr>
</tbody>
</table>

Please note that the PSC piles were modeled as “reinforced”; since we were not provided with pile reinforcing schedules we modeled the pile based on a typical reinforcing schedule for a 14-inch/16-inch PSC. Once the actual PSC pile reinforcing is designed, we should be contacted to re-evaluate the lateral deflection for an agreed-upon additional fee.

7.4.3 Settlement of PSC Piles

Pile settlement consists of two components: axial compression of the piles themselves (termed “elastic shortening”), and consolidation settlement of the piles due to deformation within the soil column. The side friction of a single PSC pile is typically fully-mobilized at vertical displacements of 0.1 to 1.0 percent of the pile diameter in cohesionless soil, taking into account the elastic shortening of the pile itself (Reese & O’Neill, 1988). For a single 14-inch or 16-inch square pile, this would typically equate to less than ¼ inch of vertical displacement associated with elastic shortening.

Settlement of pile groups is typically greater than for individual piles. Group settlements may be estimated using the equivalent footing method, assuming the enclosed area by the group to act similar to a spread footing that bears at an elevation equal to two-thirds the pile length below the surface. To use this method requires that the size of the pile group, number and spacing of piles, and axial load on the group be known. However, since the “equivalent footing” would bear near the top of the Pee Dee Formation, settlements of the pile groups would be likely small (on the order of ½ inch or less), and settlement will likely be dominated by elastic shortening of individual piles.

We should be contacted to estimate the total group settlements as well as check the differential settlement between adjacent dissimilar groups (if applicable) once the actual pile loads and the configurations of the pile groups have been finally determined.

7.4.4 Pile Hammer Selection and Driving Criteria

Compatibility of the pile driving equipment, the soil conditions and the pile type being driven are all essential elements achieving the required penetration and capacity. Criteria for terminating driving should take into account the hammer used, pile weight, allowable pile stresses, and required capacity.

1. The pile driving hammer used under these conditions should typically be rated by the manufacturer to have between 25,000 and 40,000 ft-lbs. of energy with a minimum hammer weight of 5,000 lbs. Pile hammer type, hammer base, and cushion material selected by the contractor should be provided to the Geotechnical Engineer for review prior to driving. Performance of the driving system may vary considerably due to the type and model of hammer used, type and condition of the hammer cushion, and the condition and state of maintenance of the particular hammer in use. Gravity “drop” hammers and vibratory hammers may not be used. Diesel or air-powered (pneumatic) impact hammers are recommended.

A. During construction of the Athletic Training Facility in the north end zone, which was supported on 14-inch square PSC piles, the contractor used an ICE-75 hydraulic hammer, having a ram weight of 7,500 lbs, a maximum stroke of 4 feet, and a manufacturer’s rated maximum energy of 30,000 ft-lbs. This hammer was operated at a stroke of 2 feet until the Pee Dee Formation was encountered, and was then operated at the full 4 ft stroke until the desired bearing depth and pile capacity were achieved. A 6-inch plywood cushion was used.
2. For soil bearing piles, the final rate of penetration should be estimated for the selected hammer type and energy using the latest version of the GRLWEAP computer code by Goble Rausche Likens and Associates, or equivalent. Input parameters for use in the analysis will be based on our evaluation of the subsurface profile and the PDA and CAPWAP data obtained during the test pile installation.

3. Leads are required on the hammer and should be fixed at the top and adjustable on the bottom. Piles should be installed as plumb as possible, or at the designated batter, with the pile, hammer and leads in alignment to prevent impact bowing.

4. Pile capacities should be verified by at least four (4) pile driving analyzer (PDA) tests performed by S&ME, Inc. at representative locations prior to casting of the production piles:

   A. At least four index (test) piles of minimum 45 ft length should be driven in representative locations chosen by the Geotechnical Engineer prior to production pile installation, under representative conditions. A representative of the Geotechnical Engineer should witness the index pile driving.

   B. Index pile driving equipment should be the same as to be used during production. The contractor should be prepared to advance one of the index piles with a reinforced steel tip to estimate capacity in case such tips are needed on the production piles in order to penetrate subsurface dense sand lenses observed in the borings. Following installation and testing, index piles may be cut-off, withdrawn, or used in production pile caps as desired, unless damaged by driving or if the required capacities are not achieved.

   C. The index piles should be monitored during initial driving using a Pile Driving Analyzer (PDA) Model GCXS or equivalent. Since previous projects in this area have indicated that considerable "freeze" or "set-up" of the piles occurs after initial driving, a re-strike test should be performed on each index pile several days after the initial driving. The re-strikes should also be monitored with the PDA equipment.

   D. At least one of the PDA tests should be analyzed using CAPWAP or similar computer code to verify the damping and quake parameters assumed in the PDA tests and to more closely estimate the available pile capacity.

7.4.5 Production Pile Driving

1. All production piles should be installed using the same equipment, and to approximately the same depth and hammer blow count criteria as the applicable test piles. Installation should start at the center of each pile group and work toward the outer perimeter, as applicable. Do not use jetting to advance the piles.

2. Production pile installation should be observed by an experienced inspector or engineering technician working under the guidance and supervision of the geotechnical engineer (S&ME, Inc.). Piles should be driven to the recommended design depth. However, if pre-drilling to deeper depths than 5 feet occurs, the geotechnical engineer should be allowed to analyze the effect upon the pile capacity and make corrections to the pile capacity if necessary. Deeper pre-drilling may necessitate an increase in the pile embedding depth to achieve comparable capacity values. Also, piles should be installed as plumb as possible (or at the designated batter), with the pile, hammer, and leads in alignment.

3. In the event that the piles encounter refusal to further advancement above the desired bearing depth, extra piles may need to be driven to make up for the capacity loss resulting from the early refusal pile(s). Contact the geotechnical engineer in the event of any such "early refusal".
4. Records of all piles driven should be prepared on an appropriate driving log by the geotechnical engineer’s inspector. This should include the following as applicable:

- size, length, head cut-off elevation, toe elevation, location;
- sequence of driving;
- number of blows per ft. or per inch;
- pre-augering, diameter and depth;
- driving start time, and end time;
- cushion arrangements;
- movement of adjacent piles.

7.5  Lateral Earth Pressures

The equivalent fluid pressures given below should be used to design the pile caps. The values given in the following table assume that the pile caps are excavated into the native soils and are cast neat against the sides of the excavations. These values assume contact with soils generally classified as sand (SP) or silty sand (SM) according to the Unified Soil Classification System.

<table>
<thead>
<tr>
<th>Support Condition</th>
<th>Angle of Internal Friction (φ')</th>
<th>Moist Unit Weight (γ)</th>
<th>DRAINED CONDITION</th>
<th>Dynamic Earth Pressure Coefficient PGA=0.41g (K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Condition (K_a)</td>
<td>35°</td>
<td>120 pcf</td>
<td>0.27</td>
<td>0.40</td>
</tr>
<tr>
<td>At-Rest (K_o)</td>
<td>35°</td>
<td>120 pcf</td>
<td>0.43</td>
<td>0.60</td>
</tr>
<tr>
<td>Passive (K_p)</td>
<td>35°</td>
<td>120 pcf</td>
<td>3.7</td>
<td>3.3</td>
</tr>
</tbody>
</table>

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 0.4 may be used in computation of the lateral sliding resistance.

C. Lateral earth pressure coefficients may vary if compacted backfill is used around the pile caps.

D. These earth pressure coefficients assume cohesionless soils. The actual soils are expected to have a small amount of cohesion, which is ignored for the purposes of this recommendation.

If the pile caps are overexcavated and formed, and then backfill is placed and compacted around the pile caps in accordance with the compaction recommendations given in the next section of this report, then the earth pressures may vary from those given in the above table. If this is the case, please contact us for additional information.
7.6 Shallow Foundations

The following recommendations are provided for the design and construction of shallow foundations at this site. These recommendations should only be used for lightly-loaded, single-story minor structures, and only if the structural design can accommodate the predicted magnitudes of static and earthquake-related settlements without structural collapse. Shallow foundations are not recommended for support of the stadium facility.

1. A net available bearing pressure of up to 2,000 psf may be used for design of individual spread footings and wall footings that are extended to bear within residual coastal plain deposits or within structural fill compacted as recommended in Section 7.3 of this report.

2. It should be anticipated that where footings bear directly on fill, the previously placed fill soils exposed in the bottom of the footings may need to be tamped to increase their density prior to the placement of foundation concrete. 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. It is not uncommon for these sands to require moistening prior to densification in order to improve the available bearing conditions.

3. Even if smaller dimensions are theoretically allowable from a bearing pressure consideration, the minimum wall footing width should be at least 18 inches, and the minimum column footing width should be 30 inches, to avoid punching shear. Footings should be embedded to a minimum depth of at least 12 inches, or the depth specified on the drawings, whichever is greater.

4. Have the geotechnical engineer (S&ME) observe each cleaned footing excavation prior to concrete placement to measure that the required level of soil compaction and bearing capacity is present at the foundation bearing surface. Also, have the geotechnical engineer observe any undercut areas in footings prior to backfilling, in order to confirm that poor soils have been removed and that the exposed subgrade is suitable for support of footings or backfill.

5. For the purposes of settlement estimation, we assumed the structures will be constructed near existing grade elevations. Seismic settlements are not included in these estimates, which are in addition to the static settlement potential.

A. Considering a 3 kip per linear foot wall load and a 2,000 psf spread footing bearing pressure, the estimated post construction static settlement of a typical wall strip footing 18 inches in width will likely be on the order of 1 inch or less.

B. Considering a 32 kip column load and a 2,000 psf spread footing bearing pressure, the estimated post-construction static settlement of a typical column footing measuring 4 ft by 4 ft in width will likely be on the order of 1 inch or less.

C. Differential static settlements between individual walls and columns are typically on the order of 50 percent of the maximum total settlement value under static loading, or in this case, ½ inch or less.

7.7 Grade Slab Support and Construction

The following recommendations are given for the support and construction of soil-supported grade slabs. It is important to note that soil-supported grade slabs should be considered sacrificial in the event of an earthquake unless they are pile-supported, due to the liquefaction settlement potential.
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. A modulus of subgrade reaction (k) of 150 lbs/in$^3$ may be used for reinforcing design.

2. Structural design should incorporate installation of a vapor barrier prior to placing concrete for grade slab systems to limit moisture-infiltration into finished spaces, where appropriate.

3. Below the floor slab place a layer of at least 4 inches of compacted graded aggregate base course (GABC), to provide confinement between the sandy soil subgrade and the slab concrete in finished spaces. GABC materials should meet the material and gradation requirements as defined by Section 305 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007). Compact the GABC to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557).

4. Have the geotechnical engineer observe a proofroll of all slab subgrades prior to concrete placement. Softened soils may need to be undercut or stabilized before concrete placement.

7.8 Pavement Recommendations

We assume that new pavement subgrades will be constructed atop compacted structural fill soils compacted to at least 95 percent of the modified Proctor maximum dry density. We have performed our evaluations assuming that a CBR value of at least 10 percent will be available from subgrade soils compacted to 95 percent. If soils exhibiting a CBR value of less than 10 percent at 95 percent compaction are to be used on this project, these recommendations may require revision.

Traffic volumes for the proposed development were not provided to us in preparation for our exploration and pavement section analysis; therefore, we have performed our calculations based on typical pavement section thicknesses. The recommended pavement section components are provided in Table 7-3 below.

For flexible pavements, the pavement thickness computations were made using the AASHTO method, assuming an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 95 percent. Assuming that only SCDOT approved source materials will be used in flexible pavement section construction, we used a structural layer coefficient of 0.44 for the HMA layers and a coefficient of 0.18 for the graded aggregate base course (GABC). Rigid pavement design assumes an initial serviceability of 4.5 and a terminal serviceability index of 2.5, and a reliability factor of 90 percent. Assuming that appropriately designed load transfer devices (dowels) will be used at the joints in the rigid pavement, we used an average load transfer coefficient of 3.2. We also assumed a minimum 28-day design compressive strength of at least 4,000 psi for the PCC. A sub-base drainage factor of 1.0 was assigned, based upon the assumption that the sub-base soils will consist of granular soils.

If reinforced joint design with appropriate load transfer devices (such as steel dowels) is not provided, then the rigid pavement section thickness design would need to be reconsidered using a higher load transfer coefficient, which is likely to result in an increase in the pavement section thickness to maintain a similar ESAL capacity.
Table 7-3: Recommended Minimum Pavement Sections\(^{(a)}\)

<table>
<thead>
<tr>
<th>Pavement Area</th>
<th>Theoretical Allowable Traffic Load (ESALs)</th>
<th>HMA Surface Course Type C (inches)</th>
<th>HMA Intermediate Course Type C (inches)</th>
<th>4,000 psi Doweled Joint Concrete Pavement (inches)</th>
<th>Compacted SCDOT Graded Aggregate Base Course [GABC] (inches)</th>
<th>Compacted Subgrade [95% Compaction ASTM D1557] (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light Duty Flexible (Asphalt)</td>
<td>36,400</td>
<td>2.0</td>
<td>---</td>
<td>---</td>
<td>6.0</td>
<td>12</td>
</tr>
<tr>
<td>Heavy Duty Flexible (Asphalt)</td>
<td>311,000</td>
<td>1.5</td>
<td>2.0</td>
<td>---</td>
<td>6.0</td>
<td>12</td>
</tr>
<tr>
<td>Heavy Duty Rigid (Concrete)</td>
<td>430,000</td>
<td>---</td>
<td>---</td>
<td>6.0</td>
<td>6.0</td>
<td>12</td>
</tr>
</tbody>
</table>

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

7.8.1 Permanent Underdrains

We recommended that in order to provide permanent stabilization for new pavements, a system of underdrains should be designed for the pavement area subgrades. This is due to the presence of shallow perched water conditions.

1. The site civil engineer should be consulted regarding the type and location of the underdrains. Our experience is that two types of underdrain systems are commonly used in this locality, depending upon the traffic application and the preferences of the civil engineer. One commonly used system is a gravel-filled, fabric-wrapped trench containing an embedded perforated plastic HDPE pipe. Another type of system that we see used is an edge drain product such as AdvantEdge by ADS, Inc. This is a fabric-wrapped, perforated HDPE slot style drain. Some engineers have used a combination of these two systems. Typically, the underdrains are tied into the storm water system to maintain positive gravity flow.

2. If there are landscaped islands or flower beds, do not fill the islands with clayey or silty (impermeable) spoils that may impede the movement of water into the underdrains.

7.8.2 General Pavement Construction Recommendations

The following recommendations are provided regarding pavement construction:

1. Fill placed in pavement areas should be compacted as recommended previously in this report. Prior to pavement section installation, all exposed pavement area subgrades should be methodically proofrolled at final subgrade elevation under the observation of S&ME, Inc., and any identified unstable areas should be repaired as directed.
2. Pavement underdrainage and/or ditches should be designed and constructed, as previously discussed. The pavement underdrainage should be designed by the civil engineer to assist in long-term drainage.

3. The stone base course underlying pavements should consist of a graded aggregate base course (GABC) as specified by the SCDOT 2007 Standard Specifications for Highway Construction, Section 305. Proposed materials for use should be provided by a SCDOT-approved source.

4. As stated in the SCDOT Section 305 specification, we recommend that all new base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC T-140). Base courses should not exhibit pumping or rutting under equipment traffic. Heavy compaction equipment is likely to be required in order to achieve the required base course compaction, and the moisture content of the material will likely need to be maintained very near the optimum moisture content in order to facilitate proper compaction. S&ME, Inc. should be contacted to perform field density and thickness testing of the base course prior to paving.

5. Experience indicates that a thin surface overlay of asphalt pavement may be required in about 7 to 10 years due to normal wear and weathering of the surface. Such wear is typically visible in several forms of pavement distress, such as aggregate exposure and polishing, aggregate stripping, asphalt bleeding, and various types of cracking. There are means to methodically estimate the remaining pavement life based on a systematic statistical evaluation of pavement distress density and mode of failure. We recommend the pavement be evaluated in about 7 years to assess the pavement condition and remaining life.

6. Construct the HMA surface course in accordance with the specifications of Section 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition). Construct HMA intermediate courses in accordance with the specifications of Section 402 of this same specification.

7. It is very important for this project that the asphaltic concrete be properly compacted, as specified in Section 401.4 of the SCDOT specification. Asphaltic concrete that is insufficiently compacted will show wear much more rapidly than if it were properly compacted.

8. 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.

9. For rigid pavements, we recommend air-entrained ASTM C 94 joint reinforced Portland cement concrete that will achieve a minimum compressive strength of at least 4,000 psi at 28 days after placement, as determined by ASTM C 39. We also recommend that the pavement concrete be constructed in a manner which at least meets the minimum standards recommended by the American Concrete Institute (ACI).

10. We recommend that at least 1 set of 5 cylinder specimens be cast by S&ME per every 50 cubic yards of concrete placed or at least once per placement event in order to measure achievement of the design compressive strength. We also recommend that S&ME be present on site to observe concrete placement.

8.0 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.
The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our boring and sounding locations may not become evident until construction. If variations appear evident, then we should be provided a reasonable opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.
Appendices
Appendix I

Site Vicinity Plan

Test Location Sketch

Interpreted Subsurface Profile

A-A'
Not To Scale

April, 2016

CDJ

Google Earth

Brooks Stadium Expansion
Conway, South Carolina

SITE VICINITY PLAN

Approximate Site Location
LEGEND
- SPT Boring Locations
- CPT Sounding Locations
- Double Ring Infiltrometer
- MASW Array

Test Location Sketch
Brooks Stadium Expansion and Renovation
Conway, South Carolina

Not To Scale
Google Earth
March, 2015
CDJ

S&ME
1463-16-012
Appendix II

Summary of Exploration Procedures

CPT Soil Classification Legend

CPT Sounding Logs

SPT Soil Classification Chart

SPT Boring Logs

Infiltration Rate of Soils in Field

Double-ring Infiltrometer Location Hand Auger Boring Logs
Summary of Exploration Procedures

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

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

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

Reconnaissance of the Project Area

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

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

Boring and Sampling

Electronic Cone Penetrometer (CPT) Soundings

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

A penetrometer with a conical tip having a 60 degree apex angle and a cone base area of 10 cm² 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 qc. A friction sleeve is present on the penetrometer immediately behind the cone tip. The force exerted on the sleeve was measured electronically at a minimum of every 50 mm
penetration and divided by the surface area of the sleeve to obtain the friction sleeve resistance value $f_s$. A pore pressure element mounted immediately behind the cone tip was used to measure the pore pressure induced during advancement of the cone into the soil.

**CPT Soil Stratification**

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

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

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

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

**Hand Auger Borings**

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 and later transported to the laboratory. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. At selected intervals, the augers were withdrawn and soil consistency measured with a dynamic cone penetrometer. The conical point of the penetrometer was first seated 1-3/4 inches to penetrate any loose cuttings in the boring, then driven two additional 1-3/4 inch increments by a 15 pound hammer falling 20 inches. The number of hammer blows required to achieve this penetration was recorded. When properly evaluated by qualified professional staff, the blow count is an index to the soil strength and ability to support foundations.
Soil Test Boring with Mud-Rotary Drilling

Soil sampling and penetration testing were performed in general accordance with ASTM D1586, “Standard Test Method for Penetration Test and Split Barrel Sampling of Soils.” Mud-rotary drilling methods were used to advance the holes. Soil samples were obtained with a standard 1.4 inch I. D., two-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Double Ring Infiltrometer Tests

The infiltration rate of water through the near surface soil was measured using a double ring infiltrometer test as described by ASTM D 5093, “Standard Test Method for Field Measurement of Infiltration Rate Using a Double Ring Infiltrometer.” The infiltrometer consists of an open outer ring and a sealed inner ring which are embedded and sealed in trenches excavated in the soil. Both rings are filled with water such that the inner ring is submerged. The rate of flow or infiltration into the soil was measured by connecting a flexible bag containing a known weight of water to the inner ring. As water infiltrates into the ground, water flows from the bag into the inner ring. The weight loss from the flexible bag, converted to a volume, represents the amount of water that infiltrated into the soil from the inner ring over a set time interval.

Multi-Channel analysis of Surface Waves (MASW)

Shear wave velocities were measured at the site using MASW (Multi-Channel analysis of Surface Waves) and MAM (Microtremor Array Method) with non-linear array geometry, combining the dispersion curves from both tests prior to the inversion process. Performing both MASW and MAM provides the greater depth of penetration associated with Microtremor analysis (low frequency surface waves) without sacrificing resolution at shallower depths from MASW (higher frequency surface waves). In addition, our experience indicates using a combination of both methods to develop a shear wave velocity profile is more accurate than using Refraction Microtremor (ReMi™) exclusively, particularly when the ReMi™ array geometry is linear.

The MASW and MAM testing was conducted using the 16-channel Geometrics ES3000 seismograph and 4.5 Hz vertical geophones. For the MASW testing, the geophones were spaced in a linear geometry at intervals of 7 feet and surface waves generated by a 16-pound sledge hammer striking a metal plate. MAM testing was conducted using an “L-shaped” array geometry with geophone spacing of 30 feet. Because the source locations of the microtremors are not known, the 2-dimensional array geometry is used for the MAM. The analysis was conducted using the OYO Corporation’s SeisImager/SW software (Pickwin v. 3.14 and WaveEq).

A combination of active and passive sources was used to develop the wave frequencies required to obtain velocities to a depth of 100 feet. The results of the active and passive sources were combined to produce a single shear wave velocity profile. Based on section 1613.3.2 of 2012
International Building Code, and Section 20.3.3 of Chapter 20 of ASCE 7, the calculated weighted average shear wave velocities, $v_s$, using the developed Shear Wave Velocity Profiles were determined.

**Water Level Measurement**

Subsurface water levels in the boreholes were measured during the onsite exploration and after a period of about 24 hours by measuring depths from the existing grade to the current water level using a tape.

**Backfilling of Borings**

Once subsurface water levels were obtained, boring spoils were backfilled into the open bore holes. Bore holes were backfilled to the existing ground surface.
CPT Soil Classification Legend

<table>
<thead>
<tr>
<th>Zone</th>
<th>Description</th>
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<tbody>
<tr>
<td>1</td>
<td>Sensitive, Fine Grained</td>
</tr>
<tr>
<td>2</td>
<td>Organic Soils - Peats</td>
</tr>
<tr>
<td>3</td>
<td>Clays - Clay to Silty Clay</td>
</tr>
<tr>
<td>4</td>
<td>Silt Mixtures - Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>5</td>
<td>Sand Mixtures - Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>6</td>
<td>Sands - Clean Sand to Silty Sand</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly Sand to Sand</td>
</tr>
<tr>
<td>8</td>
<td>Very Stiff Clay to Clayey Sand*</td>
</tr>
<tr>
<td>9</td>
<td>Very Stiff, Fine Grained*</td>
</tr>
</tbody>
</table>

(*) Heavily Overconsolidated or Cemented

Robertson's Soil Behavior Type (SBT), 1990

<table>
<thead>
<tr>
<th>Group #</th>
<th>Description</th>
<th>Ic</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive, fine grained</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Organic soils - peats</td>
<td>3.60</td>
<td>N/A</td>
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<tr>
<td>3</td>
<td>Clays - silty clay to clay</td>
<td>2.95</td>
<td>2.60</td>
<td>3.60</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixtures - clayey silt to silty clay</td>
<td>2.60</td>
<td>2.05</td>
<td>2.60</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures - silty sand to sandy silt</td>
<td>1.31</td>
<td>1.31</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Sands - clean sand to silty sand</td>
<td>N/A</td>
<td></td>
<td>1.31</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly sand to dense sand</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Very stiff sand to clayey sand (High OCR or cemented)</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Very stiff, fine grained (High OCR or cemented)</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>SANDS</th>
<th>Cone Tip Stress, qt (tsf)</th>
<th>Relative Density</th>
<th>SILTS and CLAYS</th>
<th>Cone Tip Stress, qt (tsf)</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Less than 20</td>
<td>Very Loose</td>
<td>Less than 5</td>
<td>Very Soft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 - 40</td>
<td>Loose</td>
<td>5 - 15</td>
<td>Soft to Firm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40 - 120</td>
<td>Medium Dense</td>
<td>15 - 30</td>
<td>Stiff</td>
<td></td>
<td></td>
</tr>
<tr>
<td>120 - 200</td>
<td>Dense</td>
<td>30 - 60</td>
<td>Very Stiff</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Greater than 200</td>
<td>Very Dense</td>
<td>Greater than 60</td>
<td>Hard</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Cone Penetration Test**

**CPT-2**

**Date:** Mar. 21, 2016  
**Estimated Water Depth:** 5.42 ft  
**Rig/Operator:** Cory Robison

**Total Depth:** 32.0 ft  
**Termination Criteria:** Maximum Reaction Force  
**Cone Size:** 1.44

---

**Depth (ft)**  
<table>
<thead>
<tr>
<th>0</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>320</td>
<td>480</td>
<td>640</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Tip Resistance**  
- $q_t$ (tsf)

**Sleeve Friction**  
- $f_s$ (tsf)

**Pore Pressure**  
- $u_s$ (tsf)

**Friction Ratio**  
- $R_f$ (%)  
- $N_{ae}$

**Equivalent N<sub>60</sub>**  
- SBT<sub>fr</sub>  
- MAI = 5

**Gravels:**  
- Gravels to Sand
- Sands to Silty Sand

**Sands:**  
- Clean Sand to Silty Sand
- Sands-Mixtures to Sandy Silt

**Clays:**  
- Clay Silt to Silty Clay

**Silt Mixtures:**  
- Sandy Silt to Sandy Silt

**Silt:**  
- Sandy Silt to Sandy Silt

**Estimated Water Depth:** 5.42 ft

---

Electronic Filename: Stadium CPT-02.cpt
Cone Penetration Test

Date: Mar. 21, 2016
Estimated Water Depth: 5.75 ft
Rig/Operator: Cory Robison

Total Depth: 31.3 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.44

Depth (ft)

Tip Resistance $q_t$ (tsf)

Sleeve Friction $f_s$ (tsf)

Pore Pressure $u_2$ (tsf)

Friction Ratio $R_f$ (%)

Equivalent $N_{eq}$

$SBT_{fr}$ MAI = 5

Estimated Water Depth: 5.75 ft

S&ME Project No: 1463-16-012
Brooks Stadium Expansion
Conway, South Carolina

Electronic Filename: Stadium CPT-03.cpt
# Soil Classification Chart

**Note:** Dual symbols are used to indicate borderline soil classifications.

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Symbols</th>
<th>Typical Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coarse Grained Soils</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel and Gravelly Soils</td>
<td>Clean Gravels</td>
<td>GW - Well-graded gravels, gravel - sand mixtures, little or no fines</td>
</tr>
<tr>
<td></td>
<td>(Little or No Fines)</td>
<td>GP - Poorly-graded gravels, gravel - sand mixtures, little or no fines</td>
</tr>
<tr>
<td></td>
<td>Gravels with Fines</td>
<td>GM - Silty gravels, gravel - sand - silt mixtures</td>
</tr>
<tr>
<td></td>
<td>(Appreciable Amount of Fines)</td>
<td>GC - Clayey gravels, gravel - sand - clay mixtures</td>
</tr>
<tr>
<td>More than 50% of Material is Larger than No. 200 Sieve Size</td>
<td>Clean Sands</td>
<td>SW - Well-graded sands, gravelly sands, little or no fines</td>
</tr>
<tr>
<td></td>
<td>(Little or No Fines)</td>
<td>SP - Poorly-graded sands, gravelly sand, little or no fines</td>
</tr>
<tr>
<td></td>
<td>Sand with Finem</td>
<td>SM - Silty sands, sand - silt mixtures</td>
</tr>
<tr>
<td></td>
<td>(Appreciable Amount of Fines)</td>
<td>SC - Clayey sands, sand - clay mixtures</td>
</tr>
<tr>
<td><strong>Fine Grained Soils</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silts and Clays</td>
<td>Liquid Limit Less than 50</td>
<td>ML - Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
</tr>
<tr>
<td></td>
<td>Liquid Limit Greater than 50</td>
<td>CL - Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td></td>
<td>Organic Silts and Organic Silty Clays of Low Plasticity</td>
<td>OL</td>
</tr>
<tr>
<td>More than 50% of Material is Smaller than No. 200 Sieve Size</td>
<td>Silts and Clays</td>
<td>MH - Inorganic silts, micaceous or diatomaceous fine sand or silty soils</td>
</tr>
<tr>
<td></td>
<td>Liquid Limit Greater than 50</td>
<td>CH - Inorganic clays of high plasticity</td>
</tr>
<tr>
<td></td>
<td>Organic Clays of Medium to High Plasticity, Organic Silts</td>
<td>OH</td>
</tr>
</tbody>
</table>

**Highly Organic Soils**

| | | PT - Peat, humus, swamp soils with high organic contents |
**Project:** Brooks Stadium Expansion
Conway, South Carolina
S&M Project No. 1463-16-012

**Boring Log SPT-1**

**Date Drilled:** 3/21/16

**Elevation:**

**Notes:** Elevation Unknown

**Drill Rig:** Diedrich D-25

**Boring Depth:** 50.0 ft

**Driller:** Mike

**Water Level:** 10’ ATD, 7.5’ 24 hr

**Hammer Type:** Automatic

**Logged By:** GHG

**Sampling Method:** Split spoon

**Drilling Method:** Mud Rotary

**Depth (feet) Graphic Log**

**Material Description**

**Topsoil -** Approximately 4 inches.

POORLY GRADED SAND WITH SILT (SP-SM)
- Mostly fine to medium sand, few low plasticity fines, brown, tan, moist, medium dense.
  - - - - - - wet, very loose.

POORLY GRADED SAND (SP)
- Mostly fine to medium sand, brown, wet to saturated, medium dense.

SANDY LEAN CLAY (CL)
- Mostly low to medium plasticity fines, some fine sand, dark gray, moist, very soft.

POORLY GRADED SAND WITH SILT (SP-SM)
- Mostly fine to medium sand, few low plasticity fines, shell fragments, gray, saturated, very loose.

CLAYEY SAND (SC)
- Mostly fine to medium sand, some medium plasticity fines, gray, saturated, loose.

POORLY GRADED SAND (SP)
- Mostly fine to medium sand, shell fragments, gray, saturated, medium dense.

**Notes:**

1. This log is only a portion of a report prepared for the named project and must only be used together with that report.
2. Boring, sampling and penetration test data in general accordance with ASTM D-1586.
3. Stratification and groundwater depths are not exact.
4. Water level is at time of exploration and will vary.
**Brooks Stadium Expansion**
Conway, South Carolina
S&ME Project No. 1463-16-012

<table>
<thead>
<tr>
<th>BORING LOG  SPT-1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DATE DRILLED:</strong> 3/21/16</td>
</tr>
<tr>
<td><strong>DRILL RIG:</strong> Diedrich D-25</td>
</tr>
<tr>
<td><strong>DRILLER:</strong> Mike</td>
</tr>
<tr>
<td><strong>HAMMER TYPE:</strong> Automatic</td>
</tr>
<tr>
<td><strong>SAMPLING METHOD:</strong> Split spoon</td>
</tr>
<tr>
<td><strong>DRILLING METHOD:</strong> Mud Rotary</td>
</tr>
</tbody>
</table>

**MATERIAL DESCRIPTION**

- - - - Very hard lens at 31 feet (Transition into Pee Dee Formation)

SANDY FAT CLAY (CH) - PEE Dee FORMATION - Mostly medium to high plasticity fines, some fine sand, dark gray to green, moist to wet, stiff.

- - - - stiff

Boring terminated at 50 ft Target Depth

**NOTES:**

1. **THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.**
2. **BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.**
3. **STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.**
4. **WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.**
NOTES:

1. THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.

2. BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.

3. STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.

4. WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.
SANDY LEAN CLAY (CL) - PEE Dee FORMATION - Mostly low to medium plasticity fines, some fine sand, dark gray, very hard, wet.

- - - - stiff.

- - - saturated, very hard.

- - - cemented.

Boring terminated at 50 ft
Target Depth
### S&ME BORING LOG  1463-16-012 LOGS.GPJ  S&ME 2011_03_09.GDT  4/25/16

### BORING LOG  SPT-3

**PROJECT:** Brooks Stadium Expansion  
Conway, South Carolina  
S&ME Project No. 1463-16-012

**DATE DRILLED:** 3/22/16  
**ELEVATION:**  
**NOTES:** Elevation Unknown

**DRILL RIG:** Diedrich D-25  
**BORING DEPTH:** 55.0 ft  
**WATER LEVEL:** 4' ATD, 5' 24 hr

**DRILLER:** Mike  
**LOGGED BY:** GHG

**HAMMER TYPE:** Automatic  
**SAMPLING METHOD:** Split spoon

**DRILLING METHOD:** Mud Rotary

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
<th>WATER LEVEL (feet)</th>
<th>ELEVATION (feet)</th>
<th>SAMPLE NO.</th>
<th>SAMPLE TYPE</th>
<th>BLOW COUNT</th>
<th>CORE DATA</th>
<th>STANDARD PENETRATION TEST DATA (blows/ft)</th>
<th>REMARKS</th>
<th>N VALUE</th>
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<td>TOPSOIL - Approximately 6 inches thick.</td>
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<td>POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sand, few low plasticity fines, dark brown, moist, medium dense.</td>
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**NOTES:**

1. **THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.**
2. **BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.**
3. **STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.**
4. **WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.**

**DATE DRILLED:** 3/22/16  
**ELEVATION:**  
**NOTES:** Elevation Unknown

**DRILL RIG:** Diedrich D-25  
**BORING DEPTH:** 55.0 ft  
**WATER LEVEL:** 4' ATD, 5' 24 hr

**DRILLER:** Mike  
**LOGGED BY:** GHG

**HAMMER TYPE:** Automatic  
**SAMPLING METHOD:** Split spoon

**DRILLING METHOD:** Mud Rotary
Brooks Stadium Expansion
Conway, South Carolina
S&ME Project No. 1463-16-012

DATE DRILLED: 3/22/16
ELEVATION:

NOTES: Elevation Unknown

DRILL RIG: Diedrich D-25
BORING DEPTH: 55.0 ft

WATER LEVEL: 4’ ATD, 5’ 24 hr

HAMMER TYPE: Automatic
LOGGED BY: GHG

SAMPLING METHOD: Split spoon
NORTHING: EASTING:

DRILLING METHOD: Mud Rotary

MATERIAL DESCRIPTION

GRAPHIC LOG

DEPTH (feet)

ELEVATION (feet)

WATER LEVEL

SAMPLE NO.

SAMPLE TYPE

1st 6in / RUN #

2nd 6in / REC

3rd 6in / RQD

STANDARD PENETRATION TEST DATA (blows/ft)

N VALUE

80 60 30 20 10

BORING TERMINATED AT 55 FT
TARGET DEPTH

- - - - Very hard lens at 31 feet (transition into Pee Dee Formation) 4 inches thick

SILTY SAND (SM) - PEE DEE FORMATION - Mostly fine to medium sand, some low plasticity fines, shell fragments, gray, greenish blue, saturated, loose.

- - - - Very hard lens at 37 feet (6 inches thick)

SANDY FAT CLAY (CH) - Mostly medium to high plasticity fines, some fine sand, silty, dark gray, moist, stiff.

- - - - Very hard lens at 41 feet (4 inches thick)

ELASTIC SILT WITH SAND (MH) - Mostly medium to high plasticity fines, few fine sands, dark gray, moist, stiff.

- - - - Very hard

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3. STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.
4. WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.

S&ME BORING LOG 1463-16-012 LOGS.GPJ S&ME 2011.10.21 09:48:36

Page 2 of 2
1. This log is only a portion of a report prepared for the named project and must only be used together with that report.

2. Boring, sampling, and penetration test data in general accordance with ASTM D-1586.

3. Stratification and groundwater depths are not exact.

4. Water level is at time of exploration and will vary.
3rd 6in / RQD
GRAPHIC LOG

MATERIAL DESCRIPTION

- - - - Very hard lens at 31 feet (transition into Pee Dee Formation) 7 inches thick.

SANDY FAT CLAY (CH) - PEE DEE FORMATION - Mostly medium to high plasticity fines, some fine sand, dark gray, moist, stiff.

- - - - Hard lens at 37 feet.

- - - - very stiff.

- - - - stiff, saturated.

Boring terminated at 50 ft Target Depth

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3. STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.
4. WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 6</td>
<td></td>
<td>TOPSOIL - Approximately 6 inches thick.</td>
</tr>
<tr>
<td>7 - 10</td>
<td></td>
<td>POORLY GRADED SAND (SP) - Mostly fine to medium sand, orange, brown, moist, medium dense.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- - - loose.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- - - dark brown, saturated, loose.</td>
</tr>
<tr>
<td>11 - 15</td>
<td></td>
<td>POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sand, some low plasticity fines, red, brown, saturated, loose.</td>
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<tr>
<td>16 - 20</td>
<td></td>
<td>SANDY LEAN CLAY (CL) - Mostly low to medium plasticity fines, some fine sand, gray, saturated, very soft.</td>
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<tr>
<td>21 - 25</td>
<td></td>
<td>POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sand, few low plasticity fines, shell fragments, gray, saturated, loose.</td>
</tr>
<tr>
<td>26 - 30</td>
<td></td>
<td>POORLY GRADED SAND WITH CLAY (SP-SC) - Mostly fine to coarse sand, few to low medium plasticity fines, gray to blue, wet, loose.</td>
</tr>
<tr>
<td>31 - 35</td>
<td></td>
<td>POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sand, few low plasticity fines, shell fragments, gray, saturated, loose.</td>
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</table>

ELEVATION: 50.0 ft

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3. STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.
4. WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.
SANDY FAT CLAY (CH) - Mostly medium to high plasticity fines, some fine sand, dark gray, saturated, firm. (continued)

- - - Very hard lens at 31 feet (transition into Pee Dee Formation)

SANDY LEAN CLAY (CL) - Mostly low to medium plasticity fines, some fine sand, dark gray, saturated, very hard.

- - - Hard drilling at 38 feet.

SANDY FAT CLAY (CH) - Mostly medium to high plasticity fines, some fine sand, dark gray, saturated, very stiff.

- - - Hard drilling at 43 feet.

- - - stiff.

- - - Hard drilling at 47 feet.

ELASTIC SILT WITH SAND (MH) - Mostly medium to high plasticity fines, few fine sands, gray, saturated, very hard.

Boring terminated at 50 ft Target Depth

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50/4”

10

4

5

4

50/4”

41

9

12

50/5”

Boring terminated at 50 ft
Target Depth

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4. Water level is at time of exploration and will vary.
**TABLE 1: INFILTRATION RATE OF SOILS IN FIELD**

( BY DOUBLE RING INFILTROMETER )

**JOB NAME:** Brooks Stadium Expansion

**JOB NO.:** 1463-16-012

**REPORT NO.:** I-1

**TEST DATE:** 03/31/16

**INVESTIGATOR:** CJ

**BORING NO.:** I-1

**DEPTH / ELEV.:** -1 Feet

**REVIEWED BY:** RF

**BORING LOCATION:** See Test Location Sketch

**SOIL DESCRIPTION:** Silty Sand (SM), Poorly Graded Sand (SP)

<table>
<thead>
<tr>
<th>CONSTANTS</th>
<th>AREA CM²</th>
<th>DEPTH OF LIQUID CM</th>
<th>MARIOTTE TUBE NO.</th>
<th>ΔVOLUME / ΔH CM³ / CM</th>
</tr>
</thead>
<tbody>
<tr>
<td>INNER RING</td>
<td>729.7</td>
<td>15.2</td>
<td>1</td>
<td>1</td>
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<tr>
<td>ANNULAR SPACE</td>
<td>2105.0</td>
<td>15.2</td>
<td>2</td>
<td>1</td>
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<thead>
<tr>
<th>READING NO.</th>
<th>DATE</th>
<th>TIME</th>
<th>ELAPSED TIME</th>
<th>FLOW READINGS</th>
<th>LIQUID TEMP.</th>
<th>INFILTRATION RATE</th>
<th>REMARKS</th>
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</thead>
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<td>INNER RING CM</td>
<td>ANNULAR SPACE CM</td>
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<td>2</td>
<td>S</td>
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<td>30 MINUTES</td>
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<td>3</td>
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<td>6</td>
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<td>9</td>
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# TABLE 1: INFILTRATION RATE OF SOILS IN FIELD
( BY DOUBLE RING INFILTROMETER )

**JOB NAME:** Brooks Stadium Expansion  
**JOB NO.:** 1463-16-012  
**REPORT NO.:** I-2  
**TEST DATE:** 03/30/16  
**INVESTIGATOR:** CJ  
**BORING NO.:** I-2  
**DEPTH / ELEV.:** -1 Feet  
**REVIEWED BY:** RF  
**BORING LOCATION:** See Test Location Sketch  
**SOIL DESCRIPTION:** Tan Poorly Graded Sand (SP)

### CONSTANTS

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<tr>
<th></th>
<th>AREA CM$^2$</th>
<th>DEPTH OF LIQUID CM</th>
<th>MARIOTTE TUBE NO.</th>
<th>ΔVOLUME / ΔH CM$^3$ / CM</th>
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<tbody>
<tr>
<td>INNER RING</td>
<td>729.7</td>
<td>15.2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>ANNULAR SPACE</td>
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<td>15.2</td>
<td>2</td>
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### FLOW READINGS

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<th>TIME ELAPSED</th>
<th>FLOW READINGS</th>
<th>LIQUID TEMP.</th>
<th>INFILTRATION RATE</th>
<th>REMARKS</th>
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<td></td>
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<td>HR:MIN:SEC</td>
<td>MINUTES</td>
<td>IN. / HOUR</td>
<td>INN. / HOUR</td>
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TABLE 1: INFILTRATION RATE OF SOILS IN FIELD (BY DOUBLE RING INFILTROMETER)

<table>
<thead>
<tr>
<th>JOB NAME</th>
<th>Brooks Stadium Expansion</th>
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<tbody>
<tr>
<td>JOB NO.</td>
<td>1463-16-012</td>
</tr>
<tr>
<td>REPORT NO.</td>
<td>I-3</td>
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<td>TEST DATE</td>
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<td>INVESTIGATOR</td>
<td>CJ</td>
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<tr>
<td>BORING NO.</td>
<td>I-3</td>
</tr>
<tr>
<td>DEPTH / ELEV.</td>
<td>-1 Feet</td>
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<td>REVIEWED BY</td>
<td>RF</td>
</tr>
<tr>
<td>BORING LOCATION</td>
<td>See Test Location Sketch</td>
</tr>
<tr>
<td>SOIL DESCRIPTION</td>
<td>Silty Sand (SM)</td>
</tr>
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<table>
<thead>
<tr>
<th>CONSTANTS</th>
<th>AREA CM²</th>
<th>DEPTH OF LIQUID CM</th>
<th>MARIOTTE TUBE NO.</th>
<th>ΔVOLUME / ΔH CM³ / CM</th>
</tr>
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<tbody>
<tr>
<td>INNER RING</td>
<td>729.7</td>
<td>15.2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>ANNULAR SPACE</td>
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<td>15.2</td>
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<table>
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<th>READING NO.</th>
<th>DATE</th>
<th>TIME</th>
<th>ELAPSED TIME</th>
<th>FLOW READINGS</th>
<th>LIQUID TEMP.</th>
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<th>REMARKS</th>
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<td></td>
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<td>ELAPSED TIME</td>
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<td>INNER RING</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>03/30/16</td>
<td></td>
<td></td>
<td>INNER RING</td>
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<tr>
<td>8</td>
<td>03/30/16</td>
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<td>0</td>
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<td>9</td>
<td>03/30/16</td>
<td></td>
<td></td>
<td>INNER RING</td>
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<td>0</td>
</tr>
<tr>
<td>10</td>
<td>03/30/16</td>
<td></td>
<td></td>
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<td>0</td>
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</tr>
</tbody>
</table>
### MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>GRAPHIC LOG</th>
<th>TOPSOIL - Approximately 4 inches.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>SILTY SAND (SM) - Mostly fine to medium sand, some low plasticity fines, dark brown, moist to wet.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>POORLY GRADED SAND (SP) - Mostly fine to medium sand, tan, moist to wet.</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>SILTY SAND (SM) - Mostly fine to medium sand, some low plasticity fines, dark brown, saturated.</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>POORLY GRADED SAND (SP) - Mostly fine to medium sand, tan, saturated.</td>
</tr>
</tbody>
</table>

Boring terminated at 4 ft
Target Depth

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.
**Project:** Brooks Stadium Expansion  
Conway, South Carolina  
1463-16-012

**Hand Auger Boring Log: I-2**

**Date Started:** 3/30/16  
**Date Finished:** 3/30/16  
**Notes:** Elevation Unknown

**Sampling Method:** Hand Auger  
**Performed By:** CDJ

**Water Level:** 3' ATD

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphic Log</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Topsoil - Approximately 2 inches.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Poorly Graded Sand (SP) - Mostly fine to medium sand, tan, brown, moist to wet.</td>
</tr>
</tbody>
</table>
| 3            |             | Poorly Graded Sand with Clay (SP-SC) - Mostly fine to medium sand, few low plasticity fines, tan, orange, saturated.  
|              |             | Trace of gravel |
| 4            |             | Boring terminated at 4 ft Target Depth |

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.
**TOPSOIL** - Approximately 6 inches.

**SILTY SAND (SM)** - Mostly fine to medium sand, some low plasticity fines, trace of gravel, brown, wet to saturated.

**CLAYEY SAND (SC)** - Mostly fine to medium sand, some low to medium plasticity fines, tan, saturated.

**POORLY GRADED SAND (SP)** - Mostly fine to medium sand, tan, saturated.

**DATE STARTED:** 3/30/16  
**DATE FINISHED:** 3/30/16  
**SAMPLING METHOD:** Hand Auger  
**PERFORMED BY:** CDJ  
**WATER LEVEL:** 1.5' ATD  
**PROJECT:** Brooks Stadium Expansion  
**Conway, South Carolina**  
**1463-16-012**  
**NOTES:** Elevation Unknown

**MATERIAL DESCRIPTION**

<table>
<thead>
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<th>Depth (feet)</th>
<th>ELEVATION (feet)</th>
<th>WATER LEVEL</th>
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</thead>
<tbody>
<tr>
<td>0</td>
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<td></td>
</tr>
<tr>
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<tr>
<td>2</td>
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</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Boring terminated at 4 ft Target Depth

**GRAPHIC LOG**

**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.  

**DATE FINISHED:** 1463-16-012  
**DATE STARTED:** 3/30/16  
**CONWAY, SOUTH CAROLINA**  
**3/30/16 3/30/16**  
**ELEVATION UNKNOWN**  
**CDJ**  
**PROJECT:** Brooks Stadium Expansion  
**PERFORMED BY:** CDJ  
**WATER LEVEL:** 1.5' ATD  
**SAMPLING METHOD:** Hand Auger  
**PROJECT:** Brooks Stadium Expansion  
**Conway, South Carolina**  
**1463-16-012**  
**NOTES:** Elevation Unknown

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>ELEVATION (feet)</th>
<th>WATER LEVEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Boring terminated at 4 ft Target Depth

**GRAPHIC LOG**

**DCP INDEX** is the depth (in.) of penetration per blow of a 10.1 lb hammer falling 22.6 in., driving a 0.79 in. O.D. 60 degree cone.
Appendix III

Summary of Laboratory Test Procedures

Laboratory Test Results
Summary of Laboratory Procedures

Examination of Recovered Soil Samples

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

Moisture Content Testing of Soil Samples by Oven Drying

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

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

Liquid and Plastic Limits Testing

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

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

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

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

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

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

**Unconfined Compressive Strength Tests of Undisturbed Cohesive Samples**
The unconfined compressive strength of relatively undisturbed cohesive soils was determined generally following the procedures described by ASTM D 2166, "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil." Relatively undisturbed Group C samples of cohesive soils were extruded from the sampler and examined as described above. Representative portions of each sample were split from the extruded material and prepared using the procedures described in Section 6.2 of ASTM D 2166. The ends of the specimen were carved by hand and trimmed as necessary to provide a surface perpendicular to the specimen's long axis, but the ends were not capped.

The prepared sample was placed in a compressive testing machine and the specimen compressed in the platen at a rate of 1 to 2 percent strain per minute. Deformation and loading of the sample were recorded at regular intervals until the load values began to decrease with increasing axial strain, or a total strain of 15 percent of the original sample length was attained. Sample stress
was corrected at each load increment for the change in cross sectional area produced by deformation of the sample using the formula in sections 8.2 and 8.3 of ASTM D 2166.
Sieve Analysis of Soils

ASTM D 422

Quality Assurance

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

Project #: 1463-16-012
Project Name: Brooks Stadium Expansion and Renovation
Client Name: Coastal Carolina University
Client Address: PO Box 261954; Conway, SC 29528
Boring #: SPT-1
Sample #: --
Sample Date: 3/21-22/2016
Location: Borings
Lab #: 3816
Depth: 3.5'-5.0'
Sample Description: Dark Brown Poorly Graded Sand with Silt (SP-SM)

<table>
<thead>
<tr>
<th>Percent Passing (%)</th>
<th>100.00</th>
<th>10.00</th>
<th>1.00</th>
<th>0.10</th>
<th>0.01</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;</td>
<td>1.5&quot;</td>
<td>1&quot; 3/4&quot;</td>
<td>3/8&quot;</td>
<td>#4</td>
<td>#20</td>
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<table>
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<tr>
<th>Cobble</th>
<th>&lt; 300 mm (12&quot;) and &gt; 75 mm (3&quot;)</th>
<th>Gravel</th>
<th>&lt; 75 mm and &gt; 4.75 mm (#4)</th>
<th>Coarse Sand</th>
<th>&lt; 4.75 mm and &gt; 2.00 mm (#10)</th>
<th>Medium Sand</th>
<th>&lt; 2.00 mm and &gt; 0.425 mm (#40)</th>
<th>Coarse Sand</th>
<th>Fine Sand</th>
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<tr>
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<tr>
<td>0.850 mm</td>
<td>0.0%</td>
<td>0.0%</td>
<td>23.9%</td>
<td>0.0%</td>
<td>66.3%</td>
<td>Fine Sand</td>
<td>66.3%</td>
<td></td>
<td></td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Liquid Limit</td>
<td>Plastic Limit</td>
<td>--</td>
<td>Plastic Index</td>
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<td>Specific Gravity</td>
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<td>Cc= 2.200</td>
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<td>Moisture Content</td>
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<td>23.9%</td>
<td>Fine Sand</td>
<td>66.3%</td>
<td></td>
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</table>

Description of Sand & Gravel Particles:
- Rounded □
- Angular X
- Hard & Durable X
- Soft □
- Weathered & Friable □

Notes / Deviations / References:

Chelsea Jones
Technical Responsibility

Signature

Staff Professional
Position

4/25/16
Date

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

Sieve Analysis of Soils

ASTM D 422

Project #: 1463-16-012  Report Date: 3/31/2016

Project Name: Brooks Stadium Expansion and Renovation  Test Date(s): 3/30/2016

Client Name: Coastal Carolina University  Boring #: SPT-1

Client Address: PO Box 261954; Conway, SC 29528  Sample #: --

Location: Borings  Lab #: 3816  Depth: 3.5'-5.0'

Sample Description: Dark Brown Poorly Graded Sand with Silt (SP-SM)

Description of Sand & Gravel Particles: Rounded  □  Angular  ☑

Hard & Durable  ☑  Soft  □  Weathered & Friable  □

Particle Size Analysis / Without Hydrometer Analysis

Material Excluded:

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<tr>
<th>Tare No.</th>
<th>Blue</th>
<th>Tare Wt.</th>
<th>% Retained Between Sieves</th>
<th>% Retained</th>
<th>% Passing</th>
<th>% Passing #200 (D1140)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Blue</td>
<td>84.0</td>
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<tr>
<td></td>
<td>Mass of Sample after Wash + Tare Wt.</td>
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<tr>
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<td>Total Sample Wet Wt. + Tare Wt.</td>
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</tr>
<tr>
<td></td>
<td>Total Sample Dry Wt. + Tare Wt.</td>
<td>127.9</td>
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<tr>
<td></td>
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</tr>
<tr>
<td></td>
<td>Total Sample Dry Weight</td>
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</table>

<table>
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<tr>
<th>Sieve Size</th>
<th>Retained Weight</th>
<th>% Retained Between Sieves</th>
<th>% Retained</th>
<th>% Passing</th>
<th>SPECs</th>
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<tr>
<td>Standard</td>
<td>mm.</td>
<td>Cumulative</td>
<td>Individual</td>
<td>Cumulative Total Sample</td>
<td>% Passing #200 (D1140) =</td>
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<tr>
<td>2.0&quot;</td>
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<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
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<tr>
<td>1.5&quot;</td>
<td>37.50</td>
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<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>1.0&quot;</td>
<td>25.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>19.00</td>
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<td>0.0%</td>
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<td>1/2&quot;</td>
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<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.50</td>
<td>0.0</td>
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<td>100.0%</td>
</tr>
<tr>
<td>#4</td>
<td>4.75</td>
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<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#20</td>
<td>0.850</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#30</td>
<td>0.600</td>
<td>4.5</td>
<td>10.3%</td>
<td>10.3%</td>
<td>89.7%</td>
</tr>
<tr>
<td>#40</td>
<td>0.425</td>
<td>10.5</td>
<td>13.7%</td>
<td>23.9%</td>
<td>76.1%</td>
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<tr>
<td>#60</td>
<td>0.250</td>
<td>25.6</td>
<td>34.4%</td>
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<td>41.7%</td>
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<tr>
<td>#100</td>
<td>0.150</td>
<td>38.5</td>
<td>29.4%</td>
<td>87.7%</td>
<td>12.3%</td>
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<tr>
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<td>39.6</td>
<td>2.5%</td>
<td>90.2%</td>
<td>9.8%</td>
</tr>
<tr>
<td>Pan</td>
<td>&lt;0.075</td>
<td>39.8</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
</tbody>
</table>

Notes / Deviations / References:

Chelsea Jones  Signature  Staff Professional  Date

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# Unconfined Compressive Strength Test

**S&ME, Inc. Myrtle Beach, 1330 Highway 501 Business, Conway, South Carolina 29526**

## Project Information

- **Project #:** 1463-16-012  
- **Report Date:** 3/31/2016  
- **Project Name:** Brooks Stadium Expansion and Renovation  
- **Test Date(s):** 3/30/2016  
- **Client Name:** Coastal Carolina University  
- **Client Address:** PO Box 261954; Conway, SC 29528  
- **Boring #:** SPT-2  
- **Sample #:** --  
- **Sample Date:** 3/21-22/2016  
- **Location:** Borings  
- **Lab #:** 3816  
- **Depth:** 13.5'-15.0'

## Sample Description

- **Sample Description:** Gray Sandy Fat Clay (CH)

## Test Equipment

<table>
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<tr>
<th>Type and Specification</th>
<th>S&amp;ME ID</th>
<th>Cal. Date</th>
<th>Type and Specification</th>
<th>S&amp;ME ID</th>
<th>Cal. Date</th>
</tr>
</thead>
<tbody>
<tr>
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<td>13161</td>
<td>1/19/2015</td>
<td>Sensor</td>
<td>13163</td>
<td>1/19/2015</td>
</tr>
</tbody>
</table>

## Load vs. Total Strain

<table>
<thead>
<tr>
<th>Dial</th>
<th>Axial (lb)</th>
<th>Total Strain (in)</th>
<th>Unit Strain</th>
<th>Corrected Area (in²)</th>
<th>Stress (psi)</th>
<th>Stress (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.75</td>
<td>9.75</td>
<td>0.014</td>
<td>0.25%</td>
<td>6.44</td>
<td>1.51</td>
<td>217.96</td>
</tr>
<tr>
<td>12.27</td>
<td>12.27</td>
<td>0.027</td>
<td>0.50%</td>
<td>6.46</td>
<td>1.90</td>
<td>273.57</td>
</tr>
<tr>
<td>13.89</td>
<td>13.89</td>
<td>0.041</td>
<td>0.75%</td>
<td>6.47</td>
<td>2.15</td>
<td>309.03</td>
</tr>
<tr>
<td>15.19</td>
<td>15.19</td>
<td>0.055</td>
<td>1.00%</td>
<td>6.49</td>
<td>2.34</td>
<td>337.08</td>
</tr>
<tr>
<td>16.33</td>
<td>16.33</td>
<td>0.068</td>
<td>1.25%</td>
<td>6.51</td>
<td>2.51</td>
<td>361.42</td>
</tr>
<tr>
<td>17.06</td>
<td>17.06</td>
<td>0.082</td>
<td>1.50%</td>
<td>6.52</td>
<td>2.62</td>
<td>376.64</td>
</tr>
<tr>
<td>17.71</td>
<td>17.71</td>
<td>0.096</td>
<td>1.75%</td>
<td>6.54</td>
<td>2.71</td>
<td>390.00</td>
</tr>
<tr>
<td>18.03</td>
<td>18.03</td>
<td>0.109</td>
<td>2.00%</td>
<td>6.56</td>
<td>2.75</td>
<td>396.15</td>
</tr>
<tr>
<td>18.12</td>
<td>18.12</td>
<td>0.123</td>
<td>2.25%</td>
<td>6.57</td>
<td>2.76</td>
<td>396.91</td>
</tr>
<tr>
<td>17.38</td>
<td>17.38</td>
<td>0.137</td>
<td>2.50%</td>
<td>6.59</td>
<td>2.64</td>
<td>379.92</td>
</tr>
</tbody>
</table>

## Graph

**Unconfined Compressive Strength**

### Notes/Deviations/References:

**ASTM D 2166: Standard Test Method for Unconfined Compressive Strength of Cohesive Soil**

- **Chelsea Jones**  
  Technical Responsibility
- **Signature**  
  Position
- **Staff Professional**  
  Date

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---

**S&ME, Inc. - Myrtle Beach**  
1330 Highway 501 Business  
Conway, South Carolina 29526  
SPT-2 Unconfined Compressive Strength.xls  
Page 1 of 1
Sieve Analysis of Soils

ASTM D 422

Quality Assurance

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

Project #: 1463-16-012
Report Date: 3/31/2016

Project Name: Brooks Stadium Expansion and Renovation
Test Date(s): 3/30/2016

Client Name: Coastal Carolina University

Client Address: PO Box 261954; Conway, SC 29528

Boring #: SPT-2
Sample #: -
Sample Date: 3/21-22/2016

Location: Borings
Lab #: 3816
Depth: 13.5'-15.0'

Sample Description: Gray Sandy Fat Clay (CH)

---

<table>
<thead>
<tr>
<th>颗粒级配</th>
<th>坚实性</th>
<th>硬度</th>
<th>湿度</th>
<th>位置</th>
<th>日期</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes / Deviations / References:

Chelsea Jones
Technical Responsibility

Signature

Staff Professional
Position

Date

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# Sieve Analysis of Soils

**ASTM D 422**

*Quality Assurance*

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

**Project #:** 1463-16-012  
**Report Date:** 3/31/2016

**Project Name:** Brooks Stadium Expansion and Renovation  
**Test Date(s):** 3/30/2016

**Client Name:** Coastal Carolina University  
**Client Address:** PO Box 261954; Conway, SC 29528

**Boring #:** SPT-2  
**Sample #:** --  
**Sample Date:** 3/21-22/2016

**Location:** Borings  
**Lab #:** 3816  
**Depth:** 13.5'-15.0'

**Sample Description:** Gray Sandy Fat Clay (CH)

### Description of Sand & Gravel Particles:
- Rounded [ ]  
- Angular [X]  
- Hard & Durable [X]  
- Soft [ ]  
- Weathered & Friable [ ]

### Particle Size Analysis / Without Hydrometer Analysis

<table>
<thead>
<tr>
<th>Tare No.</th>
<th>UUU</th>
<th>Tare Wt.</th>
<th>82.4</th>
<th>Mass of Sample after Wash + Tare Wt.</th>
<th>84.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Sample Wet Wt. + Tare Wt.</td>
<td>99.5</td>
<td>Mass of Sample after Wash</td>
<td>1.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Sample Dry Wt. + Tare Wt.</td>
<td>92.7</td>
<td>Mass passing #200</td>
<td>8.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Sample Dry Weight</td>
<td>10.3</td>
<td>% Passing #200 (D1140)</td>
<td>83.5%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Sieve Size Analysis

<table>
<thead>
<tr>
<th>Standard</th>
<th>Retained Weight</th>
<th>% Retained Between Sieves</th>
<th>% Retained</th>
<th>% Passing</th>
<th>SPECs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Individual</td>
<td></td>
<td>Cumulative</td>
<td>Total Sample</td>
<td></td>
</tr>
<tr>
<td>2.0&quot;</td>
<td>50.00</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>37.50</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>1.0&quot;</td>
<td>25.00</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>19.00</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>12.50</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.50</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#4</td>
<td>4.75</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#20</td>
<td>0.850</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#30</td>
<td>0.600</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#40</td>
<td>0.425</td>
<td>0.00%</td>
<td>0.0%</td>
<td>0.0%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#60</td>
<td>0.250</td>
<td>1.0%</td>
<td>1.0%</td>
<td>1.0%</td>
<td>99.0%</td>
</tr>
<tr>
<td>#100</td>
<td>0.150</td>
<td>3.9%</td>
<td>2.9%</td>
<td>2.9%</td>
<td>97.1%</td>
</tr>
<tr>
<td>#200</td>
<td>0.075</td>
<td>12.6%</td>
<td>15.5%</td>
<td>15.5%</td>
<td>84.5%</td>
</tr>
<tr>
<td>Pan</td>
<td>&lt; 0.075</td>
<td>1.6</td>
<td>% Passing #200 (D1140) = 84.5%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### D2487

<table>
<thead>
<tr>
<th>Maximum Particle Size</th>
<th>Retained Weight</th>
<th>% Retained Between Sieves</th>
<th>% Retained</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.425 mm</td>
<td>Medium Sand</td>
<td>&lt; 2.00 mm and &gt; 0.425 mm (#4)</td>
<td>0.0%</td>
<td></td>
</tr>
<tr>
<td>0.0%</td>
<td>Fine Sand</td>
<td>&lt; 0.425 mm and &gt; 0.075 mm (#200)</td>
<td>15.5%</td>
<td></td>
</tr>
<tr>
<td>0.0%</td>
<td>% Silt &amp; Clay</td>
<td>&lt; 0.075 mm</td>
<td>84.5%</td>
<td></td>
</tr>
</tbody>
</table>

### Notes / Deviations / References:

---

**Signed:**

*Chelsea Jones*

**Technical Responsibility**

*Signature*

**Staff Professional**

**Position**

**Date:** 4/25/16

---

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Liquid Limit, Plastic Limit, and Plastic Index

**ASTM D 4318**

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

**Project #:** 1463-16-012  
**Report Date:** 3/31/2016

**Project Name:** Brooks Stadium Expansion and Renovation  
**Test Date(s):** 3/30/2016

**Client Name:** Coastal Carolina University

**Client Address:** PO Box 261954; Conway, SC 29528

**Boring #:** SPT-2  
**Sample #:** --  
**Sample Date:** 3/21-22/2016

**Location:** Borings  
**Lab #:** 3816  
**Depth:** 13.5'-15.0'

**Sample Description:** Gray Sandy Fat Clay (CH)

<table>
<thead>
<tr>
<th>Pan #</th>
<th>Type and Specification</th>
<th>S&amp;ME ID #</th>
<th>Cal Date</th>
<th>Type and Specification</th>
<th>S&amp;ME ID #</th>
<th>Cal Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Balance (0.01 g)</td>
<td>00401</td>
<td>2/18/2015</td>
<td>Grooving tool</td>
<td>11368</td>
<td>5/1/2015</td>
</tr>
<tr>
<td>LL Apparatus</td>
<td>18801</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oven</td>
<td>17745</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pan #</th>
<th>Tare #:</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>106 114 96 4 5 6 74 102 9</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Tare Weight</td>
<td>14.96 14.88 14.69</td>
<td>14.74 14.36</td>
</tr>
<tr>
<td>C</td>
<td>Dry Soil Weight + A</td>
<td>19.21 19.42 19.04</td>
<td>20.33 20.30</td>
</tr>
<tr>
<td>D</td>
<td>Water Weight (B-C)</td>
<td>2.12 2.46 2.59</td>
<td>0.91 0.99</td>
</tr>
<tr>
<td>E</td>
<td>Dry Soil Weight (C-A)</td>
<td>4.25 4.54 4.35</td>
<td>5.59 5.94</td>
</tr>
<tr>
<td>F</td>
<td>% Moisture (D/E)*100</td>
<td>49.9% 54.2% 59.5%</td>
<td>16.3% 16.7%</td>
</tr>
<tr>
<td>N</td>
<td># OF DROPS</td>
<td>35 25 15</td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>LL = F * FACTOR</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ave.</td>
<td>Average</td>
<td></td>
<td>16.5%</td>
</tr>
</tbody>
</table>

**One Point Liquid Limit**

<table>
<thead>
<tr>
<th>N</th>
<th>Factor</th>
<th>N</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.974</td>
<td>26</td>
<td>1.005</td>
</tr>
<tr>
<td>21</td>
<td>0.979</td>
<td>27</td>
<td>1.009</td>
</tr>
<tr>
<td>22</td>
<td>0.985</td>
<td>28</td>
<td>1.014</td>
</tr>
<tr>
<td>23</td>
<td>0.999</td>
<td>29</td>
<td>1.018</td>
</tr>
<tr>
<td>24</td>
<td>0.995</td>
<td>30</td>
<td>1.022</td>
</tr>
<tr>
<td>25</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NP, Non-Plastic

- Liquid Limit 54
- Plastic Limit 17
- Plastic Index 37

**Group Symbol:** CH

**Multipoint Method:**

- One-point Method

**Notes / Deviations / References:**

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

**Chelsea Jones**
Technical Responsibility

**Staff Professional**

**4/25/16**

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Liquid Limit, Plastic Limit, and Plastic Index

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

Project #: 1463-16-012  Report Date: 3/31/2016
Project Name: Brooks Stadium Expansion and Renovation  Test Date(s): 3/30/2016
Client Name: Coastal Carolina University
Boring #: SPT-3  Sample #: --  Sample Date: 3/21-22/2016
Location: Borings  Lab #: 3816  Depth: 28.5'-30.0'
Sample Description: Gray Clayey Sand (SC)

<table>
<thead>
<tr>
<th>Balance (0.01 g)</th>
<th>LL Apparatus</th>
<th>Oven</th>
</tr>
</thead>
<tbody>
<tr>
<td>00401</td>
<td>18801</td>
<td>17745</td>
</tr>
<tr>
<td>2/18/2015</td>
<td>5/1/2015</td>
<td>5/6/2015</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type and Specification</th>
<th>S&amp;ME ID #</th>
<th>Cal Date</th>
<th>Type and Specification</th>
<th>S&amp;ME ID #</th>
<th>Cal Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>S &amp; ME ID #</td>
<td></td>
<td></td>
<td>S &amp; ME ID #</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL Apparatus</td>
<td>18801</td>
<td>5/1/2015</td>
<td>Grooving tool</td>
<td>11368</td>
<td>5/1/2015</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pan #</th>
<th>Tare #</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>114</td>
<td>95</td>
</tr>
<tr>
<td>A</td>
<td>Tare Weight</td>
<td>14.85</td>
<td>14.88</td>
</tr>
<tr>
<td>B</td>
<td>Wet Soil Weight + A</td>
<td>22.80</td>
<td>22.85</td>
</tr>
<tr>
<td>C</td>
<td>Dry Soil Weight + A</td>
<td>20.76</td>
<td>20.66</td>
</tr>
<tr>
<td>D</td>
<td>Water Weight (B-C)</td>
<td>2.04</td>
<td>2.19</td>
</tr>
<tr>
<td>E</td>
<td>Dry Soil Weight (C-A)</td>
<td>5.91</td>
<td>5.78</td>
</tr>
<tr>
<td>F</td>
<td>% Moisture (D/E)*100</td>
<td>34.5%</td>
<td>37.9%</td>
</tr>
<tr>
<td>N</td>
<td># OF DROPS</td>
<td>35</td>
<td>24</td>
</tr>
<tr>
<td>LL</td>
<td>LL = F * FACTOR</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ave.</td>
<td>Average</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Moisture Contents determined by ASTM D 2216
13.8%

One Point Liquid Limit

<table>
<thead>
<tr>
<th>N</th>
<th>Factor</th>
<th>N</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.974</td>
<td>26</td>
<td>1.005</td>
</tr>
<tr>
<td>21</td>
<td>0.979</td>
<td>27</td>
<td>1.009</td>
</tr>
<tr>
<td>22</td>
<td>0.983</td>
<td>28</td>
<td>1.014</td>
</tr>
<tr>
<td>23</td>
<td>0.999</td>
<td>29</td>
<td>1.018</td>
</tr>
<tr>
<td>24</td>
<td>0.995</td>
<td>30</td>
<td>1.022</td>
</tr>
<tr>
<td>25</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NP, Non-Plastic

| Liquid Limit | 38 |
| Plastic Limit | 14 |
| Plastic Index | 24 |
| Group Symbol | SC |

Wet Preparation ☑  Dry Preparation ☐  Air Dried ☑

Notes / Deviations / References:

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

Chelsea Jones  Signature  Staff Professional  Date

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Sieve Analysis of Soils

**ASTM D 422**

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

**Project #:** 1463-16-012

**Report Date:** 3/31/2016

**Project Name:** Brooks Stadium Expansion and Renovation

**Test Date(s):** 3/30/2016

**Client Name:** Coastal Carolina University

**Client Address:** PO Box 261954; Conway, SC 29528

**Boring #** SPT-4

**Sample #:** --

**Sample Date:** 3/21-22/2016

**Location:** Borings

**Lab #:** 3816

**Depth:** 23.5'-25.0'

**Sample Description:** Gray Poorly Graded Sand with Silt (SP-SM)

### Grain Size Analysis

<table>
<thead>
<tr>
<th>Particle Size</th>
<th>Percent Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4</td>
<td>100</td>
</tr>
<tr>
<td>#20</td>
<td>50</td>
</tr>
<tr>
<td>#30</td>
<td>20</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
</tr>
<tr>
<td>#60</td>
<td>5</td>
</tr>
<tr>
<td>#100</td>
<td>1</td>
</tr>
<tr>
<td>#200</td>
<td>0.5</td>
</tr>
</tbody>
</table>

### Particle Size Distribution

<table>
<thead>
<tr>
<th>Particle Size</th>
<th>Texture</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 300 mm (12&quot;) and &gt; 75 mm (3&quot;)</td>
<td>Fine Sand</td>
</tr>
<tr>
<td>&lt; 75 mm and &gt; 4.75 mm (#4)</td>
<td>Silt</td>
</tr>
<tr>
<td>&lt; 4.75 mm and &gt;2.00 mm (#10)</td>
<td>Clay</td>
</tr>
<tr>
<td>&lt; 2.00 mm and &gt; 0.425 mm (#40)</td>
<td>Colloids</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Maximum Particle Size</th>
<th>12.5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Sand</td>
<td>12.1%</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>20.7%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Gravel</th>
<th>9.4%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium Sand</td>
<td>47.4%</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>--</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>--</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Coarse Sand</th>
<th>12.1%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium Sand</td>
<td>47.4%</td>
</tr>
</tbody>
</table>

### Description of Sand & Gravel Particles:

- Hard & Durable: ✓
- Soft: □
- Rounded: □
- Angular: ×
- Weathered & Friable: □

---

**Notes / Deviations / References:**

---

**Technical Responsibility:**

**Signature:**

**Staff Professional:**

**Position:**

**Date:** 4/5/16

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---

S&ME, Inc. - Myrtle Beach

1330 Highway 501 Business

Conway, SC 29526

SPT-4 (GRAIN SIZE).xls

Page 2 of 2
### Sieve Analysis of Soils

**ASTM D 422**

**Project #**: 1463-16-012  
**Report Date**: 3/31/2016

**Project Name**: Brooks Stadium Expansion and Renovation  
**Test Date(s)**: 3/30/2016

**Client Name**: Coastal Carolina University

**Client Address**: PO Box 261954; Conway, SC 29528

**Boring #**: SPT-4  
**Sample #:**: --  
**Sample Date**: 3/21-22/2016

**Location**: Borings  
**Lab #:**: 3816  
**Depth**: 23.5' - 25.0'

**Sample Description**: Gray Poorly Graded Sand with Silt (SP-SM)

**Description of Sand & Gravel Particles**: Rounded □  
**Angular**: ×

<table>
<thead>
<tr>
<th>Hard &amp; Durable</th>
<th>Soft</th>
<th>Weathered &amp; Friable</th>
</tr>
</thead>
<tbody>
<tr>
<td>×</td>
<td></td>
<td>□</td>
</tr>
</tbody>
</table>

**Particle Size Analysis / Without Hydrometer Analysis**

<table>
<thead>
<tr>
<th>Tare No.</th>
<th>Tare Wt.</th>
<th>E</th>
<th>Tare Wt.</th>
<th>Mass of Sample after Wash + Tare Wt.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>84.6</td>
<td></td>
<td>141.1</td>
</tr>
</tbody>
</table>

| Total Sample Wet Wt. + Tare Wt. | 163.0 | Mass of Sample after Wash | 56.5 |
| Total Sample Dry Wt. + Tare Wt. | 147.5 | Mass passing #200 | 6.4 |
| Total Sample Dry Weight | 62.9 | % Passing #200 (D1140) | 10.2% |

**Sieve Size** | **Retained Weight** | **% Retained Between Sieves** | **% Retained** | **Cumulative** |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard</strong></td>
<td><strong>mm.</strong></td>
<td><strong>Cumulative</strong></td>
<td><strong>Individual</strong></td>
<td><strong>Total Sample</strong></td>
</tr>
<tr>
<td>2.0&quot;</td>
<td>50.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>37.50</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1.0&quot;</td>
<td>25.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>19.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>12.50</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.50</td>
<td>3.2</td>
<td>5.1%</td>
<td>5.1%</td>
</tr>
<tr>
<td>#4</td>
<td>4.75</td>
<td>5.9</td>
<td>4.3%</td>
<td>9.4%</td>
</tr>
<tr>
<td>#20</td>
<td>0.850</td>
<td>13.5</td>
<td>12.1%</td>
<td>21.5%</td>
</tr>
<tr>
<td>#30</td>
<td>0.600</td>
<td>36.2</td>
<td>36.1%</td>
<td>57.6%</td>
</tr>
<tr>
<td>#40</td>
<td>0.425</td>
<td>43.3</td>
<td>11.3%</td>
<td>68.8%</td>
</tr>
<tr>
<td>#60</td>
<td>0.250</td>
<td>49.3</td>
<td>9.5%</td>
<td>78.4%</td>
</tr>
<tr>
<td>#100</td>
<td>0.150</td>
<td>53.8</td>
<td>7.2%</td>
<td>85.5%</td>
</tr>
<tr>
<td>#200</td>
<td>0.075</td>
<td>56.3</td>
<td>4.0%</td>
<td>89.5%</td>
</tr>
<tr>
<td>Pan</td>
<td>&lt;0.075</td>
<td>56.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| % Passing #200 (D1140) | 10.5% |

**D2487**

- **Maximum Particle Size**: 12.5 mm
- **Medium Sand**: < 2.00 mm and > 0.425 mm (#40) 47.4%
- **Fine Sand**: < 0.425 mm and > 0.075 mm (#200) 20.7%
- **% Silt & Clay**: < 0.075 mm 10.5%

**Notes / Deviations / References:**

---

**Chelsea Jones**  
Technical Responsibility

**Staff Professional**  
Signature  
Position  
Date  

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Liquid Limit, Plastic Limit, and Plastic Index

ASTM D 4318  □ AASHTO T 89  □ AASHTO T 90  □ Quality Assurance

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

Project #: 1463-16-012  Report Date: 3/31/2016
Project Name: Brooks Stadium Expansion and Renovation  Test Date(s): 3/30/2016
Client Name: Coastal Carolina University
Client Address: PO Box 261954; Conway, SC 29528
Boring #: SPT-5  Sample #: --  Sample Date: 3/21-22/2016
Location: Borings  Lab #: 3816  Depth: 33.5'-35.0'
Sample Description: Gray Sandy Lean Clay (CL)

<table>
<thead>
<tr>
<th>Pan #</th>
<th>Tare Weight</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>14.77</td>
<td>14.54</td>
<td>14.63</td>
</tr>
<tr>
<td>B</td>
<td>31.88</td>
<td>32.46</td>
<td>32.12</td>
</tr>
<tr>
<td>C</td>
<td>28.40</td>
<td>28.55</td>
<td>28.02</td>
</tr>
<tr>
<td>D</td>
<td>3.48</td>
<td>3.91</td>
<td>4.10</td>
</tr>
<tr>
<td>E</td>
<td>13.63</td>
<td>14.01</td>
<td>13.39</td>
</tr>
<tr>
<td>F</td>
<td>% Moisture (D/E)*100</td>
<td>25.5%</td>
<td>27.9%</td>
</tr>
<tr>
<td>N</td>
<td># OF DROPS</td>
<td>34</td>
<td>24</td>
</tr>
</tbody>
</table>

LL Apparatus: 18801  5/1/2015
Oven: 17745  5/6/2015

Pan and Tare: 47 88 17 4 5 6 96 36 9

Moisture Contents determined by ASTM D 2216

Ave. Average

11.5%

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

Chelsea Jones  Technical Responsibility

Staff Professional  Position

Date

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# Liquid Limit, Plastic Limit, and Plastic Index

**Project #:** 1463-16-012  
**Report Date:** 3/31/2016  
**Project Name:** Brooks Stadium Expansion and Renovation  
**Test Date(s):** 3/30/2016  
**Client Name:** Coastal Carolina University  
**Client Address:** PO Box 261954; Conway, SC 29528  
**Boring #:** SPT-6  
**Sample #:** --  
**Sample Date:** 3/21-22/2016  
**Location:** Borings  
**Lab #:** 3816  
**Depth:** 13.5'-15.0'  
**Type and Specification:** Gray Sandy Lean Clay (CL)  
**Type and Specification:** S&ME ID #  
**Cal Date:**  
**Balance (0.01 g):** 00401  
**Cal Date:** 2/18/2015  
**LL Apparatus:** 18801  
**Cal Date:** 5/1/2015  
**Oven:** 17745  
**Cal Date:** 5/6/2015

<table>
<thead>
<tr>
<th>Pan #</th>
<th>Tare Weight</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>14.73</td>
<td>14.87</td>
<td>108</td>
</tr>
<tr>
<td>C</td>
<td>24.86</td>
<td>24.78</td>
<td>24.47</td>
</tr>
<tr>
<td>D</td>
<td>2.84</td>
<td>3.32</td>
<td>3.76</td>
</tr>
<tr>
<td>E</td>
<td>10.13</td>
<td>9.91</td>
<td>9.93</td>
</tr>
<tr>
<td>F</td>
<td>28.0%</td>
<td>33.5%</td>
<td>37.9%</td>
</tr>
<tr>
<td>N</td>
<td># OF DROPS</td>
<td>34</td>
<td>22</td>
</tr>
</tbody>
</table>

### Wet Preparation
- [x] Dry Preparation
- [ ] Air Dried

**Notes / Deviations / References:**

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

- **Chelsea Jones**  
  Technical Responsibility  
  Signature

- **Staff Professional**  
  Position  
  Date: 4/25/16

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S&ME, INC. • Myrtle Beach  
1330 Highway 501 Business, Conway, SC 29526
Sieve Analysis of Soils

ASTM D 422

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

Project #: 1463-16-012 Report Date: 3/31/2016
Project Name: Brooks Stadium Expansion and Renovation Test Date(s): 3/30/2016
Client Name: Coastal Carolina University
Client Address: PO Box 261954; Conway, SC 29528
Boring # SPT-6 Sample #: -- Sample Date: 3/21-22/2016
Location: Borings Lab #: 3816 Depth: 18.5'-20.0'

Sample Description: Gray Poorly Graded Sand with Silt (SP-SM)

---

<table>
<thead>
<tr>
<th>Maximum Particle Size</th>
<th>Coarse Sand</th>
<th>Fine Sand</th>
<th>Coarse Sand</th>
<th>Fine Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5 mm</td>
<td>12.0%</td>
<td>36.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.1%</td>
<td>39.9%</td>
<td>6.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--</td>
<td>Plastic Limit</td>
<td>Plastic Index</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>0.610</td>
<td>Cu</td>
<td>Ce= 4.000</td>
<td>Moisture Content</td>
<td></td>
</tr>
<tr>
<td>12.0%</td>
<td>Medium Sand</td>
<td>39.9%</td>
<td>Fine Sand</td>
<td></td>
</tr>
<tr>
<td>Hard &amp; Durable</td>
<td></td>
<td>Soft</td>
<td>Weathered &amp; Friable</td>
<td></td>
</tr>
</tbody>
</table>

Notes / Deviations / References:

Chelsea Jones
Technical Responsibility

Signature

Staff Professional
Position

4/25/16
Date

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# Sieve Analysis of Soils

**ASTM D 422**

## Project Information

<table>
<thead>
<tr>
<th>Project #:</th>
<th>1463-16-012</th>
</tr>
</thead>
<tbody>
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<td>Project Name:</td>
<td>Brooks Stadium Expansion and Renovation</td>
</tr>
<tr>
<td>Report Date:</td>
<td>3/31/2016</td>
</tr>
<tr>
<td>Test Date(s):</td>
<td>3/30/2016</td>
</tr>
<tr>
<td>Client Name:</td>
<td>Coastal Carolina University</td>
</tr>
<tr>
<td>Client Address:</td>
<td>PO Box 261954; Conway, SC 29528</td>
</tr>
<tr>
<td>Boring #:</td>
<td>SPT-6</td>
</tr>
<tr>
<td>Sample #:</td>
<td>--</td>
</tr>
<tr>
<td>Sample Date:</td>
<td>3/21-22/2016</td>
</tr>
<tr>
<td>Location:</td>
<td>Borings</td>
</tr>
<tr>
<td>Lab #:</td>
<td>3816</td>
</tr>
<tr>
<td>Depth:</td>
<td>18.5'-20.0'</td>
</tr>
</tbody>
</table>

## Sample Description:
Gray Poorly Graded Sand with Silt (SP-SM)

## Particle Size Analysis

<table>
<thead>
<tr>
<th>Particle Size Analysis / Without Hydrometer Analysis</th>
<th>Material Excluded:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tare No.</td>
<td>O</td>
</tr>
<tr>
<td>Total Sample Wet Wt. + Tare Wt.</td>
<td>159.6</td>
</tr>
<tr>
<td>Total Sample Dry Wt. + Tare Wt.</td>
<td>139.2</td>
</tr>
<tr>
<td>Total Sample Dry Weight</td>
<td>56.7</td>
</tr>
</tbody>
</table>

## Sieve Analysis

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Retained Weight</th>
<th>% Retained Between Sieves</th>
<th>% Retained Individual</th>
<th>% Passing Total Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td>mm.</td>
<td>Cumulative</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0&quot;</td>
<td>50.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>37.50</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1.0&quot;</td>
<td>25.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>19.00</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>12.50</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.50</td>
<td>0.0</td>
<td>0.0%</td>
<td>0.0%</td>
</tr>
<tr>
<td>#4</td>
<td>4.75</td>
<td>2.9</td>
<td>5.1%</td>
<td>5.1%</td>
</tr>
<tr>
<td>#20</td>
<td>0.850</td>
<td>9.7</td>
<td>12.0%</td>
<td>17.1%</td>
</tr>
<tr>
<td>#30</td>
<td>0.600</td>
<td>26.6</td>
<td>29.8%</td>
<td>46.9%</td>
</tr>
<tr>
<td>#40</td>
<td>0.425</td>
<td>32.3</td>
<td>44.1%</td>
<td>57.0%</td>
</tr>
<tr>
<td>#60</td>
<td>0.250</td>
<td>40.3</td>
<td>44.1%</td>
<td>57.0%</td>
</tr>
<tr>
<td>#100</td>
<td>0.150</td>
<td>51.2</td>
<td>19.2%</td>
<td>90.3%</td>
</tr>
<tr>
<td>#200</td>
<td>0.075</td>
<td>53.0</td>
<td>3.2%</td>
<td>93.5%</td>
</tr>
<tr>
<td>Pan</td>
<td>&lt;0.075</td>
<td>53.0</td>
<td>% Passing #200 (D1140) = 6.5%</td>
<td></td>
</tr>
</tbody>
</table>

## Notes / Deviations / References:

Sieves #4 and #20 had 100% shells retained.

---

**Chelsea Jones**

Technical Responsibility

**Staff Professional**

Signature

Position

Date: 4/25/16

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Appendix IV

L-Pile Results
Bending Moment (in-kips)

14" x 14" PSPC Pile - Static Case

Depth (ft)

-1800  -1600  -1400  -1200  -1000  -800  -600  -400  -200   0    200   400   600   800   1000   1200

0  2  4  6  8  10  12  14  16  18  20  22  24  26  28  30  32  34  36  38

26.4 kips
37.6 kips
44.5 kips
47.6 kips
14" x 14" PSPC Pile - Seismic Case

Lateral Deflection (inches)

Depth (ft)

-0.1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38

21.1 kips
30.3 kips
36.3 kips
38.3 kips
16" x 16" PSPC Pile - Static Case

Lateral Deflection (inches)

Depth (ft)

-0.1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36

34.3 kips
48.7 kips
58.4 kips
62.6 kips
16" x 16" PSPC Pile - Seismic Case

Lateral Deflection (inches)

Depth (ft)

-0.1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36

25.0 kips
35.3 kips
43.1 kips
45.6 kips
16" x 16" PSPC Pile - Seismic Case

Bending Moment (in-kips)

Depth (ft)

-2200 -2000 -1800 -1600 -1400 -1200 -1000 -800 -600 -400 -200 0 200 400 600

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38

25.0 kips
35.3 kips
43.1 kips
45.6 kips
16" x 16" PSPC Pile - Seismic Case

Shear Force (kips)

Depth (ft)

-10 -5 0 5 10 15 20 25 30 35 40 45 50

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38

25.0 kips
35.3 kips
43.1 kips
45.6 kips
Appendix V

Site-Specific Response Analysis
Problem

Determine the site-specific spectral accelerations in accordance with the IBC2012/ASCE7-10. The design spectral acceleration parameters, $S_{DS}$ and $S_{D1}$, are to be taken from a site-specific acceleration response spectrum (ARS) computed at the ground surface. The analysis will also be used to determine PGA$_{M}$, which can be used for liquefaction triggering evaluation and seismic stability evaluation.

Overview

The ARS was developed using the equivalent-linear, one-dimensional computer program STRATA. STRATA uses the soil column described herein, a base rock acceleration time history, and dynamic soil properties to model the passage of seismic waves through the site. The measured shear wave velocity profiles were used to generate our basic site response model. The dynamic soil properties were modeled using published modulus versus strain curves and damping versus strain curves (i.e., modulus reduction and damping curves). The acceleration time histories were generated using the USGS Seismic Hazard website (http://geohazards.usgs.gov/deaggint/2002/). In an effort to account for the uncertainty in soil properties and input motion depth, we performed multiple iterations using different velocity profiles, modulus reduction and damping curves, depths of the input base rock motion, and acceleration time histories. The resulting output was averaged to create a single response spectrum, which was then modified in general accordance with Chapter 21 of ASCE 7-10 to develop the design ARS.

Site Characterization

Area Geology

The site lies within the Coastal Terraces Region of the Lower Coastal Plain of South Carolina. These sediments overlie crystalline (metamorphic) bedrock, which is presumed to occur at a depth of approximately 1,100 feet in the site area.

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

Materials comprising the terraces typically consist of a strand or beach ridge deposit of clean sands at the seaward margin. Between the strand and the toe of the next inland terrace are mainly finely interlayered clays and sands termed backbarrier deposits.

Over wide areas in Horry County seams of poorly consolidated silts or clays occur near the base of the terrace sediments. These sediments were weathered or eroded from the underlying materials and redeposited a short distance away in a low-energy environment. Erosion and redeposition of these soils largely leached out the calcareous cement binding the soil grains of the intact soils together, and the redeposited soils have generally not been subjected to confining stresses significantly in excess of their own weight. Under these conditions, the in-place soils often exhibit little strength and are highly...
compressible. Such very soft clays were observed in the soundings and borings at this site at depths of between 13 feet and about 17 to 18 feet.

Shell-laden sands or Coquina often occur below the soft clays in beds ranging up to 10 feet in thickness. These soils consist mostly of loose to medium dense, fine to medium sized sands with some silt and occasional phosphate nodules. Recovered samples are often highly structured but weakly cemented masses of shell. Samples typically react vigorously to dilute HCl reflecting the calcium carbonate based natural cement between the particles, but penetration values are often low due to disruption of the soil structure by the sampling tool.

The Pee Dee Formation underlies these shell sands, and consists of a thick, massively bedded, dark gray to dark grayish-green, micaceous calcareous clay-sand or sand-clay. Ledges of thin limestone are often encountered on top of the Pee Dee Formation and again about every 6 to 8 feet, and may range in thickness from 6 inches to 4 feet. The Pee Dee Formation is estimated to be late Cretaceous age. Two water wells at the nearby Dolph Grainger generating station in Conway, South Carolina, obtained from SCDNR, imply the base of the Pee Dee occurs at approximately 125 feet to 150 feet below the surface in the general area of the site. Our borings on this site did not fully penetrate the Pee Dee Formation.

The Black Creek Formation (or Group) underlies the Pee Dee Formation and is approximately 500 feet in thickness. The Black Creek Group consists of interlayered cross-bedded sands, clays and sandy clays deposited in a mostly terrestrial environment. Soils are generally well consolidated. Calcareous deposits indicating marine deposition are rare. The Middendorf Formation forms the lowermost Coastal Plain strata in the site area, extending to metamorphic bedrock at around 1,100 feet. Soils consist mostly of interlayered kaolinitic sands, clays and silts interspersed with coarse sands or sandstone inclusions.

Subsurface Conditions

The subsurface exploration encountered surficial terrace deposits consisting of very loose to medium dense poorly graded sands with silt overlying a very soft sandy lean clay and fat clay layer ranging from a depth of approximately 13 to 18 feet. Underlying the clay layer is approximately 10 feet of very loose to medium dense poorly graded sands with varying amounts of silt and clay. The Pee Dee Formation was encountered at an average depth of approximately 30 feet beneath the ground surface and consisted of sandy lean clay, sandy fat clay, and elastic silt with sand. The CPT soundings were terminated soon after encountering the top of the cemented Pee Dee layer at depths ranging from 31 to 34 feet. The soil test borings were advanced to the planned termination depths of 50 to 55 feet.

The two MASW arrays performed at the site measured shear wave velocities to depths ranging from approximately 130 to 200 feet beneath the ground surface. The measured average shear wave velocities within the upper 100 feet of the two MASW arrays were approximately 920 feet per second (fps). The measured shear wave velocity profiles from the two MASW arrays are included in Figure A, along with the model profile which was used in the analysis. According to the SCDNR water wells, the Pee Dee Formation extends to a depth of approximately 125 to 150 feet in the project area. Therefore the MASW measurements extend into the Black Creek Formation and terminate with a shear wave velocity of approximately 1,400 fps at a depth of approximately 200 feet. Our soil model considers a profile with the base rock motion input at a point in the soil profile at which the shear wave velocity exceeds 2,500 fps.
(Site Class B/C boundary). In this analysis we have assumed this occurs at a depth of approximately 250 feet, within the Black Creek Formation.

Stabilized groundwater levels at the site were measured to be approximately 5 feet beneath the existing ground surface.

Figure A – Shear Wave Velocity Profiles
Ground Motions

Site Hazard

The earthquake hazard associated with this site was deaggregated using the 2002 update of the USGS Interactive Deaggregation website to determine the modal magnitude and site-to-source distance of the earthquake contributing to the peak surface acceleration at frequencies of 5 Hz and 1 Hz. Considering a 2 percent probability of exceedance (2% PE) in 50 years, the modal magnitudes and site-to-source distances are approximately M=7.36 and 66 km at both the 5 Hz and 1 Hz frequencies.

Ground Motions

Using the USGS Interactive Deaggregation Website (2002 update), synthetic seismograms were generated for the site latitude and longitude. Considering a Return Period of 2% PE in 50 years and Spectral Acceleration (SA) frequencies of 1 Hz (1 second period) and 5 Hz (0.2 second period), two sets of six seismograms were generated assuming an event equal to the modal magnitude and distance. The synthetic motions were scaled to account for the 2008 update to the USGS motions.

Upon the completion of the analysis, the computed acceleration response spectra was converted from an event considering a 2% PE in 50 years to an event considering a 1% risk of structural collapse in 50 years. Previous to the 2012 edition of the IBC, the Hazard Maps considered an event with 2% PE in 50 years. Per ASCE 7-10, the Hazard Maps in the 2012 IBC have been converted from an event considering a 2% PE in 50 years to an event considering a 1% risk of structural collapse in 50 years.

Methodology

S&ME used the STRATA software package to perform an equivalent-linear, one-dimensional seismic site response analysis for the design event. The required input includes acceleration time histories and soil profiles (layer thicknesses, unit weights, shear wave velocities, modulus reduction curves, and damping curves). A parametric analysis considering different soil models and acceleration time histories was performed to account for the uncertainty in the input parameters and to evaluate the sensitivity of the results to the model assumptions. The parametric analysis included using all 12 of the USGS synthetic acceleration time histories (i.e., 6 from the 1 Hz match and 6 from the 5 Hz match) with variations in the depth to the input motion (i.e., the B-C boundary or top of "rock"), the shear wave velocities, and the selection of damping and shear modulus reduction curves. The parametric analysis considered variations in shear wave velocity of each layer by ± 15% of the measured velocity, variations in the plasticity index of each layer (excluding the surficial sand layer) by ± 15 percentage points, and variations in the depth of the input base rock motion from 250 to 500 feet. Numerous ground surface response spectra (with equivalent viscous damping ratios of 5%) were generated for each group of analyses with the median spectrum used to represent the site-specific response acceleration spectrum.

The spectral acceleration analyses using the 1 Hz ground motions were performed separately from those using the 5 Hz ground motions. The spectra resulting from the 5 Hz ground motions were used within the region ranging from a period of zero to $T_S$ (0.69 seconds), while the spectra resulting from the 1 Hz ground motions were used within the region ranging from $T_S$ to 2 seconds. Figures C and D in the Additional Output section of the package illustrate the variance in the 1 Hz and 5 Hz motions.
The median response spectra from each analytical run, which included all six time histories from each frequency match, were averaged together to compute a single acceleration response spectrum for both 1 Hz and 5 Hz frequency matches. The six USGS base motions for each frequency match were applied to each of the different soil models resulting in 120 different computations of site response, capturing the uncertainty in the input parameters assumed in the base model. An example of the STRATA output is presented in the Additional Output section of this package as Figure E.

As required by Section 1613 of the International Building Code and Chapter 21 of ASCE 7-10, the site-specific ARS has been limited to a minimum value of 80 percent of the general response spectrum for a Seismic Site Class E. The limiting reduction is based on the general procedure ARS per ASCE 7-10 and IBC 2012. The design spectral response for the short period ($S_{DS}$) must also be at least 90 percent of the peak average spectral acceleration at any period and the design spectral response for the long period ($S_{DL}$) must be at least two times the spectral acceleration at the 2.0-second period.

**Models**

A general summary of the soil subsurface information used in STRATA is presented in Table 1. The shear wave velocity profile that was used in the base model is plotted in Figure A. The different iterations of the analysis considered various shear wave velocity profiles that were bound between ± 15% of the base shear wave velocity model. Variance in the two different MASW measurements prompted additional analyses where the shear wave velocity of a single layer was varied independently from all other layers. These additional iterations include increasing the shear wave velocity of the 40 to 67-foot layer from 1,175 fps to 1,250 fps and decreasing the shear wave velocity of the 115 to 165-foot layer from 1,200 fps to 1,150 fps. A third iteration included varying the shear wave velocity of the 200 to 250-foot layer, which is beneath the termination depth of the MASW measurements, from 2,000 fps to 1,800 fps. The shear modulus reduction and damping ratio relationships proposed by Darendelli were used in the analysis and were varied to consider the mean effective stress, plasticity index, and over-consolidation ratio of each layer. With the exception of the surficial sand layer, the plasticity indices of all other layers presented in Table 1 were varied ± 15 percentage points. The input motions were applied at depths of 250, 350, and 500 feet.

---

1 In accordance with IBC 2012 and ASCE 7-10, due to the liquefaction potential, the site is designated as Site Class F. When performing a site specific response analysis for Site Class F conditions, the code requires the resulting spectrum to be compared and limited to the Site Class E general procedure spectrum.
## Table 1 – STRATA Base Model Summary

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Soil Type</th>
<th>Shear Wave Velocity (fps)</th>
<th>Unit Weight (pcf)</th>
<th>Shear Modulus Reduction &amp; Damping Ratio Curves Parameters</th>
<th>Mean Effective Stress (atm)</th>
<th>Plasticity Index</th>
<th>Over-Consolidation Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 5</td>
<td>Terrace Deposits</td>
<td>515</td>
<td>105</td>
<td>0.17</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 – 12</td>
<td>Terrace Deposits</td>
<td>525</td>
<td>105</td>
<td>0.26</td>
<td>30</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>12 – 15</td>
<td>Terrace Deposits</td>
<td>550</td>
<td>105</td>
<td>0.34</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15 – 20</td>
<td>Terrace Deposits</td>
<td>650</td>
<td>110</td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 – 22</td>
<td>Terrace Deposits</td>
<td>750</td>
<td>115</td>
<td>0.39</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22 – 27</td>
<td>Terrace Deposits</td>
<td>825</td>
<td>115</td>
<td>0.47</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>27 – 31</td>
<td>Terrace Deposits</td>
<td>1,000</td>
<td>120</td>
<td>0.60</td>
<td>20</td>
<td>1.5</td>
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<tr>
<td>31 – 35</td>
<td>Pee Dee Formation</td>
<td>1,075</td>
<td>120</td>
<td>0.85</td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35 – 40</td>
<td>Pee Dee Formation</td>
<td>1,125</td>
<td>125</td>
<td>0.98</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40 – 67</td>
<td>Pee Dee Formation</td>
<td>1,175</td>
<td>125</td>
<td>1.69</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>67 – 87</td>
<td>Calcareous clay-sand or sand-clay</td>
<td>1,150</td>
<td>125</td>
<td>2.22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>87 – 100</td>
<td>Calcareous clay-sand or sand-clay</td>
<td>1,175</td>
<td>125</td>
<td>2.56</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 – 115</td>
<td>Black Creek Formation</td>
<td>1,200</td>
<td>125</td>
<td>2.96</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>115 – 165</td>
<td>Black Creek Formation</td>
<td>1,200</td>
<td>125</td>
<td>4.28</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>165 – 190</td>
<td>Black Creek Formation</td>
<td>1,300</td>
<td>130</td>
<td>4.99</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>190 – 200</td>
<td>Black Creek Formation</td>
<td>1,400</td>
<td>130</td>
<td>5.27</td>
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<td></td>
<td></td>
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<tr>
<td>200 – 250</td>
<td>Black Creek Formation</td>
<td>2,000</td>
<td>130</td>
<td>6.70</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>Black Creek Formation</td>
<td>2,500</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>
Results

The code-limited acceleration response spectrum computed at the ground surface is represented by the SSRA design curve shown on Figure B. The design spectral response acceleration parameters $\text{PGA}_M$, $S_{DS}$, and $S_{D1}$ from the general procedure and the site-specific response analysis are presented on Figure B and also in Table 2. Review of these values along with Figure B shows the design spectral response acceleration is governed by the site-specific ARS curve at the short period ($S_{DS}$), while the long period ($S_{D1}$) is controlled by 80 percent of the general procedure ARS. The $\text{PGA}_M$ of the site-specific analysis is taken as the acceleration at the 0.0 second period before the $\frac{2}{3}$ reduction is applied to the acceleration spectrum to obtain the structural design curve. It is also noted that per ASCE 7-10, the $\text{PGA}_M$ of the site-specific analysis considers a hazard with a 2% PE in 50 years while the seismic design parameters $S_{DS}$ and $S_{D1}$ consider an event with a 1% risk of structural collapse in 50 years. Risk coefficients $C_{RS}$ and $C_{R1}$ (both equal to 0.848) were applied to convert the response spectrum from a 2% PE in 50 years to 1% risk of structural collapse in 50 years.

Given the seismic design parameters $S_{DS}$ and $S_{D1}$, the site-specific analysis indicates a **Seismic Design Category D** (Section 1613.3.5 of IBC 2012) should be considered for Risk Categories I-IV.

<table>
<thead>
<tr>
<th>Method</th>
<th>$\text{PGA}_M$</th>
<th>$S_{DS}$</th>
<th>$S_{D1}$</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>IBC 2012 Site Class E</td>
<td>0.36 g</td>
<td>0.57 g</td>
<td>0.40 g</td>
<td>D</td>
</tr>
<tr>
<td>General Procedure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site-Specific ARS</td>
<td>0.41 g</td>
<td>0.67 g</td>
<td>0.32 g</td>
<td>D</td>
</tr>
</tbody>
</table>

**Additional Output**

The following figures and table are presented as additional output:

- Figure B – Design Acceleration Response Spectra
- Figure C – Acceleration Response Spectrum – 5 Hz Motion Parametric Analysis
- Figure D – Acceleration Response Spectrum – 1 Hz Motion Parametric Analysis
- Figure E – STRATA output of Computed Acceleration Response Spectrum of 5 Hz Motions
- Figure F – 5 Hz Deaggregation
- Figure G – 1 Hz Deaggregation
- Figure H – 5 Hz Accelerograms
- Figure I – 1 Hz Accelerograms
Figure B – Design Acceleration Response Spectra
5% Damping, 1% Risk of Structural Collapse in 50 Years
Figure C – Acceleration Response Spectrum - 5 Hz Motion Parametric Analysis
Maximum Considered Earthquake – 5 % Damping – 2% Probability of Exceedance in 50 Years
Figure D – Acceleration Response Spectrum – 1 Hz Motion Parametric Analysis
Maximum Considered Earthquake – 5 % Damping – 2% Probability of Exceedance in 50 Years
Figure E – STRATA Output of Computed Acceleration Response Spectrum of 5 Hz Motions
Prob. Seismic Hazard Deaggregation
CCU_Stadium_Reno  79.018° W, 33.793 N.
SA period 0.20 sec. Accel.>=0.6589 g
Mean Return Time of GM 2475 yrs
Mean (R,M,\varepsilon) 56.3 km, 7.19, 0.15
Modal (R,M,\varepsilon) = 65.7 km, 7.36, 0.60 (from peak R,M bin)
Modal (R,M,g\textsuperscript{\textcircled{g}}) = 86.8 km, 7.37, 1 to 2 sigma (from peak R,M,g bin)
Binning: \Delta R=25. km, \Delta M=0.2, \Delta \varepsilon=1.0

Figure F – 5 Hz Deaggregation
Prob. Seismic Hazard Deaggregation
CCU_Stadium_Reno 79.018° W, 33.793 N.
SA period 1.00 sec. Accel>=0.1840 g
Mean Return Time of GM 2475 yrs
Mean (R,M,e0) 65.1 km, 7.24, 0.27
Modal (R,M,e0) = 66.0 km, 7.36, 0.53 (from peak R,M bin)
Modal (R,M,e#) = 86.9 km, 7.37, 1 to 2 sigma (from peak R,M,e bin)
Binning: DeltaR=25. km, deltaM=0.2, Deltae=1.0

Figure G – 1 Hz Deaggregation
Calculation Package: Site-Specific Response Analysis

Coastal Carolina University - Brooks Stadium Renovation
Conway, South Carolina
April 13, 2016

Figure H – 5 Hz Accelerograms
Figure I – 1 Hz Accelerograms
References

Brooks Stadium Expansion and Renovation
Supplemental Report of Alternative
Foundation Support Types
Conway, South Carolina
S&ME Project No. 1463-16-012
December 19, 2016

Coastal Carolina University
Post Office Box 261954
Conway, South Carolina 29528

Attention: Mr. Mark Avant

Reference: Brooks Stadium Expansion and Renovations
Supplemental Report of Alternative Foundation Support Types
Conway, South Carolina
S&ME Project No. 1463-16-012

Dear Mr. Avant:

S&ME, Inc. has completed this supplemental report for the referenced project after receiving your emailed authorization to proceed on December 12, 2016. This work was conducted in general accordance with our Proposal for Additional Geotechnical Analysis, dated December 9, 2016.

The purpose of this supplemental report is to provide a drilled pile alternative to the driven piles that were recommended in the original geotechnical report. This information was requested by Jimmy Jones, PE, of ADC Engineering on December 9, 2016.

This report provides our geotechnical conclusions and recommendations regarding this foundation alternative. 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.

Chelsea Jones
Staff Professional

Joshua D. Jordan, P.E.
Project Engineer

Ronald P. Forest, Jr., P.E.
Senior Engineer
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Appendices

Appendix I
1.0 Limitation of Applicability

This is not a comprehensive, stand-alone geotechnical report. This report relies upon the original Report of Geotechnical Exploration dated April 25, 2016, S&ME Project No. 1463-16-012.

Therefore, this supplemental report should not be considered without reviewing the information provided in the primary (original) geotechnical report.

2.0 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 original exploration. If the construction scope is altered, the proposed structure location is changed, or if conditions are encountered during construction that differ from those encountered, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

Our geotechnical exploration indicates that the site is adaptable for the proposed construction. However, shallow foundations do not appear feasible for support of the stadium structure due to the magnitude of the anticipated structural loads compared to the strength and compressibility of the soils, and because of the potential for up to 5 inches of liquefaction-related settlement during an earthquake for structures supported on shallow foundations at the surface. Further challenges may be presented by coordinating large, spread footings in the vicinity of existing stadium foundations.

The original geotechnical report recommended the use of driven, pre-cast, pre-stressed concrete (PSC) piles to support the stadium, because PSC piles have been used successfully on this same project site in the past, and their performance aspects were well documented, including prior load testing by S&ME, Inc. with Pile Driving Analyzer (PDA) test equipment. However, we understand from our conversations with Jimmy Jones, PE, of ADC Engineering on December 9, 2016, that the process of casting and curing the PSC piles may be too slow to accommodate the aggressive project schedule, and therefore a drilled pile alternative was requested.

S&ME has recommended drilled, cast-in-place piles for Coastal Carolina University in the past, and such piles have been utilized with success at various places on campus. For example, in 2010, foundations consisting of 16-inch diameter, 50-ft. long auger cast piles were used at the nearby Convocation and Recreation Center project. One of those piles was static load tested and shown to be capable of carrying up to 90 tons of working load with an acceptable factor of safety (S&ME Project No. 1633-10-105). Another example, in 2011, foundations consisting of 16-inch diameter, 37-ft. long auger cast piles were used at the nearby Academic Office and Classroom Building project. One of those piles was static load tested and shown to be capable of carrying up to 70 tons of working load with an acceptable factor of safety (S&ME Project No. 1633-11-261). The axial capacity of the piles at the Brooks Stadium site may not be quite as high as those at the Academic Offices, because the soils at the anticipated bearing depth are somewhat weaker, but somewhat lower capacity would be acceptable according to Mr. Jones.

Mr. Jones said in our conversation on December 9, 2016, that he does not need for us to increase the capacity of the piles above the values provided in our original geotechnical report for Brooks Stadium,
which provided both 40 ton and 50 ton options for the PSC piles, depending upon size. Therefore, this report analyzed and considered recommendations for augered, cast-in-place piles having a similar range of axial capacities as the PSC piles that we previously recommended. The minimum pile diameter considered was 14 inches, as discussed with Mr. Jones; however, we also considered 16-inch diameter piles because lateral reinforcement and deflection can sometimes govern the design of smaller diameter augered piles.

The following sections present our geotechnical recommendations regarding a drilled pile alternative.

2.1 Augered Pressure Grouted Displacement (APG-D) Piles

APG-D piles are recommended for foundation support at this site. APG-D piles are relatively economical to install and have a comparatively high available axial capacity in the soil conditions observed. This pile type appears to be feasible to install at this site, although significant consideration will need to be given to crane access issues around the existing structure. Some constructability issues for this foundation type are discussed later in this report.

APG-D piles are different from traditional auger cast-in-place piles in the following way; traditional piles are drilled to the specified depth while exporting the spoils up the auger flights to the ground surface, whereas APG-D piles push most of the soils down and to the sides of the shaft during drilling. The traditional drilling method of evacuating the soils from the pile shaft results in significant surface spoils, and for piles advanced below the water table (as would be the case at this site), a significant amount of water is also typically transported to the surface. On an undeveloped site, these issues typically don’t present a problem, but on a developed site the results can create poor, messy conditions on the ground surface.

Also, since APG-D piles push the soils to the sides and bottom of the shaft during drilling, they have a densifying effect on the subsurface which can result in increased frictional capacities for the piles, whereas the evacuation of soil that occurs during traditional auger cast pile installation can undermine existing foundations if there are any are located near the pile. For these reasons, APG-D piles are recommended over traditional auger cast-in-place piles on this project.

It is advisable for the ratio of the pile length not to exceed about 40 to 45 times the pile diameter. Since these piles must extend into the Pee Dee Formation at depths of about 37 feet below the ground surface in order to bear a sufficient distance into the bearing stratum, either 14 or 16 inches would be acceptable pile diameters. (18 inches would also be an acceptable pile diameter, but this size is not anticipated to be needed based upon the loading information provided to us by Mr. Jones).

2.1.1 APG-D Axial and Uplift Capacities

Axial capacities versus depth were estimated for individual 14-inch and 16-inch diameter APG-D piles based upon the subsurface conditions encountered in the borings and soundings.

The soil profile was generally modeled based upon the subsurface conditions observed in test sounding CPT-4 to a depth of 37 feet. The estimated axial capacities are summarized in Table 2-1 below.
Table 2-1: Single APG-D Vertical Capacities

<table>
<thead>
<tr>
<th>Modeled APG-D Diameter (inches)</th>
<th>Pile Embedment Depth (feet)</th>
<th>Allowable Axial Capacity per pile (tons)</th>
<th>Allowable Uplift Capacity per pile (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>With Axial load test*</td>
<td>Without Axial load test**</td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Pullout test*</td>
<td>Without Pullout test**</td>
</tr>
<tr>
<td>14-inch</td>
<td>37</td>
<td>37</td>
<td>24</td>
</tr>
<tr>
<td>16-inch</td>
<td>37</td>
<td>50</td>
<td>34</td>
</tr>
</tbody>
</table>

*Allowable capacity assumes a factor of safety of 2 applied to the estimated ultimate capacity.

**Allowable capacity assumes a factor of safety of 3 applied to the estimated ultimate capacity.

The soil coefficients used in our axial capacity analyses were developed using published correlations relating soil skin friction and end bearing unit capacities to SPT N-value and CPT tip stresses measured. Soils in the upper five feet of the soil profile are assumed to be in contact with the pile cap, not the individual piles, and were considered not to contribute to individual pile axial resistance.

To use the higher of the axial capacity values provided in Table 2-1 and meet IBC requirements, the single pile axial capacity must be verified at the start of construction by performing at least one static load test, ideally to failure, or to at least two and one-half times the design load, using the “quick load test method” of ASTM D 1143 – “Standard Method of Testing Piles Under Static Axial Compressive Load”. The static load test should be performed under the observation of the Geotechnical Engineer.

If the lower (more conservative) pile capacity values shown in Table 2-1 suffice for the structural design, then the axial static load test may be omitted, particularly since we have other test data on nearby sites. This may save a week or two in the project construction schedule, but could also result in more piles. The cost tradeoff between doing the load test and the extra piles needed because of the lower pile capacity can be value engineered once a preliminary pile layout pattern has been determined.

For piles under critical uplift loading, pullout testing should be performed in accordance with ASTM D 3689 - “Standard Test Methods for Deep Foundations under Static Axial Tensile Load.” In this case, we recommend that at least one static uplift pullout test be performed by the contractor on one pile that is constructed to these dimensions (Procedure A “Quick Test” is acceptable), loaded to at least twice the design working uplift single pile capacity, to confirm that the assumed ultimate design strength is available.

The static load tests are typically setup and performed by the pile contractor using their equipment, and observed by a representative of the Geotechnical Engineer.

2.1.2 Difficult Drilling Conditions and Auger Refusal

Our cone penetration test (CPT) soundings each refused to advance beyond depths of 31 to 34 feet under the maximum available down pressure applied by the drill rig. Our standard penetration test (SPT) soil test borings were able to be advanced beyond these depths using a tri-cone roller bit, but they also encountered a lens of cemented materials at the top of the Pee Dee Formation within this same general depth range. This very hard zone may be referred to as the “cap rock” of the Pee Dee Formation. It is not technically rock, but it is a very hard cemented marine layer of variable thickness and density. Beneath the “cap rock”, the Pee Dee Formation soils transition into stiff high plasticity clays and silts. If an insufficiently powerful rig is used to install the APG-D piles, then it is likely that the piles may encounter early refusal above the desired penetration depth. Ideally, these piles need to be socketed at least several feet into the
Pee Dee Formation in order to provide fixity at the tip and to provide optimum lateral deflection resistance and axial capacity. Therefore, the next section of this report recommends some minimum rig installation requirements which should be specified as part of the bid requirements.

The auger refusal criterion is recommended to be defined as a less than 1 inch per minute for at least 10 minutes at the full down-crowd pressure of the specified equipment. It is important that the pile installer does not stop trying to advance the pile at the first encounter of a hard lens.

2.1.3 Installation Rig Minimum Requirements

In order to advance the piles into the dense soils to reach the desired termination depths, a strong rotary turntable is going to be required, as well as sharp reinforced cutting bits. We consulted with Berkel & Company Contractors, Inc., which is a specialty contractor in this industry, regarding the degree of torque and rig weight that would likely be required to advance APG-D piles to these depths under these conditions. Based upon that consultation, we recommend that the installation rig have a minimum ram weight of 10,000 pounds and a minimum installation torque of 70,000 ft-lbs. This may require a large rig, so access to the pile cap locations needs to be considered. The bidding pile contractors should visit the site to make sure that they have a suitable crane that can reach the installation locations considering any access restrictions that may exist.

We anticipate that a drilled pile that is advanced using reinforced cutting teeth should be able to advance to the desired penetration depth, since our drill rig was able to advance through these materials with a tricone roller bit. However, slow augering should be expected to occur within the bearing stratum (below 30 feet), and the contractor should be prepared to spend extra time advancing the piles by grinding into these materials.

2.1.4 APG-D Capacity Reductions and Group Effects, Group Uplift

The actual capacity for each pile and each group of piles will be somewhat dependent upon the final pile layout configuration that is selected. Group effects should be checked once the actual final pile configuration is known. The actual pile layout configuration should be determined by the structural design engineer. We recommend that the individual piles have a center-to-center spacing of not less than 3 pile diameters (4 feet for the 16-inch piles).

Under 2015 IBC Section 1810.3.3.1.6, the maximum uplift of a column supported by a pile group would be limited by the lesser of (1) the individual uplift working load times the number of elements in the group, and (2) two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block. Pile groups used on this project should be checked for group uplift capacity.

2.1.5 Lateral Pile Reactions for Assumed Loads

Our lateral pile analyses were performed using the computer program LPILE Plus©. This program performs a beam-column analysis of single piles, which are subjected to given lateral and axial loading, and assumes a non-linear soil response.
Individual 14 inch APG-Ds reinforced with four, #6 reinforcing bars positioned vertically and in a square pattern were analyzed, and individual 16 inch diameter APG-Ds, reinforced with four, #8 reinforcing bars positioned vertically and in a square pattern were analyzed. A vertical load equivalent to the allowable axial compressive capacity was applied to each modeled auger cast pile based upon the axial load values shown in Table 2-1 above. Loading options considered and reported include those with an axial load test, and those without an axial load test.

Lateral loads ranging from approximately 16.9 to 42 kips were applied at the pile head to evaluate the resulting lateral deflection and bending moment at the pile head along the pile. The single pile analysis modeled fixed head restraint conditions with a constant elastic modulus (i.e., no reduced stiffness to account for non-linear bending stiffness). Both static and seismic (liquefied soil) loading conditions were considered. No adjustment was made to the p-y curves to reflect group action. The lateral deflection versus depth curves, moment versus depth curves, shear force versus depth curves, pile-head deflection versus lateral load curves, and lateral load versus maximum bending moment curves output from the program are attached to this report in the appendix.

The lateral load that can be withstood by a typical pile will be limited by the maximum allowable shear stress for the pile material and the radius of curvature introduced by bending. For purpose of preliminary assessment of the auger cast pile sections described herein, lateral deflections at the pile heads were computed for applied lateral loading and applied moments and are provided in Tables 2-2 through 2-6.

### Table 2-2: Fixed Head, Static Load Condition, 14 inch dia., 37 ft. length

<table>
<thead>
<tr>
<th>Applied Axial Load (kips)</th>
<th>Embedment Depth (feet)</th>
<th>Deflection (inches)</th>
<th>Static Lateral Load (kips)</th>
<th>Minimum/Maximum Bending Moments (in-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>74</td>
<td>37</td>
<td>¼</td>
<td>18.0</td>
<td>-578</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>½</td>
<td>24.3</td>
<td>-793</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>¾</td>
<td>27.9</td>
<td>-883</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>1</td>
<td>29.6</td>
<td>-900</td>
</tr>
</tbody>
</table>

### Table 2-3: Fixed Head, Earthquake Load Condition, 14 inch dia., 37 ft. length

<table>
<thead>
<tr>
<th>Applied Axial Load (kips)</th>
<th>Embedment Depth (feet)</th>
<th>Deflection (inches)</th>
<th>Static Lateral Load (kips)</th>
<th>Minimum/Maximum Bending Moments (in-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>74</td>
<td>37</td>
<td>¼</td>
<td>16.9</td>
<td>-534</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>½</td>
<td>23.4</td>
<td>-750</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>¾</td>
<td>27.1</td>
<td>-853</td>
</tr>
<tr>
<td>74</td>
<td>37</td>
<td>1</td>
<td>29.0</td>
<td>-897</td>
</tr>
</tbody>
</table>
Table 2-4: Fixed Head, Static Load Condition, 16 inch dia., 37 ft. length

<table>
<thead>
<tr>
<th>Applied Axial Load (kips)</th>
<th>Embedment Depth (feet)</th>
<th>Deflection (inches)</th>
<th>Static Lateral Load (kips)</th>
<th>Minimum/Maximum Bending Moments (in-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>37</td>
<td>¼</td>
<td>24.6</td>
<td>-904</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>½</td>
<td>34.1</td>
<td>-1299</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>¾</td>
<td>39.7</td>
<td>-1468</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>1</td>
<td>42.7</td>
<td>-1518</td>
</tr>
</tbody>
</table>

Table 2-5: Fixed Head, Earthquake Load Condition, 16 inch dia., 37 ft. length

<table>
<thead>
<tr>
<th>Applied Axial Load (kips)</th>
<th>Embedment Depth (feet)</th>
<th>Deflection (inches)</th>
<th>Static Lateral Load (kips)</th>
<th>Minimum/Maximum Bending Moments (in-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>37</td>
<td>¼</td>
<td>21.2</td>
<td>-771</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>½</td>
<td>29.5</td>
<td>-1078</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>¾</td>
<td>35.9</td>
<td>-1329</td>
</tr>
<tr>
<td>100</td>
<td>37</td>
<td>1</td>
<td>39.0</td>
<td>-1448</td>
</tr>
</tbody>
</table>

Depth to essential fixity of a 14-inch diameter auger displacement cast pile under fixed head conditions appears be about 22 feet under static loading and about 30 feet under earthquake loading. Depth to essential fixity of a 16-inch diameter auger displacement cast pile under fixed head conditions appears be about 24 feet under static loading and about 31 feet under earthquake loading. Point of fixity is defined as the second point of zero deflection of the pile under the applied lateral shear force. Beyond this depth pile length does not influence lateral resistance.

The structural integrity of the APG-Ds has not been considered in this report, and proper steel reinforcement of the piles will need to be designed by the structural engineer for each support situation. We have not performed a structural analysis of the proposed pile. Since we performed our analysis using a constant elastic modulus for the pile, which in reality has a non-linear modulus, the moment capacity of the pile should be checked to verify that the pile is not cracking. We note that beyond a deflection of about 0.5 in. the constant modulus assumption may underestimate the deflection since the actual stiffness will likely be less than that estimated by a constant modulus.

2.1.6 Settlement of Augered Displacement Piles and Pile Groups

Pile settlement consists of two components: axial compression of the piles themselves (termed “elastic shortening”), and consolidation settlement of the piles due to deformation within the soil column. The side friction of a single auger cast pile is typically fully-mobilized at vertical displacements of 0.1 to 1.0 percent of the pile diameter in cohesionless soil, taking into account the elastic shortening of the pile itself (Reese & O’Neill, 1988). For a single 14-inch to 16-inch diameter pile, this would typically equate to about ¼ inch of vertical displacement associated with elastic shortening. Considering consolidation of the bearing soils to be represented by an average elastic modulus of about 1,000 ksf, total settlement of a
single pile is estimated to be roughly \( \frac{3}{4} \) to \( \frac{1}{2} \) inch. To this would be added the elastic shortening of the individual piles as described above of less than \( \frac{1}{4} \) inch, resulting in a total settlement magnitudes of \( \frac{1}{2} \) to \( \frac{3}{4} \) inches.

Settlement of pile groups may be somewhat greater than for individual piles. To check group settlement requires that the size of the pile group, number and spacing of piles, and axial load on the group be known. We may be contracted to estimate the total group settlements as well as check the differential settlement between adjacent dissimilar groups (if applicable) once the actual pile loads and the configurations of the pile groups have been finally determined.

### 2.1.7 Augered Displacement Pile Construction and Testing Protocol

The following tests and procedures are recommended for the test piles and production piles:

1. A minimum of one index (or “test”) pile should be installed at a location chosen by the structural design engineer prior to production pile installation. The index pile installation should be observed by the Geotechnical Engineer or his representative.
2. The installation equipment used to install the index (test) pile should be the same as the equipment to be used in production.
3. Following installation, index piles may be abandoned or used in production pile caps as desired. If used as production piles, the reinforcing cage should match the design requirements.
4. At least one axial compressive load test should be performed. The purpose of the axial compressive load testing is to confirm that the estimated capacity of the piles is in fact available. The test(s) should be performed in accordance with ASTM D 1143 using the hydraulic jack loading procedure\(^1\).

   A. The testing should be performed by the pile installation contractor and under the observation of the Geotechnical Engineer (S&ME). At each location, the test pile and associated reaction piles should be constructed to the diameter and depths of the production piles specified for that area. A group of four reaction piles, each equally spaced at least 5 to 6 pile diameters away from the test pile, is anticipated to provide sufficient uplift frictional capacity to obtain the desired force against the test pile.

   B. During axial compressive testing, the test pile should initially be loaded to 2.0 times the single pile allowable design (working load) capacity, then unloaded to zero, then reloaded to at least 2.5 times the single-pile allowable design capacity. (Note: It is desirable to reload the piles to 3.0 times the single pile capacity during the reload cycle if the load frame and jack equipment can accommodate it.)

   C. If twice the allowable pile capacity is achieved for the test pile using acceptable deflection-limited failure criteria, then the allowable working design capacities may be considered verified. If less than twice the allowable pile capacity is achieved, then the Geotechnical

\(^1\) If uplift controls the design, the structural engineer may decide that it is more appropriate to perform an uplift (tension) pullout static load test in addition to, or in lieu of, the axial download test. Uplift testing should be performed in accordance with ASTM D 3689 - “Standard Test Methods for Deep Foundations Under Static Axial Tensile Load.”
Engineer should be consulted to re-evaluate the pile design capacities based upon the test pile results. Modifications to the pile layout design may be required in this case.

5. Full-time observation of production piles by a Foundation Special Inspector is required; therefore, we recommend that S&ME, Inc. be retained to observe all production pile installation and perform testing as specified.

6. Minimum grout strength of 4,000 psi is recommended for construction of the auger cast piles. Grout properties are critical in installing piles that will perform satisfactorily. The grout should include additives that will adequately control setting shrinkage. The grout must be fluid enough to be pumped easily and must flow without excessive pressure losses.

   A. One set of 6 grout cube samples should be cast by S&ME, Inc. personnel per every 30 cubic yards of grout delivered to the site, or at least twice per day of production.
   B. Grout pressure should be observed and documented during pumping.

7. A sufficient volume of grout should be continuously pumped under sufficient head to prevent suction from developing as the augers are withdrawn from the borehole. Suction could cause the soil to mix with the grout, loss of bearing capacity, or hole collapse. A head of at least 10 feet of grout above the injection point should be maintained at all times to help prevent collapse of the pile.

8. Auger withdrawal rate should not exceed 10 feet per minute. Sudden pulls of the auger, which may cause “necking” or collapse of the hole should be avoided.

9. Pile reinforcing may consist of bundled steel rods, rolled steel sections, or reinforcing bar cages as determined by the structural engineer. All reinforcing should be installed before the grout sets up, normally within 10 minutes of auger withdrawal. Center the reinforcing steel in the hole with centering devices.

10. Equipment for controlling and measuring the flow rate of grout should be calibrated before the commencement of construction. The pump calibration curve of stroke vs. volume should be provided to the S&ME, Inc. testing representative on-site, in order to facilitate volumetric calculations.

   A. The volume of grout pumped into each pile should be recorded and compared to the theoretical volume of pile by the testing representative.
   B. Where the ratio of actual volume to theoretical volume is less than 1.15 for APG-Ds, the pile will need to be re-drilled unless otherwise directed by the Geotechnical Engineer.

11. Have the Geotechnical Engineer observe each cleaned pile cap excavation prior to concrete placement. Also, have the Geotechnical Engineer observe any undercut areas in pile cap excavations prior to backfilling, in order to confirm that the poor soils have been removed and that the exposed subgrade is suitable for support of foundations.

12. We recommend that at least one set of four ASTM C 31 cylinder specimens be cast by S&ME per every 50 cubic yards of structural concrete placed as pile caps or mats, in order to measure achievement of the design compressive strength. We also recommend that S&ME be present on-site to observe all concrete placements.
3.0 Limitations of Report

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

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our boring and sounding locations may not become evident until construction. If variations appear evident, then we should be provided a reasonable opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.
Appendices
Appendix I

L-Pile Results
14-inch diameter APG-D Piles, Static Loading

Pile-head Deflection (in)

Lateral Load (kips)

Case 1
14-inch diameter APG-D Piles, Static Loading

Shear Force (kips)

Depth (ft)
14-inch diameter APG-D Piles, Dynamic Loading

![Graph showing the relationship between Pile-head Deflection (in) and Lateral Load (kips).](image)
16-inch diameter APG-D Piles, Static Loading
16-inch diameter APG-D Piles, Static Loading

Lateral Deflection (inches)

Depth (ft)

-0.1 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1

24.6 kips
34.1 kips
39.7 kips
42.7 kips
16-inch diameter APG-D Piles, Dynamic Loading
Bending Moment (in-kips)

Depth (ft)

-1400 -1200 -1000 -800 -600 -400 -200 0 200 400

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38

21.2 kips
29.5 kips
35.9 kips
39.0 kips
Shear Force (kips)

Depth (ft)

16-inch diameter APG-D Piles, Dynamic Loading

21.2 kips
29.5 kips
35.9 kips
39.0 kips