Florence County Government

Procurement Department

January 15, 2016

ADDENDUM NO.1- BUILDERS RISK INSURANCE FOR THE NEW JUDICIAL CENTER (BID NO. 24-15/16)

Florence County is sending to all interested firms clarification information and answers to frequently asked questions (FAQs) concerning this invitation to bid. The answers are highlighted in RED and underlined.

1. Has a Geotech Report been completed? **The Geotech Report and the Geotech Supplemental Report are attached.**

2. Is there a site plan? **The C3.00 – site plan is attached.**

3. What is the project timeline? **Preliminary detailed project schedule is attached. Please note that this schedule is subject to change.**

4. What is the construction budget? **Please see page 4 of the bid document.**

5. What is the description of security for the project? **BE&K Building Group will have chain link fencing around the entire site during the construction phase of the project. Any time work is being performed on the site, BE&K will have a responsible person at the site to manage the work. When no work is being performed on site, the site fencing will be closed and padlocked.**

6. What type of construction materials will be used for the support framing, exterior walls, floors, roof supports and roof decking? **Refer to attached structural design General Notes sheet S0.01.**

7. What is the name of the general contractor, how many years have they been in business, and did they have had any builders risk losses in the last 3 years? - **BE&K Building Group, LLC, through its successor companies have been in business since 1968, with current management in excess of 20 years. There have been no builder's risk losses in the last 3 years.**

8. Is there a separate limit for soft costs? **Limits for "soft cost" should be set at $1,000,000.00.**

PLEASE ACKNOWLEDGE THIS ADDENDUM BY SIGNING BELOW AND SUBMIT IT WITH YOUR BID.

I have read and acknowledged this addendum for bid no. 24-15/16.

Authorized Signature	Printed Name	Date

Company Name

REVISED REPORT OF
GEOTECHNICAL EXPLORATION

Florence County Judicial Center

Florence, South Carolina
S&ME Project No. 1439-14-021, R1

Prepared For:

Stevens & Wilkinson
1501 Main Street
Columbia, South Carolina 29201

Prepared By:

S&ME

2327 Prosperity Way, Suite 9
Florence, South Carolina 29501

September 30, 2014

September 30, 2014

Stevens & Wilkinson
1501 Main Street
Columbia, South Carolina 29201

Attention: Michelle Motchos, PE, LEED AP BD+C
Senior Associate/Director of Structural Engineering

Reference: Revised Report of Geotechnical Exploration
Florence County Judicial Center
Florence, South Carolina
S&ME Project No. 1439-14-021, R1

Dear Ms. Motchos:

S&ME, Inc. has completed the subsurface exploration for the referenced project after receiving signed authorization to proceed from your firm on August 11, 2014. Our exploration was conducted in general accordance with our Proposal No. 14-1400557, dated July 25, 2014. This revised report includes updated seismic design information not included in our original report, dated September 16, 2014 and supersedes that report.

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, pavement section construction, and foundation support for the proposed structure.

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

Respectfully submitted,
S&ME, Inc.

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 28224 WILLIAM D. KANNON 9/30/14

William D. Kannon, P.E.
Project Engineer
wkannon@smeinc.com

Ronald P. Forest, Jr., P.E.
Senior Project Engineer
rforest@smeinc.com

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION S&ME, INC. No. C00473

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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. **Subsurface Conditions:** At the ground surface or beneath asphalt pavements, a layer of loose undocumented sandy fill with debris was encountered in some locations to depths of about 1 to 3 feet below the ground surface. Beneath this fill stratum, medium dense Coastal Plain upper sands and silty sands (Stratum I) were encountered to a depth of about 2 to 4 ½ feet. Underlying the sands, a layer of generally firm silts and clays interbedded with loose to medium dense silty and clayey sands (Stratum II) was encountered to depths ranging from 17 to 21 feet. Beneath stratum II, a layer of generally soft silts and clays (Stratum III) with thin sand seams was encountered to depths of about 48 to 49 feet. Beneath the silts and clays of Stratum III, the soundings encountered dense sands and silty sands of the Donoho Creek Formation (Stratum IV) to the maximum exploration depth of 55 feet.

2. **Water Level Measurements**: At the time of drilling, the subsurface water level was interpreted to be about 7 to 8 ½ feet below the ground surface based upon pore pressure data measured in the CPT soundings. This may represent perched groundwater trapped within Stratum II, atop the relatively impermeable soils of Stratum III, and may not represent a stable aquifer condition. Perched water levels may vary across the site, due to variations in soil stratigraphy and other factors.

3. **Site Preparation & Surface Stabilization:** The existing buildings, surface slabs, subsurface foundations[1], and existing pavements should be demolished and removed in their entirety from beneath the footprints of the new building and pavements. Underground utilities should be removed or grouted in-place and re-routed. Following demolition, the exposed surface soils within proposed building pad and parking areas should be thoroughly densified at the surface with a heavy vibratory roller prior to new fill placement or the next phase of construction. Excavations made to demolish old foundations should be backfilled with compacted fill. Following surface densification, the subgrade soils should be proofrolled with a loaded tandem-axle dump truck prior to new fill placement. Some overexcavation of soft/loose soils or debris-laden fills should be anticipated.

4. **Seismic Site Class**: Cone sounding data and shear wave velocity field test data indicates that this site is best described as IBC 2012 (Code) seismic Site Class D. Based on the apparent age and soil structure of the subsurface soils, widespread liquefaction was determined not to pose a significant risk at this site, considering the

[1] The extent and type of foundations of the existing structures is not known at this time. Shallow footings should be removed and the excavations backfilled with soil before new foundation construction begins. If the existing buildings are found to be supported on deep foundations (piles or piers), then we should be notified and allowed to consider and advise upon options for proper surface/subsurface preparation to facilitate new building foundation construction.

anticipated ground accelerations associated with the design magnitude earthquake, although there is some potential for liquefaction to occur in isolated, discontinuous pockets and lenses of saturated, loose sands in the subsurface, potentially resulting in relatively small magnitudes of earthquake-related settlement.

5. **Seismic Design Parameters**: Based on the soil profile, and using the general procedure described in the Code, the following Site Class D seismic design parameters are applicable: F_A = 1.35, F_V = 2.02, S_{DS} = 0.51g, S_{D1} = 0.26g, and Mapped MCE Geometric Mean Peak Ground Acceleration (PGA_M) = 0.37g. For structures in Seismic Risk Category I, II, III, or IV, these parameters indicate Seismic Design Category D.

6. **Foundation Types:** Based on the provided loading of up to 675 kips, shallow foundations without ground improvement do not appear to be feasible for this structure due to excessive static settlements (up to 2 inches total, up to 1 ½ inches differential). Therefore, we recommend two options for the support of the building. The first option is shallow foundations supplemented by vibro-replacement stone columns. Following installation of the stone columns, shallow foundations may be designed for a working load bearing pressure of up to 5,000 psf. The second option is to support the structure on augered, cast-in-place, reinforced concrete piles (ACPs). The installation of 18-inch diameter ACPs embedded to a depth of at least 55 feet beneath the existing ground surface is estimated to provide 85 tons of allowable axial capacity with 22 tons of allowable axial uplift.

7. **Pavements:** For light-duty flexible (asphalt) pavements not subjected to truck traffic, we recommend the following minimum pavement section: 2.5 inches of Type C hot mixed asphalt (HMA) surface course over 8 inches of compacted graded aggregate base course. For heavy-duty flexible pavements subjected to truck traffic, we recommend the following minimum pavement section: 3.5 inches of Type B HMA consisting of 1.5 inches Type B Surface HMA over 2 inches Type B Intermediate HMA, over 8 inches of compacted graded aggregate base course. For heavy-duty rigid pavement areas, we recommend 4,000 psi compressive strength Portland cement concrete with a thickness of 7 inches, with dowel reinforcing at the joints, overlying a compacted graded aggregate base course thickness of at least 6 inches.

8. **Additional Exploration:** We recommend that several confirmation borings be performed after demolition of the existing structures, to confirm that the soils beneath the footprints of the demolished buildings are similar to the soils explored herein. If the deep foundation (pile) support alternative is selected, these supplemental borings are also needed in order to confirm that the soil conditions between depths of 55 and 65 feet (within ten feet below the recommended tip depth of the piles) is similar to the conditions that were assumed for the purposes of the pile support recommendation option that is presented in this report.

1. INTRODUCTION

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site, develop recommended geotechnical parameters for the design team to use during foundation design, slab design, and pavement design, and geotechnical recommendations to be considered during the construction of earthworks, foundations, and pavements. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations.

Sketches showing the approximate test locations are included in Appendix A. Appendix B includes a discussion of the field exploration procedures, and the hand auger, test pit, and sounding logs. Appendix C contains the results of the laboratory testing.

2. SITE AND PROJECT DESCRIPTION

Project information was initially provided in email correspondence from Ms. Michelle Motchos, P.E. of Stevens & Wilkinson to Mr. Marty Baltzegar, P.E., of S&ME, Inc. (S&ME) on July 17, 2014. Ms. Motchos' email attached two untitled, undated project drawings, a recent aerial photograph, and a Request for Proposal (RFP). Additional information was provided by Ms. Motchos during a telephone conversation with Mr. Will Kannon, P.E. of S&ME on July 18, 2014.

The proposed project site is located on North Irby Street across the street (west of) the existing Florence City-County Complex in Florence, South Carolina. The project site is currently developed with several commercial buildings fronting N. Irby Street in the eastern portion of the site. A site vicinity map is included in Appendix A as Figure 1. The remainder of the site is mostly covered in asphaltic pavements. Demolition of the existing structures and pavements have not been performed prior to our issuing this report.

As we understand it, site improvements include the construction of a three-story tall judicial building and associated pavements. The structure is planned to be steel framed and have a footprint of about 40,000 square feet in plan area.

We understand that the project is still early in the planning phase; however, the provided current maximum column and wall loads are 675 kips and 6 kips per linear foot, respectively. We anticipate that uniform floor area loads on the grade slab may be 150 pounds per square foot or less. If loading conditions differ significantly from the assumptions that we have made to facilitate our analysis, revisions to our recommendations scope may be warranted.

The proposed parking lot contains 196 public parking spaces, 11 secure parking spaces, as well as driveway areas. We anticipate that site pavements will consist of some areas of hot mixed asphaltic concrete as well as some areas of rigid Portland cement concrete. The anticipated traffic loading and volumes for the proposed parking lots and driveways were not provided to us prior to our issuing this report. For purposes of our analysis, we assumed traffic loading and frequencies. These assumptions may or may not reflect the actual traffic loads to be experienced by the pavements at this site.

Based on the RFP, we understand that the building pad may be established at an elevation of 146.5 feet Mean Sea Level (MSL), resulting in fill placements of up to 30 inches to establish plan grades. We have not been provided grading information for the remainder of the site, so for purposes of our analysis, we assume that proposed grades may be established near existing site grades, resulting in cuts or fills of less than 2 feet. Parking lot grading information should be provided to us once it becomes known. A retention or detention pond is proposed to occupy some portion of the eastern part of the site but is not currently reflected on the site plan.

3. EXPLORATION PROGRAM

This section describes the field exploration and laboratory testing program.

3.1 Field Exploration

On August 14, 15 and 18, 2014, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

1. We performed a site walkover, observing general features of topography, existing structures, ground cover, and surface materials at the project site;
2. We explored the subsurface soils at sixteen (16) discrete test locations. See Figure 2 in Appendix A for the approximate test locations. The following outlines our exploration procedures for this site:
   - We advanced four Cone Penetration Test (CPT) soundings, one Seismic Cone Penetration Test (SCPT) sounding, and one Marchetti Dilatometer (DMT) sounding to depths ranging from 10 to 55 feet. In conjunction with SCPT sounding, shear wave velocity measurements were recorded at approximate 1 meter depth intervals. In conjunction with the DMT sounding, modulus measurements were performed at approximate 1-foot depth intervals.
   - Subsurface water levels were not directly measured in the CPT and DMT soundings; the subsurface water levels at these locations were interpreted based upon pore pressure measurements. Where encountered, the water level was directly measured in the hand auger borings and test pits.
   - We performed one hand auger boring at each of eight (8) parking lot test locations to a depth of approximately four feet beneath the pavement surface. At locations HA-3, HA-6, HA-7, and HA-8, we cut cores in the asphalt surface using a truck-mounted coring machine to gain access to the underlying soils. The recovered asphalt cores were measured for thickness. Small grab samples of subsurface soil materials were collected from representative subsurface strata within the borings. Some of the soil cuttings within the upper 2 feet of the borings were also collected to form two composite bulk samples. Soils recovered from borings HA-1, HA-2, HA-4, HA-5, HA-6, and HA-7 formed composite sample “Bulk 1”, and soils

recovered from borings HA-3 and HA-8 formed composite sample "Bulk 2". Within the borings, our engineer observed and documented the subgrade soil types observed and subsurface water levels, where encountered.

- Dynamic Cone Penetrometer (DCP) testing was performed at regular depth intervals within the hand auger borings in general accordance with ASTM STP 399 procedures to help us estimate the relative density and consistency of the subgrade soils. Upon completion of our field work, we backfilled the boreholes with soil cuttings to the existing ground surface. Where borings were performed in asphaltic paved areas, the boreholes were backfilled with soil cuttings to within several inches of the asphalt pavement surface and patched with asphaltic cold patch.
- We conducted one double-ring infiltrometer test near the center portion of the site. The test, designated DRI-1, was performed at a depth of about 5 feet below the existing ground surface in an excavated test pit (TP-1). The test was performed at the approximate location marked as DRI-1/TP-1/HA-9 on Figure 2 attached in Appendix A. The infiltration 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". One hand auger boring (HA-9) was performed within the test pit bottom and was advanced to a depth of about 4 feet below the pit bottom.
- We excavated three test pits (TP-1 through TP-3) at the project site. The test pit locations were performed near the central portion of the site. During the test pit excavations, our on-site geotechnical professional, Will Kannon, P.E., observed the excavated test pit soils to visually-manually estimate the distribution of grain sizes, plasticity, moisture condition, color, presence of lenses and seams, and apparent geologic origin of the soil in general accordance with ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." At the conclusion of the field work, several recovered samples were returned to our laboratory; the test pits were backfilled with the excavation spoils and run over several times with the machine at the surface.

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

For a more complete description of the field exploration procedures used, please see the *"Summary of Field Exploration Procedures"* attached in Appendix B.

3.2 Laboratory Testing

After the recovered soil samples were brought to our laboratory, a geotechnical professional examined and/or tested each sample to estimate its distribution of grain sizes, plasticity, organic content, moisture condition, color, presence of lenses and seams, and apparent geologic origin in general accordance with ASTM D 2488, *"Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)"*. The resulting classifications are presented on the boring logs included in Appendix B. Similar soils were grouped into representative strata on the logs.

We performed the following quantitative ASTM-standardized laboratory tests on the composite bulk samples and small grab samples to help classify the soil and formulate our conclusions and recommendations. The laboratory tests performed included the following:

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

- Four samples tested in general accordance with ASTM D 4318, *"Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"*, to measure the plastic behavior of the soil.

- Four samples tested in general accordance with ASTM D 1140, "*Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve"*, to measure the percent clay and silt fraction.

- One sample tested in general accordance with ASTM D 422, *"Standard Test Method for Particle Size Analysis of Soils*," without the hydrometer portion, to measure the distribution of particle sizes greater than 75 micrometers.

- One sample tested in general accordance with ASTM D 1557, *"Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 lbf/ft3)"*, to measure the moisture-density relationship of the soil.

- One specimen from the selected bulk sample re-compacted and tested in general accordance with ASTM D 1883, *"Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils"*, to evaluate soil support characteristics for pavements.

The laboratory data sheets and a brief description of the procedures used for the above listed laboratory tests are attached to this report in Appendix C. The laboratory testing program was modified from the proposed testing program at the discretion of the engineer: one grain size analysis without hydrometer tests was replaced with two additional silt/clay fines content tests.

4. SITE AND SURFACE CONDITIONS

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

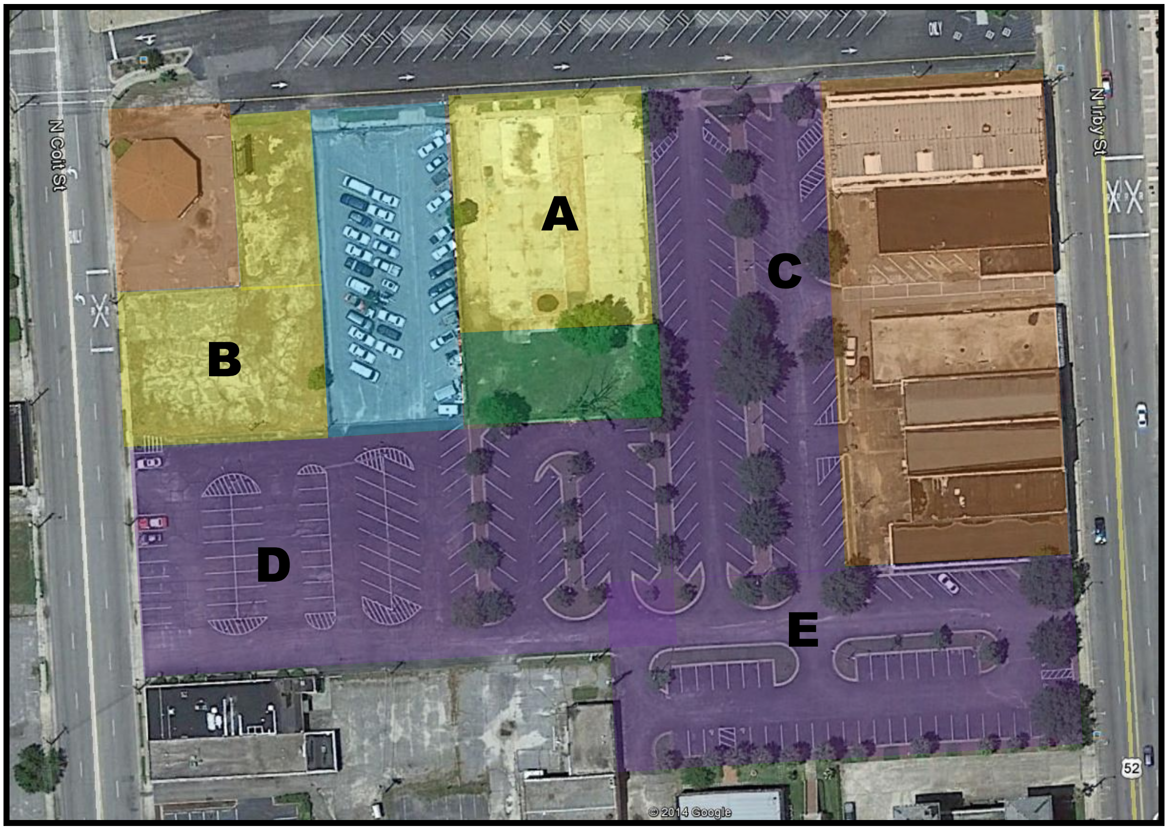
4.1 Topography

Ground surface elevations were not directly surveyed; however, based on our review of the USGS topographic map, site elevations appear to vary from approximately elevation 143 to 146 feet Mean Sea Level (MSL), generally sloping gradually downwards from east to west. Site elevations have been modified by previous development.

4.2 Ground Cover

The aerial photograph below (Figure 1) represents current site surface conditions. We have color-coded the different sections of the site in an effort to facilitate our description of existing site conditions.

Figure 1 – Existing Site Conditions



The **orange** colored sections represent existing structures and their associated walkways and concrete pavements. These sections will require demolition to remove the existing structures and concrete pavements.

The yellow colored sections represent existing concrete slabs or pavements. The concrete near the center of area "A" appears to be the old floor slab of a previously-demolished structure; the slab appears partially fragmented in places. Where we performed sounding C-2/TP-3, the concrete was approximately 12 inches thick and appeared to be comprised of two 5 to 6-inch thick slabs separated by a thin layer (1/2 to 1 inch) of soil. We attempted to penetrate the slab in several other areas with the backhoe, but were unsuccessful. Area "B" consisted of concrete pavements which are highly weathered and fragmented with numerous areas of exposed earth. We were able to perform hand auger borings and test pits within this area with relative ease.

The **purple** colored areas represent existing asphaltic concrete pavements. The general condition of these pavements is moderately distressed, with some areas in the western portion of the parking lot and behind the existing buildings (areas "C" and "D") exhibiting more distress generally in the form of block cracking and fatigue cracking. The main thoroughfare connecting N. Irby Street with N. Coit Street also exhibits some fatigue and longitudinal cracking.

The **blue** colored area represents a Florence Police impound area which was locked during the time of our exploration. The ground surface in this area is covered with asphaltic concrete pavements. These pavements appeared to be in poor repair with numerous areas of block and fatigue cracking throughout.

The **green** colored area represents an area of exposed bare earth. The near-surface soils are generally moist and we did not observe any areas of ponded water within this area of the site at the time of our exploration.

5. SUBSURFACE CONDITIONS

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

5.1 Regional and Local Geology

The site is located in the Coastal Plain Physiographic Region of South Carolina. The Coastal Plain extends from the eastern limit of the Piedmont ("Fall Line") eastward to the coast and consists of a wedge-shaped deposit of ancient marine sediments of the Late Cretaceous Period and younger. Coastal Plain soils comprise interbedded layers of normally-consolidated and over-consolidated limestone, gravels, sands, silts, and clays. This deposit ranges in thickness from near zero at the Fall Line to thousands of feet at the coast. In the site area depth to crystalline metamorphic rock is mapped to be roughly 300 meters.

A review of local geologic mapping indicates that the site area lies within an outcrop area of the Duplin Formation (T_d), typically inter-layered terrestrial clays, silts, and sands laid down during the Lower Pliocene Epoch approximately 3 million years ago. These materials weathered in place and have formed a mantle of about 50 feet in thickness which overlie less weathered, much older (approx. 65 million years), calcareous soils below. The surface has been reworked by erosional processes over geologic time, and the limestone residuum has been masked by deposits of loose to dense sands or firm to very stiff clays. The upper contact of the lower sands may be irregular due to localized scouring and redeposition of the overlying clays. Soils below a depth of about 49 to 50 feet at this site were mapped as Cretaceous-age sediments of the Donoho Creek Formation (Kdc).

5.2 Interpreted Subsurface Profiles

One subsurface cross-sectional profile of the site soils is attached in Appendix A as Figure 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 borings and soundings presented on the profile 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 profile to allow their properties to be systematically described.

5.3 Soil Stratigraphy

This section describes soil conditions observed across the site, as represented by profile A-A' in Figure 3 of Appendix A.

5.3.1 Asphaltic Concrete

Asphaltic concrete was encountered at the ground surface in borings HA-3, HA-6, HA-7, and HA-8. At each of these locations, the asphalt thickness was measured to be approximately 2 inches.

5.3.2 Aggregate Base Course

Beneath the asphalt pavements in borings HA-6, HA-7, and HA-8, aggregate base course materials were encountered. The aggregate base ranged in thickness from about 4 to 7½ inches.

5.3.3 Portland Cement Concrete

Portland Cement Concrete (PCC) was encountered at the ground surface in sounding C-2. The thickness of the PCC was about 12 inches total and was divided between two slabs about 5 to 6 inches thick each. A thin layer of soil about ½ to 1 inch in thickness separated the two slabs.

5.3.4 Undocumented Fill

At the ground surface at test locations HA-1, TP-1, and TP-2, beneath the asphalt in boring HA-3, beneath the aggregate base in borings HA-6, HA-7, and HA-8, and beneath the Portland cement concrete in sounding C-2, old (undocumented) fill soil was encountered to depths ranging from 1 to 3 feet beneath the ground surface. We suspect that fill soils and possible debris may also be present in some of the other cone soundings based on the abnormally high tip resistances observed in this layer to depths of about 2 to 3 feet.

The fill was generally comprised of sandy soils consisting of clayey sand (SC), silty sand (SM), silty-clayey sand (SC-SM), and poorly graded sand with silt (SP-SM). The fill soils were typically dark brown, red, or tan in coloration and were moist. Some construction debris (glass, brick fragments, and wood) were encountered beneath the concrete slab to a depth of about 3 feet in test pit TP-3, performed near test sounding location C-2. A layer of what appeared to be coal was also encountered in test pits TP-1 and TP-2 between depths of about 1 to 2 feet.

Penetration resistances to a Dynamic Cone Penetrometer (DCP) advanced through the fill soils ranged from 5 to 9 blows per increment (bpi), indicating a generally loose relative density. CPT tip resistance values were in the range of very loose to very dense sandy soils (10 to 400 tsf), with resistance values typically being higher near the ground surface. Some of the higher tip resistance values may have been amplified by debris within the upper several feet of the fill soils.

Dilatometer modulus values ranged from 140 to 1,160 tsf. DMT material index (Id) values obtained were mostly between 0.8 and 3, typical of “sandy” soils. Shear wave velocity measured within the fill layer was about 520 feet per second (fps).

Laboratory index testing of the fill soils typically exhibited natural moisture contents ranging from 8.0 to 12.3 percent, fines content (silt and clay fraction) ranging from 23.3 to 23.7, and Atterberg limits testing indicated liquid limits ranging from 21 to 25 percent, plastic limits ranging from 14 to 17 percent, and plasticity indices ranging from 4 to 11 percent, generally indicating low plastic behavior. Soils in this stratum may be considered as non-expansive as defined in IBC Section 1803.5.3.

Two bulk samples were collected from a combination of this old fill layer and of Stratum I. Soils recovered from borings HA-1, HA-2, HA-4, HA-5, HA-6, and HA-7 formed composite sample “Bulk 1”, and soils recovered from borings HA-3 and HA-8 formed composite sample “Bulk 2”. Bulk sample No. 2 was selected for further testing based upon the soil index test results, and was re-compacted in the laboratory using modified effort in accordance with ASTM D 1557 procedures; the resulting maximum dry density was 123.7 pounds per cubic foot (pcf) at an optimum moisture content of 9.8 percent.

California Bearing Ratio (CBR) testing was performed on a re-compacted portion of this sample (Bulk 2), with a test point re-compacted in the laboratory to approximately 95 percent of the modified maximum dry density. The CBR value at 95 percent compaction was 9.9 percent at 0.1 inches of penetration. These results indicate that these soils should provide suitable subgrade support when properly compacted near the optimum moisture content.

5.3.5 Stratum I: Upper Sands/Silty Sands

Underlying the fill soils at test locations HA-1, HA-3, HA-6, HA-7, HA-8, TP-1, TP-2, and TP-3, and at the ground surface at test locations HA-2, HA-4, and HA-5, Coastal Plain sandy soils consisting primarily of silty sands (SM) and clayey sands (SC) were encountered to depths of about 2 to 4½ feet.

Penetration resistances to a Dynamic Cone Penetrometer (DCP) ranged from 5 to 11 bpi, with an average penetration value of about 8 bpi indicating a generally loose relative density, with some medium dense layers. These soils were typically dark brown, brown, tan, and orange in color and were moist.

CPT tip resistance values were also typically in the range of very loose to loose sandy soils (about 5 to 30 tsf). Dilatometer modulus values ranged from about 162 to 197 tsf. Shear wave velocity measured within this layer was about 520 feet per second (fps).

Laboratory index testing of the Stratum I soils typically exhibited natural moisture contents ranging from 13.3 to 13.6 percent, fines content (silt and clay fraction) ranging from 25.9 to 29.4, and Atterberg limits testing indicated non- plastic behavior. DMT material index (Id) values obtained were mostly between 1.8 to 2.8, typical of “sandy” soils. Soils in this stratum may be considered as non-expansive as defined in IBC Section 1803.5.3.

5.3.6 Stratum II: Firm to Stiff Silts and Clays Interbedded with Sands

Underlying Stratum I, a layer of silts and clays interbedded with sands was encountered beginning at depths of about 2 to 4½ feet and continuing to depths of about 17 feet to 21 feet. CPT tip resistance values were typically in the range of soft to very stiff cohesive soils (10 to 60 tsf), but were typically in the range of 15 to 30 tsf, indicating stiff consistency soils. Occasional pockets of loose to medium dense sand were also observed in this stratum. Dilatometer modulus values ranged from 13 tsf to 430 tsf, but were typically on the order of about 150 to 300 tsf. DMT material index (Id) values obtained were between 0.6 to 1.8, typical of “silty” soils. Shear wave velocity within this stratum ranged from about 840 to 1,240 fps, and averaged about 1,000 fps.

5.3.7 Stratum III: Soft Silts and Clays

Underlying Stratum II, soft silts and clays were encountered to depths ranging from about 48 to 49 feet. DMT material index (Id) values obtained were mostly between 0.2 to 1, typical of “silty” and “clayey” soils. CPT tip resistance values were highly variable, ranging from about 2 tsf to 100 tsf, but were typically in the 10 to 20 tsf range, indicating soft consistency soils. Some layers within this stratum between depths of about 21 to 25 feet classified as sensitive clays, with very low cone tip resistance, cone sleeve friction, and modulus measurements being recorded. Dilatometer modulus values ranged from about 24 tsf to 199 tsf, but were typically on the order of about 60 to 80 tsf. Shear wave velocities within this stratum generally ranged from 550 to 820 fps, and averaged about 650 fps.

5.3.8 Stratum IV: Donoho Creek Formation: Interbedded Sands and Silty Sands

Underlying Stratum III, beginning at depths of 48 to 49 feet, a stratum of sands and silty sands was encountered to the deepest sounding termination depth of about 55 feet. DMT material index (I_d) values obtained were mostly between 2 to 4, typical of “sandy” soils. CPT tip resistance values were also typically in the range of medium dense to very dense sandy soils (about 80 to 250 tsf), but ranged as high as 400 tsf at a depth of about 53 feet. Dilatometer modulus values ranged from 235 to 335 tsf. Shear wave velocity within this stratum, was measured to range from about 500 to 740 fps; shear wave velocity is anticipated to increase significantly with depth below 55 feet. Shear wave velocity test results are also summarized on Figure 4 in Appendix A.

5.3.9 Laboratory Test Results

We performed laboratory testing on several samples to better assess the engineering properties of the subsurface soils. The laboratory soil index test results are presented in Appendix C and are summarized in the following tables.

Table 1 – Summary of Laboratory Soil Index Testing Results

Boring/ Sample No.	Sample Depth (Feet)	Natural Moisture Content (%)	Fines Content (%)	Atterberg Limits			USCS Classification
				LL	PL	PI	
HA-1	1 – 2	12.3	23.7	21	17	4	SC-SM
HA-5	1 – 2	13.6	25.9	--	NP*	---	SM
HA-9	8 – 9	5.9	14.4	--	--	--	SC
HA-1,2, 4-7 BULK 1	0.5 – 2	13.3	29.4	20	17	3	SM
HA-3,8 BULK 2	1 – 2	8.0	23.3	25	14	11	SC

*NP = Non-plastic

Table 2 – Summary of Moisture-Density and CBR Test Results

Boring/Sample No.	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	CBR at 0.1 in. Penetration – 95% Compaction (%)
HA-3, 8/BULK 2	123.7	9.8	9.9

5.4 Subsurface Water

The subsurface water level was interpreted to range from depths of about 7 to 7.7 feet below the ground surface at the time of the exploration, based upon the pore pressure readings measured in the CPT soundings. The measured water level in boring HA-9 at the time of boring was about 8.5 feet below the ground surface. We anticipate that these water levels may represent perched water trapped in sandy lenses of Stratum II, atop the less permeable soils of Stratum III, rather than a true water table or aquifer. USGS testing of wells in this vicinity indicate that stable shallow water table levels may vary between about 19 to 27 feet below the ground surface. 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 Rate

The stabilized (saturated) infiltration rate measured at test location DRI-1, which was performed at a depth of about 5 feet below the existing ground surface and within the sandy lean clay layer encountered at that depth, was about 0.02 inches per hour (iph). The USDA Soil Survey classifies an infiltration rate of 0.02 iph as being “very slow”.

A summary of the field test results is presented in Appendix B. When choosing the value for infiltration rate that is ultimately used in design, the designer needs to consider the variability of the soils and understand that a slight change in the silt or clay fines content could have a significant impact upon the infiltration rate. As silt or clay content increases, infiltration rate is likely to decrease.

The above description of subsurface conditions is relatively brief and general. More detailed information may be obtained from review of individual sounding and test pit logs, included in Appendix B of this report.

6. SEISMIC SITE CLASS AND DESIGN PARAMETERS

Seismic-induced ground shaking at the foundation is the effect taken into account by seismic-resistant design provisions of the 2012 International Building Code (IBC). Other effects, including landslides and soil liquefaction, must also be considered.

6.1 Selection of 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.

6.1.1 Evaluation of the Potential for Site Class F Conditions

The initial step in site class definition is to check for the four conditions described for Site Class F, which would require a site specific evaluation to determine site coefficients F_A and F_V. Soils vulnerable to potential failure include the following: 1) quick and highly sensitive clays or collapsible weakly cemented soils, 2) peats and highly organic clays, 3) very high plasticity clays, and 4) very thick soft/medium stiff clays. These soils were not evident in the borings or sounding.

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

Our analysis, which is more fully described in Section 6.3 below, indicates widespread liquefaction to be unlikely at this site; therefore, Site Class F does not apply at this site.

6.1.2 Average Shear Wave Velocity

Based on shear wave velocities measured at the site, we determined that site response factors F_A and F_V corresponding to Site Class D would be applicable to determine spectral values for design. This recommendation is provided based on the average weighted shear wave velocities measured to a depth of 55 feet and interpolated to a depth of 100 feet. The average weighted shear wave velocities was estimated to be about 1,000 feet per second, which is greater than the 600 feet per second that is required for consideration of Site Class D design parameters. See Figure 4 in Appendix A for the shear wave velocity profile used in this analysis.

Note: It may be possible to improve this site to Site Class C, which requires an average shear wave velocity to 100 feet of greater than 1,200 fps; however, this would require additional testing such as Multi-channel Analysis of Surface Waves (MASW) in order to measure the shear wave velocities of the soils between depths of 55 and 100 feet.

6.2 Seismic Design Coefficients for Site Class D

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

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

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

The 2012 IBC specifically references ASCE 7-10 for determination of peak ground acceleration value for computation of seismic hazard. Peak ground acceleration is separately mapped in ASCE 7-10 and corresponds to the geometric mean Maximum

Credible Earthquake (MCE_G). The mapped PGA value is adjusted for site class effects to arrive at a design peak ground acceleration value, designated as PGA_M.

Table 3: Spectral Design Values

	2012 IBC (2008 Seismic Hazard Maps)
S_{DS}	0.51
S_{D1}	0.26
F_A	1.35
F_V	2.02
PGA_M	0.37 g

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

6.3 Analysis of Liquefaction Potential

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

We performed a liquefaction analysis based on the design earthquake prescribed by the 2012 edition of the International Building Code (IBC 2012), the "simplified procedure" as presented in Youd et al. (2001), and recent research concerning the liquefaction resistance of aged sands (Hayati & Andrus, 2008; Andrus et al. 2009; Hayati & Andrus, 2009). Our analysis was based upon a peak ground surface acceleration of 0.37g.

The sands encountered at this site do not appear likely to undergo widespread liquefaction in the event of the design earthquake. Our qualitative assessment considered the relative density, fines content, and apparent geologic age of the soils. However, due to the sandy soil profile, the presence of water, and the pockets of loose sands that were intermittently observed at different depths and within different thickness zones within the test soundings, it is possible that minor soil liquefaction may occur in discontinuous pockets and isolated lenses during seismic shaking associated with the design level earthquake.

Our analysis shows that, in the event that this occurred, the anticipated settlements associated with the liquefaction are unlikely to exceed 2 inches. If the site is improved with vibro-replacement stone columns, the stone columns are likely to act as pressure relief vessels for the excess pore pressures that may try to develop in the soils during seismic shaking, and the anticipated settlements would therefore likely be reduced to near zero in the vicinity of the improved soils.

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

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

The average LPI for this site was estimated to be about 5, which indicates that the risk of surface damage due to liquefaction is low. Using this information as a guide, it was determined that Site Class F conditions should not apply to this site.

7. 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, if the proposed building location is changed, if either the structural or civil design information is revised, 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.

Undocumented fill soils were encountered in several of our borings and undocumented fill is likely present across much of the site. The recommendations given below for site preparation and foundation and grade slab construction are dependent on the nature and extent of the undocumented fill soils. These soils will need to be evaluated in the field after the site has been stripped to determine the nature and extent of fill remaining on site and any potential stabilization requirements.

7.1 Surface Preparation

Site preparation should include removal of unsuitable materials from within the building and pavement footprints. This should include surface vegetation, organic-laden topsoil, debris, and any unstable surface or subsurface soils.

This proposed site is located in an area of previous development. Existing structures and several large concrete slabs are located at the site. Undocumented fill was also encountered in several of the borings to depths of 1 to 3 feet below the existing ground surface. The potential exists that additional old underground structures, foundations, or debris may be encountered during construction as well as previously placed fill material. The following recommendations are provided regarding site preparation and earthwork:

1. Remove grade slabs, underground structures, or other debris from beneath the footprint of the structure. Strip existing pavements where these occur in the areas of proposed grade slabs or pavements. The extent and type of foundations of the existing structures is not known at this time. Shallow footings should be removed and the excavations backfilled with soil in accordance with Section 7.2 before new foundation construction begins. If the existing buildings are found to be supported on deep foundations (piles or piers), then we should be notified and allowed to consider and advise upon options for proper surface and subsurface preparation to facilitate new building foundation construction.

2. Remove or plug existing utilities to be abandoned prior to construction. If not removed or plugged, pipes may serve as conduits for subsurface erosion resulting in formation of voids below foundations or floor slabs. Where existing utilities are left in place and plugged in the building footprint, it may be necessary to undercut poorly compacted backfill to provide adequate support for footings or slabs. Re-route existing utilities remaining in use around the proposed building footprint.

3. Existing buildings to be demolished may have asbestos-containing interior finishes, insulation, or roofing and restrictions which may apply to disposal of demolition debris. Assessment of these conditions was beyond the scope of this exploration, but you may wish to investigate this matter further before demolition. S&ME is able to offer this service, if desired.

4. During grading, the site should be crowned and ditched to promote positive drainage away from the working surface. This will help reduce the potential for moisture damage to the subgrade during earthwork operations and should help to maintain stabilization of the subgrade.

5. After surface drainage is established, but before mass grading or foundation construction begins, the existing subgrade surface should be densified with a heavy vibratory roller prior to placement of any new fill. The exposed 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 8 inches below the surface.

   a) Under favorable moisture conditions and with the proper equipment, this may be able to be accomplished by densifying the soil from the working surface. However, under less favorable conditions, it may be necessary for the contractor to re-work (or remove, condition, and replace) the upper 8 inches of the native material, using moistening or drying techniques, in order to achieve the desired level of compaction.

   b) The densification of these soils should be performed under the observation of an S&ME representative.

6. Following densification, the densified native subgrade surface should be proofrolled by the contractor under the observation of an S&ME representative. Proofrolling should be performed by making several passes with a fully-loaded dump truck or water truck, or similar high ground pressure equipment. The proofrolling should be conducted only during dry weather. Areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to new fill placement, as advised by the Geotechnical Engineer (S&ME) at the time of construction.

7. Place fill in accordance with Section 7.2 below. Once final design soil subgrade elevation has been achieved, all subgrade soil surfaces should be proofrolled by the contractor under the observation of an S&ME representative. Proofrolling should be performed by making several passes with a fully-loaded dump truck or water truck, or similar high ground pressure equipment. The proofrolling should be conducted only during dry weather. Areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to base course construction, as determined by the geotechnical engineer.

7.2 Fill Placement and Compaction

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

1. It is recommended that fill soils used to build up the ground for structures and pavements meet the following minimum requirements: plasticity index of 10 percent or less; clay/silt fines content of not greater than 30 percent. This may include soils from the following ASTM soil classifications: SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, and/or SC. However, not all soils in these categories will comply with the plasticity and fines content requirements. 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.

2. Structural fill under buildings and pavements should be compacted to at least **95 percent** of the maximum dry density as defined by ASTM D1557-09 *"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 8 inches in thickness prior to compaction (limited to 4 inches if using small, hand-operated equipment such as a walk-behind vibrating plate tamp or pneumatic "jumping jack" tamp).

c) Structural fill should extend at least 5 feet laterally beyond the edge of buildings, foundations, and pavements before being allowed to exhibit a lesser degree of compaction.

d) In non-structural fill areas only, such as in landscaped areas that are located at least 5 feet outside the footprint of buildings, foundations, and pavements fill should be compacted to at least 90 percent of the maximum dry density by the Modified Proctor criterion (ASTM D 1557).

3. Fill placement should be observed by an S&ME soils testing technician working under the guidance of the geotechnical engineer.

   a) At least one field density test should be performed per each 2,500 square feet for each lift of soil in large area fills, with a minimum of 2 tests per lift.

   b) 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 or behind walls, with a minimum of 1 test per lift.

   c) At least one field density test should be conducted for each 250 linear feet of road alignment backfill, with a minimum of 1 test per lift per section.

7.3 Use of Excavated Soils as Structural Fill (Fill Suitability)

The sandy soils of Stratum I generally appear to meet the criteria recommended in Section 7.2 for fill source material, but may require some moisture conditioning prior to compaction. If an on-site detention pond is excavated and the intent is to use the excavated material as fill on the building pad, it should be considered that significant pockets of material that is unsuitable for use as fill may be encountered. Previous fill soils containing debris may not be suitable for use as fill. The portions of the old fill containing coal or similar materials will not be suitable for use as fill. The clayey and silty soils of Stratum II and Stratum III do not appear to be suitable for use as fill. It should be anticipated that most if not all of the new fill used to build up the pad for this site may need to be imported.

The native sandy soils, when properly compacted and with proper erosion control measures, should be capable of holding a stable slope of 2H:1V, or gentler.

None of the upper soils observed on this site appear to be expansive. Swell measurements taken during CBR testing of the bulk sample indicated swell during saturation of 0.1 percent.

7.4 Consideration of Shallow Foundations

For the proposed building, based on the provided maximum column load of 675 kips, an assumed uniform floor slab applied area load of 150 psf, and a 2,000 psf bearing pressure for isolated spread foundations, the estimated total post-construction static settlement of an individual spread footing measuring approximately 18.5 feet by 18.5 feet in plan area will likely be on the order of 2 inches, and differential settlements between dissimilarly loaded footings may be up to 1½ inches. Depending upon individual column spacing, settlements could be greater if footing loading zones of influence overlap.

Based on our conversation with Ms. Motchos on September 9, 2014, settlements of this magnitude are likely to be unacceptable, and the resulting footing size may be prohibitive. For these reasons, we do not recommend the utilization of shallow foundations alone for the support of the structure. We recommend that the structure be supported on either shallow foundations supplemented with ground improvement using vibro-replacement stone columns (Option 1), or supported on deep foundations (Option 2), as described in the following sections of this report.

7.5 Option 1 – Shallow Foundations with Stone Columns

One option to consider would be to support the new building on vibro-replacement stone columns. Stone columns can provide two benefits. First, the columns act as stiff, reinforcing elements within the soft consistency soils beneath the building, which may reduce static settlement magnitudes to acceptable levels (typically 1 inch or less). Second, the stone columns can provide densification of loose sands in the immediate vicinity of the columns, and provide a drainage pathway for the loose sands which further reduces the already low potential for soil liquefaction during the shaking associated with the design seismic event. For the purposes of this report, the liquefaction mitigation benefit is considered secondary, because the LPI is less than 5 and the liquefaction hazard is below the threshold typically considered necessary to require design mitigation; nevertheless, the stone columns do provide a benefit in this regard. The primary purpose of the stone columns is to reduce static settlement and to allow a significant increase in the design bearing pressure of the spread footings.

If vibro-replacement stone columns are designed and installed, the proposed structure can then be supported by shallow strip and spread footings resting on existing soil reinforced by the stone columns. The columns are typically constructed by driving a hollow mandrel to the design depth and compacting aggregate fed through the hollow mandrel in thin lifts as the mandrel is removed. Installation and compaction densifies the aggregate and increases lateral stress in the soil matrix. The system serves to reduce settlement by displacing and densifying and reinforcing the soils below the footing with a stiffer composite soil matrix.

We preliminarily estimate that reinforcement depths may range from about 40 to 50 feet. Based on our past experience, when stone columns are utilized in conjunction with shallow foundations, bearing capacity can generally be increased to about 5,000 psf to size the shallow foundations. Footing size of a 675 kip column could then be reduced from an estimated 18.5 feet by 18.5 feet (if an unimproved soil bearing pressure of 2,000 psf were used), to about 11.5 feet by 11.5 feet. Based upon a preliminary estimated replacement ratio of approximately 20 percent, we preliminarily estimate that each 675 kip column footing may possibly be supported by up to five, 24 to 30-inch diameter stone columns in an “X” pattern configuration.

Stone columns are typically provided in a design-build contract by a specialty contractor. In developing the final design criteria, the actual column spacing and diameter should be determined by requesting a design-build cost proposal from selected specialty contractors experienced with these methods.

The goal of the ground modification program should be to limit total and differential settlements of the foundations to tolerable levels. Based on our experience with similar projects, total settlements can usually be reduced to less than 1 inch, and differential settlements to less than ½ inch. The contractor should submit a proposal to furnish all necessary labor, equipment, and materials to *design and install* a ground modification program based on these or other specified criteria. The proposals should be evaluated by the project Geotechnical and Structural Engineers, and then a contractor should be selected based on technical approach, past experience, and cost.

A test program should be conducted prior to full-scale ground modification of the site. At least one compression load test of a stone column should be performed to confirm the contractor's modulus design. If the columns will be used for pullout resistance, then a pullout load test should also be performed. Load testing should be witnessed by a representative of the Geotechnical Engineer.

The Geotechnical Engineer's representative should make continuous observations of ground improvement operations to confirm that:

1) The proper depth of improvement is achieved, and

2) The volume of material installed is sufficient to obtain the theoretical column diameter.

Field observation reports should include a log of each column that includes: column identification, date of installation, probe number, start/finish time, backfill quantities, theoretical diameter of column, column location, existing ground surface elevation, and top/bottom elevation of each column.

7.6 Option 2 – Augered Cast-in-Place Reinforced Concrete Piles (ACPs)

ACPs are a secondary recommendation option for the support of the structure in the event that stone columns are not preferred or are not deemed to be cost effective. ACP's have the advantage of being relatively economical to install and have a comparatively high axial capacity with regard to the soil conditions observed at this site versus other types of piles. Additionally, construction-related noise and vibration impact to surrounding structures are typically lower than that of driven piles. For these reasons, this pile type appears to be preferable to install at this site. Some constructability issues for this deep foundation type are discussed later in this report.

Most of the time it is advisable for the ratio of the pile length not to exceed about 40 times the pile diameter. Since piles will have to extend to a depth of about 55 feet to bear a sufficient distance into the Donoho Creek Formation, 18-inch diameter piles are preferable. Continuous observation of the pile installation by a qualified Special Inspector will be required during construction, per the IBC (Table 1705.8).

7.6.1 ACP Capacities

Axial capacities versus depth were estimated for individual 18-inch diameter ACPs based upon the subsurface conditions encountered in the borings. The soil profile for this recommendation was modeled based upon the subsurface conditions observed in sounding C-1.

We note that in order to maximize the available pile capacity, we had to extend the pile to the greatest depth to which we have soil data, which is 55 feet. Therefore, if this foundation support alternative is selected, then we should be asked to return to the site and advance at least one additional soil boring or test sounding to a depth of 65 feet, in order to confirm that the soil conditions between depths of 55 and 65 feet are similar to those observed at a depth of 55 feet. Based upon the soil formation that our deepest test sounding terminated in, we don't expect a reduction in soil strength below 55 feet; however, this should be verified by supplemental exploration, as further described in Section 7.10 of this report.

The estimated axial capacities available for design are summarized in Table 4 below.

Table 4 – Single ACP Vertical Capacities

Pile Type & Diameter	Approximate Pile Length (feet)	Allowable Axial Capacity* (tons)	Allowable Uplift Capacity** (tons)
18-inch ACP	55	85	22

*Allowable capacity assumes a factor of safety of 2 applied to the estimated ultimate axial end bearing capacity and a factor of safety of 3 applied to the estimated ultimate skin friction capacity.
** Uplift capacity assumes a factor of safety of 3 applied to the estimated ultimate skin friction 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. Soils in the upper five feet of the soil profile were considered not to contribute to pile resistance or down-drag. 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 ACP capacities recommended herein should 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. More information regarding the test pile program is discussed in Section 7.6.6.

7.6.2 Difficult Drilling Conditions and Auger Refusal

If during the installation of the ACPs, auger refusal is not met, then the piles should be advanced to at least 55 feet below existing grade. This was considered during the development of our pile capacity recommendations. Based on the soils encountered during our exploration, we do not anticipate that auger refusal will be routinely encountered above the specified pile tip termination depth; however, it is possible that isolated, very hard lenses within the Donoho Creek Formation could cause auger refusal above this depth at some locations. Therefore, the auger refusal criterion is recommended to be defined as an auger advancement rate of less than 1 inch per minute for at least 10 minutes at the full down-crowd pressure. It is important that the pile installer does not stop trying to advance the pile at the first encounter of a hard lens, because such lenses may be relatively thin, and in such case would not be suitable for support of the pile.

7.6.3 ACP Capacity Reductions and Group Effects

Auger cast piles are essentially small-diameter drilled shafts. Therefore, for "large groups" of shafts or auger cast piles where each pile in the group is completely surrounded by other piles at a spacing of no less than 3 pile diameters center-to-center, a reduction factor may need to be applied to the estimated single pile capacities given above. The reduction factor may range from 0.7 to 1.0, and depends upon the pile spacing and soil conditions. If the piles are spaced at least 6 diameters apart center-to-center (9 feet for an 18-inch diameter pile), then no reduction factor for group effects needs to be considered. Intermediate reduction factors may be used for small groups of piles or intermediate spacings, depending upon other factors.

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, unless all of the piles are spaced at least six diameters center-to-center. The actual pile layout configuration should be determined by the structural design engineer.

Under 2012 IBC Section 1808.2.8.5, the maximum uplift of a column supported by a pile group would be limited by two-thirds of the effective weight of the soil contained within a block outlined by the perimeter of the pile group. Pile groups proposed for use on this project will need to be checked for uplift capacity, but a typical 2 x 2 pile group of 18-inch diameter piles with tips bearing 55 feet below the surface and a pile spacing of at least 4.8 diameters center-to-center would have an effective total uplift capacity of four times the single pile uplift capacity, or about 88 tons, using this approach. For spacing less than 4.8 pile diameters, the block weight is anticipated to control the design for uplift. For a minimum pile spacing of 3.0 pile diameters, the estimated pullout of the block is about 44 tons, assuming the entire block and cap to be in uplift at the same time.

7.6.4 Lateral Pile Reactions for Assumed Loads

Our lateral pile analyses were performed using the computer program LPILE Plus ©[2]. 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 18-inch diameter ACPs, reinforced with 4, # 8's positioned vertical and in a square pattern embedded about 55 feet below existing grades were analyzed. A vertical load

[2] Reese, Lymon C., Wand, Shin-Tower, LPILE^{PLUS}, Version 5, Ensoft, Inc., 2000.

equivalent to the allowable axial compressive capacity was applied to each modeled auger cast pile. Lateral loads ranging from approximately 28 to 47 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).

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 Appendix C. Lateral deflection and maximum bending moment of typical 18-inch diameter auger cast piles were estimated for the assumed lateral shear loads to consider possible non-uniform loading of individual pile reaction within a group for static loading conditions.

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 above, lateral deflections at the pile heads were computed for applied lateral loading and applied moments and are provided in Table 5.

Table 5 – Lateral Loads for Fixed Head Conditions, 18 inch dia. ACPs

Pile Diameter (inches)	Applied Load (tons)	Embedment Depth (feet)	Deflection (inches)	Static Lateral Load (kips)	Maximum Shear Force (kips)	Maximum Bending Moment (in-kips)
18	85	55	¼	28.4	28.4	495
	85	55	½	37.4	37.4	725
	85	55	¾	44.4	44.4	885
	85	55	1	47.4	47.4	1,000

Depth to essential fixity of an 18-inch diameter auger cast pile under fixed head condition appears to range from about 30 to 35 feet. Point of fixity was 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 ACPs 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.

7.6.5 Settlement of Auger Cast 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 18-inch diameter pile, this would typically equate to less than ¼ inch of vertical displacement associated with elastic shortening. Considering consolidation of the bearing soils to be represented by an average elastic modulus of roughly 600 ksf, total settlement of a single pile is estimated to be roughly ¼ to ½ inch. To this would be added the elastic shortening of the individual piles as described above of less than ¼ inch, for a single pile settlement on the order of about ½ to ¾ inches at the full working load.

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.

We should 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.

7.6.6 Auger Cast 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 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 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 verify 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.

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.

b) During axial compressive testing, the test pile should be loaded to at least 2.5 times the single-pile allowable design capacity. It is desirable to load the piles to 3.0 times the single pile capacity if the contractor is able. 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. If twice the allowable pile capacity is achieved for the test pile, then the allowable working design capacities may be considered verified. If less than twice the allowable pile capacity is achieved for the test pile, then the geotechnical engineer should be consulted to re-evaluate the pile design capacities based upon the test pile results.

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 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 1.2 for ACPs, 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 verify achievement of the design compressive strength. We also recommend that S&ME be requested to be present on-site to observe all concrete placements.

7.7 Grade Slab Support and Construction

The following recommendations are given for the support and construction of soil-supported grade slabs, if any. It is important for the design engineer to recognize that soil-supported grade slabs will settle differentially from pile-supported portions of the building frame, and from foundation elements that are supported on stone column improved soils unless the soils beneath the grade slabs are also improved. The magnitude of differential settlement may not be estimated until actual floor slab loads are known.

1. Soils similar to those penetrated by the borings should provide adequate support to lightly-loaded[3] soil-supported grade slabs, assuming preparation and compaction of the subgrade as recommended. A modulus of subgrade reaction (k) of 175 lbs/in^3 may be used for reinforcing design.

2. In areas of the facility where finished climate-controlled spaces are present, such as office space, we recommend that a polyethylene vapor barrier such as

[3] The design engineer should consider structurally tying the floor slab to the foundation, so that the load on the slab is distributed to the foundations for partial support, and does not rely entirely upon the immediate slab subgrade soil for support. This may help to reduce the differential settlement potential between the building frame, which is expected to be either pile-supported or supported on stone column improved soils, and the floor slab. Alternatively, a slab that is pile-supported, or supported on soils that have been improved with vibro-replacement stone columns, may be designed, rather than a soil-supported grade slab.

"Visqueen," or equivalent, be placed over the subgrade prior to placing interior floor slab concrete in order to limit moisture vapor infiltration into the finished spaces.

3. Have the geotechnical engineer observe all slab subgrades prior to concrete placement. Softened or weakened soils may need to be undercut or stabilized before concrete placement.

7.8 Lateral Earth Pressures for Shallow Buried Structures

The equivalent fluid pressures given below should be used to design near surface soil retaining structures in the upper 5 feet of the native soil profile or within fill zones. Under static conditions, the equivalent at-rest fluid pressure should be used to design soil-retaining structures which are fixed at the top against rotation.

Walls which will not be fixed at the top prior to application of the lateral pressures should also be designed to withstand the active earth pressures as a cantilevered wall. The values given in the following table assumes placement and compaction of backfill around these structures in accordance with the compaction recommendations given in Section 7.2 of this report.

These values assume level backfill generally classified as silty sand (SM) or sand with silt (SP-SM) soils according to the Unified Soil Classification system. These assumptions were made based upon the use of the on-site near surface sands (Stratum I) as the typical backfill material.

Table 6 – Lateral Earth Pressure Coefficients

Support Condition	Angle of Internal Friction (φ')	Moist Unit Weight (γ)	DRAINED	
			Static Earth Pressure Coefficient (K)	Dynamic Earth Pressure Coefficient (K) PGA = 0.37g
Active Condition (K_a)	30°	117 pcf	0.33	0.46
At-Rest Condition (K_o)	30°	117 pcf	0.50	0.68
Passive Condition (K_p)	30°	117 pcf	3.00	2.67

1. 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.
2. A coefficient of sliding friction (tan δ) of 0.36 may be used in computation of the lateral sliding resistance.
3. Lateral earth pressure coefficients may vary if compacted backfill is used around subsurface structures.
4. These earth pressure coefficients assume cohesionless soils. The actual soils may have a small amount of cohesion, which is ignored for the purposes of this recommendation.

If soil retaining structures are overexcavated and formed, and then backfill is placed and compacted in accordance with the compaction recommendations given in section 7.2 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.

Organic silts (OL or OH), inorganic silts or elastic silts (ML or MH), or inorganic plastic clays (CL or CH) soils should not be used as backfill behind earth-retaining structures.

Footings near proposed retaining walls may impose surcharge loads in addition to the earth pressures tabulated above. Alternatively, you may elect to extend footings to bear entirely below a line projected upward at a 45 degree angle from the inner toe of the wall to avoid placing surcharge pressures on the wall due to footing loads.

Compact the backfill directly behind cantilevered walls with light, hand-held compactors. Heavy compactors and grading equipment should not be allowed to operate within 10 feet of cantilevered walls during backfilling to avoid developing excessive temporary or long-term lateral soil pressures. The soil backfill placed behind retaining walls should be compacted to at least 95 percent of the soil's Modified Proctor maximum dry density. We caution that operating compaction equipment directly behind earth retaining structures can create lateral earth pressures far in excess of those recommended for design. Therefore, bracing of the walls may be needed during backfilling operations.

Provide positive gravity drainage of the backfill using a permanent toe drain to limit buildup of hydrostatic pressures in the backfill. Gravity drainage may consist of a minimum two foot wide blanket of clean crushed stone or washed sand, separated from the backfill by a properly graded filter or approved filter fabric, or a specially designed geotextile material such as Enka-drain, or equivalent. Vertical drains should be tied into a permanent "toe" drain installed at the base of the wall. Where gravity drainage of retaining walls is not feasible, design walls to resist hydrostatic forces in addition to lateral earth pressure.

7.9 Pavement Recommendations

Based on the subsurface conditions and assuming our grading recommendations will be implemented as specified, the following presents our recommendations regarding typical pavement sections and materials.

We anticipate and assume that new pavement subgrades will be constructed atop compacted fill soils or native soils densified to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). We performed CBR testing on one bulk composite sample recovered between depths of approximately 0.5 and 2 feet within the proposed pavement areas. We have performed our pavement calculations assuming a minimum CBR value of 7 percent. If soils exhibiting a CBR value of less than 7 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. However, in order to illustrate the potential traffic capacities that may result from some typical pavement sections, we have calculated the allowable equivalent single axle loads (ESALs) during the design life of the pavement for typical flexible and rigid pavement sections. These results are provided in Table 7 below.

For the purpose of developing our pavement thickness recommendations, we assumed standard-duty pavements may experience up to 500 passenger cars and light (2-axle) truck two-way trips per day and 10 light (6-wheel) delivery truck trips per week for 20 years duration, producing a design load of about 55,000 ESALs for consideration during our pavement thickness calculations.

For the main thoroughfare connecting N. Irby Street and N. Coit Street, we assumed a total 500 passenger car and light truck two-way trips per day, 10 bus trips per day, 2 garbage truck trips per week, and 10 light delivery truck round trips per week, 5 tractor trailer truck two-way trips per day for 20 years duration, producing a design load of about 420,000 ESALs for consideration during our pavement thickness calculations.

If the actual traffic will be greater than the values assumed, then the pavement section thicknesses may need to be increased above those presented in Table 7.

7.9.1 Pavement Thickness Computations

Pavement computations were made using the AASHTO 1993 method. Flexible pavement design assumes an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 95 percent. ESALs per axle were estimated using data provided in AASHTO literature. Assuming that only SCDOT approved source materials will be used in flexible pavement section construction, we used a structural layer coefficient of 0.44 for the HMA layers and a coefficient of 0.18 for the graded aggregate base course (GABC) layer.

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 all of the joints in the rigid pavement, we used an average load transfer coefficient of 2.7. Unreinforced concrete pavements would need to be at least 1 inch thicker to accommodate the same traffic loading. We also assumed a minimum 28-day design compressive strength of at least 4,000 psi for the PCC.

An overall sub-base drainage factor of 0.85 was assigned, based upon the assumption that the sub-base soils may consist of native silty/clayey soils.

Table 7: Recommended Minimum Pavement Sections [(a)]

Pavement Type	Theoretical Available Traffic Capacity (ESALs)	HMA Surface Course (inches)	HMA Intermediate Course (inches)	4,000 psi PCC Pavement Section (inches)	Compacted SCDOT Graded Aggregate Base Course [GABC] (inches)
Parking HMA Flexible (no heavy trucks)	55,000	2.5 (Type C)	---	---	6
Heavy-Duty HMA Flexible (with truck traffic)	420,000	1.5 (Type B)	2.0 (Type B)	---	8
Heavy-Duty PCC with joint reinforcement	420,000	---	---	7	6
Heavy-Duty PCC without joint reinforcement	420,000	---	---	8	6

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

7.9.2 General Recommendations for Pavement Areas

1. At least one laboratory California Bearing Ratio (CBR) test should be performed upon a representative soil sample of each soil type which is planned to be used as pavement subgrade material. This is to establish the relationship between relative compaction and CBR for the soil in question, and to confirm that the obtained CBR value at the required level of compaction is equal to or greater than the CBR value utilized during design of the pavement section.

2. All fill placed in pavement areas should be compacted as recommended in Section 7.2 "Fill Placement and Compaction". Prior to placement of graded aggregate base course stone, all exposed pavement subgrades should be methodically proofrolled under the observation of the geotechnical engineer (S&ME), and any identified unstable areas should be repaired as directed.

7.9.3 Base Course Construction

1. Crushed stone aggregate base material used in pavement section construction should consist of graded aggregate base course (GABC) as defined by Section 305 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007). The base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC-T-140). The base course material should not exhibit pumping or rutting under equipment traffic.

2. Heavy compaction equipment is likely to be required in order to achieve the required base course compaction, and the moisture content of the material will likely need to be maintained near optimum moisture content in order to facilitate proper

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

7.9.4 Asphaltic Concrete Construction

1. Construct the surface course hot mixed asphalt (HMA) in accordance with the specifications of Section 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition).

2. It is important that the HMA be properly compacted, as specified in Section 401.4 of the SCDOT specification. HMA that is insufficiently compacted will show wear much more rapidly than if it were properly compacted.

3. 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. We recommend that the specifications include requirements for obtaining pavement core samples for thickness and density measurements.

4. Experience indicates that a thin surface overlay of asphalt pavement may be required in about 7 to 10 years due to normal wear and weathering of the surface. Such wear is typically visible in several forms of pavement distress, such as aggregate exposure and polishing, aggregate stripping, asphalt bleeding, and various types of cracking. There are means to methodically estimate the remaining pavement life based on a systematic statistical evaluation of pavement distress density and mode of failure. We recommend the pavement be evaluated in about 6 years to assess the pavement condition and remaining life.

7.9.5 Rigid Concrete Construction

1. For rigid pavements, we recommend air-entrained ASTM C 94 jointed 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).

2. Our pavement thickness recommendations assumed that appropriately designed load transfer devices (dowels) will be used at all of the joints in the rigid pavement.

3. We recommend that at least 1 set of 5 test cylinder specimens be cast by S&ME per every 100 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 a certified S&ME concrete technician be requested to be present on site to observe all concrete placement activities.

7.10 Recommendations for Additional Exploration Work

Because this exploration was performed prior to demolition, it is our recommendation that additional exploration is necessary in the areas beneath the existing structures once demolition is completed.

As a preliminary scope of supplemental exploration, we recommend that at least two CPT soundings be performed within the footprint of the building in the areas overlain by the existing structures. One sounding should be advanced to a depth of at least 25 feet. The other sounding should be advanced to a depth of at least 65 feet to allow us to evaluate the bearing strata for possible deep foundations. We also recommend that several test pits be excavated within this area to further assess the near-surface soils with respect to old fill and possible debris.

It may be possible to improve this site to seismic Site Class C, which requires an average shear wave velocity to 100 feet of greater than 1,200 fps; however, this would require additional testing such as Multi-channel Analysis of Surface Waves (MASW) in order to measure the shear wave velocities of the soils between depths of 50 and 100 feet. Please let us know if this additional service is desired.

8. 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 test 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; this may result in an additional fee for services.

In the event that any changes in the nature, design, or location of the structures, pavements, or other appurtenances are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions are modified or verified in writing by the submitting engineers.

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

APPENDIX A

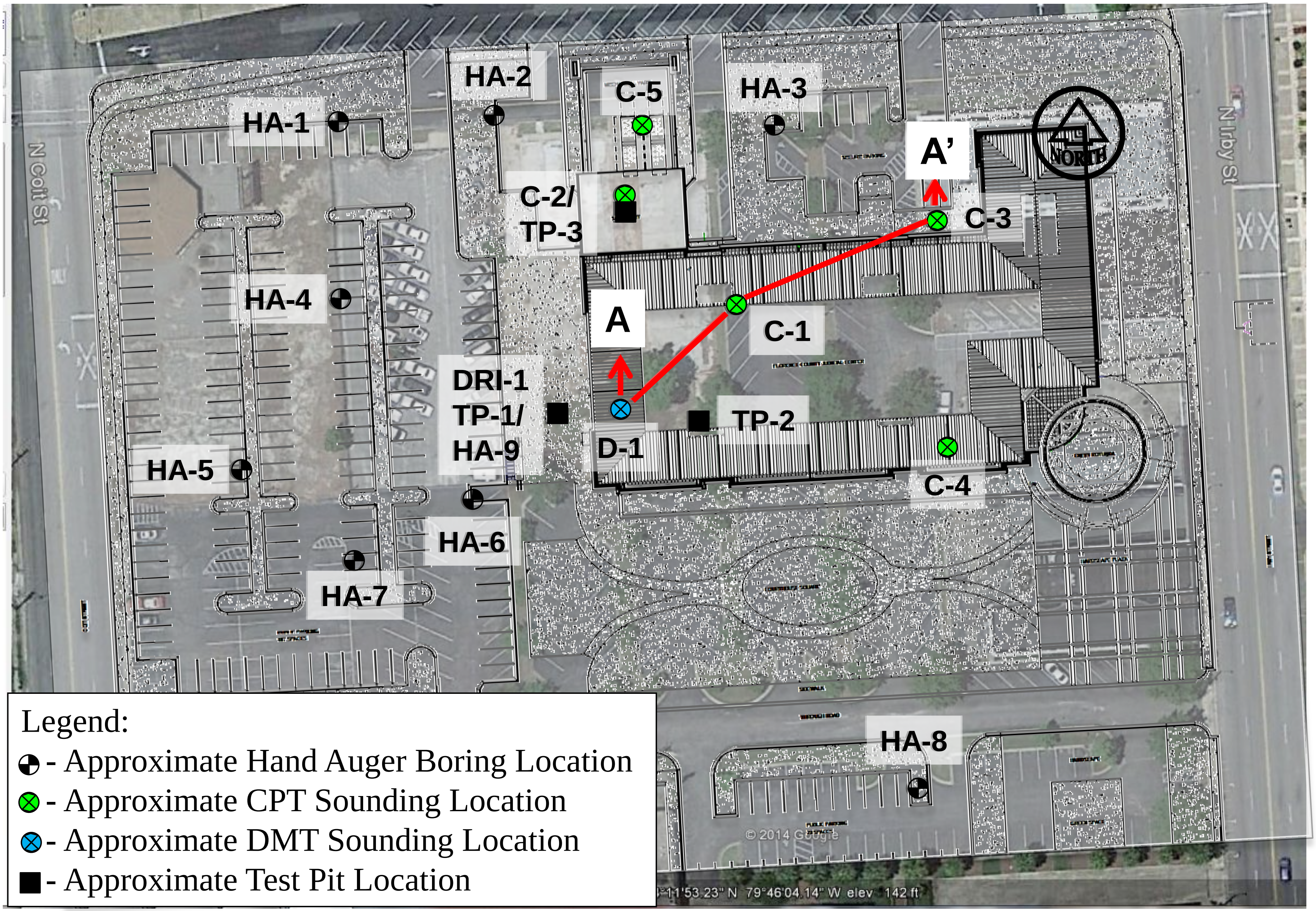
FIGURE 1: SITE VICINITY MAP

FIGURE 2: TEST LOCATION SKETCH

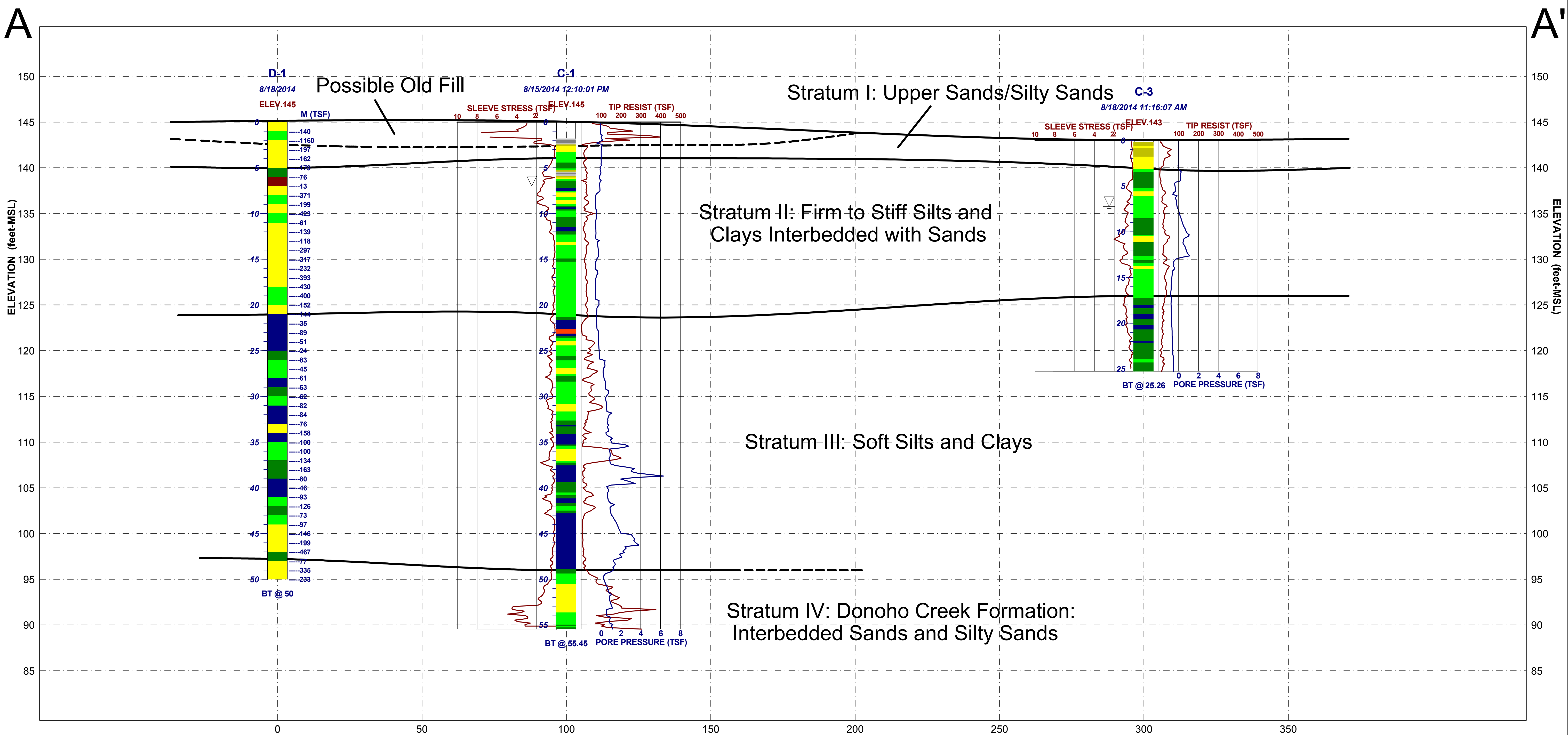
FIGURE 3: SUBSURFACE CROSS-SECTIONAL SOIL PROFILE A-A'

FIGURE 4: SHEAR WAVE VELOCITY PROFILE

SCALE: NTS	S&ME ENGINEERING • TESTING ENVIRONMENTAL SERVICES	**SITE VICINITY MAP** Florence County Judicial Center Florence, South Carolina	FIGURE NO. 1
DRAWN BY: WDK			
CHECKED BY: RPF			
DATE: Sept. 2014		PROJECT NO.: 1439-14-021	

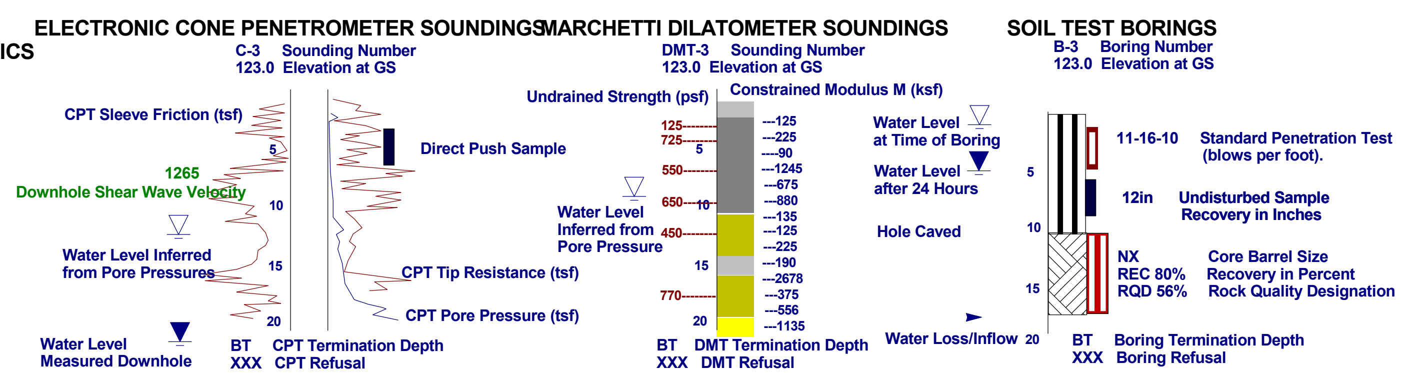


SCALE: NTS	S&ME	BORING LOCATION SKETCH Florence County Judicial Center Florence, South Carolina	FIGURE NO 2
CHECKED BY: RPF			
DRAWN BY: WDK			
DATE: September 2014		JOB NO. 1439-14-021	



CPT/DMT MATERIAL GRAPHICS

- Sensitive Fine Grained Soils
- Organic Soils, Peats
- Clay to Silty Clay
- Clayey Silt to Silty Clay
- Silty Sand to Sandy Silt
- Clean Sand to Silty Sand
- Gravelly Sand to Sand
- OC Sand to Clayey Sand
- OC Fine Grained Soils

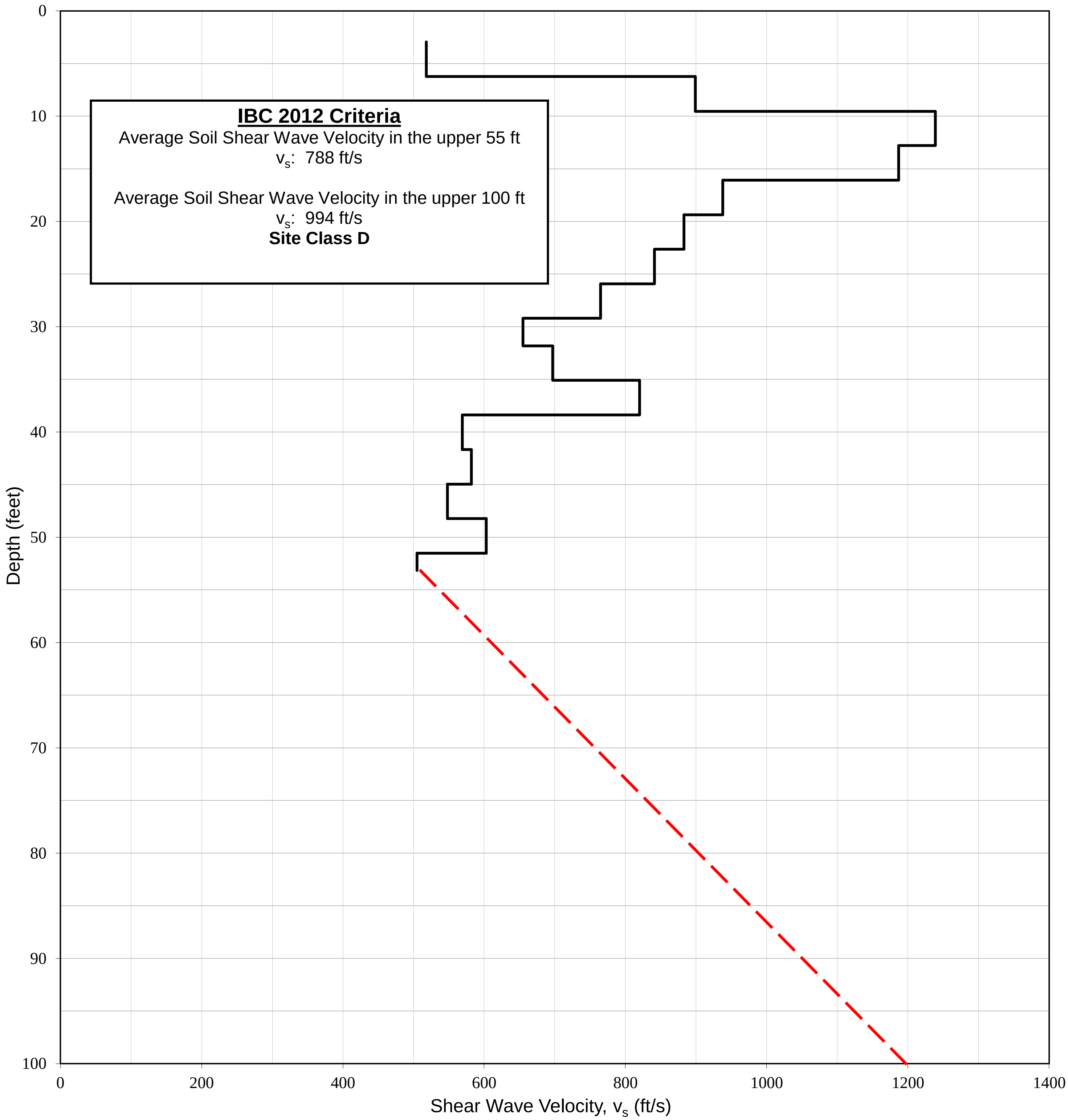


LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

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

SUBSURFACE PROFILE - FIGURE 3	JOB NO: 1439-14-021
PROJECT: Florence Judicial Center	
LOCATION: Florence, SC	DATE: 9/8/14

S&ME
ENGINEERING • TESTING
ENVIRONMENTAL SERVICES



Note: The average soil shear wave velocity in the upper 100 ft. was determined assuming a shear wave velocity of 1,200 fps from 50 ft. to 100 ft.

Shear Wave Velocity Profile	S&ME	Sounding ID:	SC-1	Drawn By:	WDK
Florence County Judicial Center		Date:	9/26/14	Approved By:	RPF
Florence, South Carolina		Project Number:	1439-14-021	Figure:	4
		File No.:	N/A		

APPENDIX B

SUMMARY OF EXPLORATION PROCEDURES

CPT CLASSIFICATION LEGEND

CPT SOUNDING LOGS

DMT FORMULAS AND SOIL CLASSIFICATION LEGEND

DMT SOUNDING LOG

SOIL CLASSIFICATION CHART

HAND AUGER LOGS

TEST PIT LOGS

DRI DATA SHEET

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.

Underground utility surveys were conducted by Duke Energy Progress personnel. S&ME was not involved with the utility location process.

BORING AND SAMPLING

Electronic Cone Penetrometer (CPT) Soundings

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

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

Refusal to CPT Push

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

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.

Water Level Determination

Subsurface water levels in the soundings were interpreted from pore pressure readings obtained during the performance of the CPT soundings. Water levels were not directly measured.

SUMMARY OF EXPLORATION PROCEDURES

Marchetti Flat-Plate Dilatometer Soundings

A single dilatometer test consists of pushing a flat blade located at the end of a series of rods to a target depth. Dilatometer soundings consisted of a series of individual dilatometer tests conducted on one to two foot intervals. At each testing depth, a circular steel membrane located on one side of the blade was expanded horizontally into the soil using gas pressure. The pressure on the membrane was recorded before expansion, after expansion, and again after deflation. After appropriate corrections for membrane stiffness and gage pressure deviation from zero, the corrected readings were used to estimate soil constrained modulus, coefficient of lateral earth pressure, material classification, and pore pressure using the procedures described in FHWA Publication SA-91-044, "*The Flat Dilatometer Test*."

Backhoe Excavated Test Pits

Test pits were excavated to obtain information about the shallow soil conditions. Test pits allow observation of the soil composition with depth. A field engineer was present to examine the soil strata exposed in the pits, observe the relative ease of excavation, observe the amount of subsurface water entering the pits, and document the soil types encountered and the depth that the pits were excavated. After completion of excavation, the pit was backfilled with the spoil materials; however, since the pit was a relatively narrow, deep excavation, very limited compactive effort could be applied to the backfill. Backfill was bucket-tamped during placement.

Hand Auger Borings

The asphaltic concrete was cored at boring locations HA-3, HA-6, HA-7, and HA-8 to provide access to the underlying soils. The cores were measured for thickness in the field.

Auger borings were advanced using hand-operated augers. The soils encountered were identified in the field by cuttings brought to the surface. Representative samples of the cuttings were placed in plastic bags and transported to the laboratory. 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.

Ground Water Level Determination

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

SUMMARY OF EXPLORATION PROCEDURES

Backfilling and Patching

After the groundwater measurements, boreholes HA-3, HA-6, HA-7, and HA-8 were backfilled with soil cuttings to a depth of about 3 inches below the asphalt surface and then patched with asphalt cold patch. The other hand borings were backfilled with soil cuttings to the ground surface.

CPT Soil Classification Legend

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

(*) Heavily Overconsolidated or Cemented

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

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

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

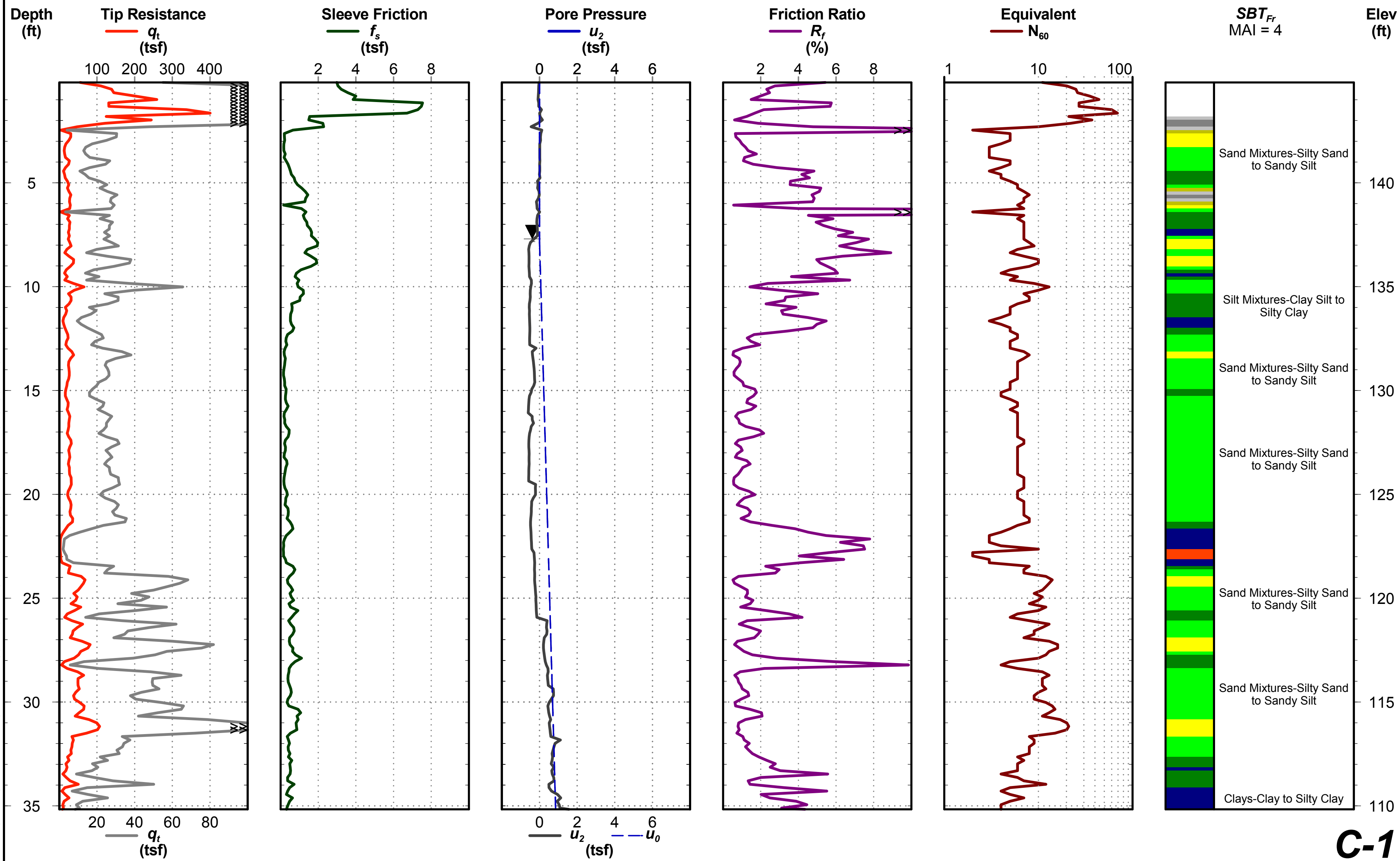
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-1

Date: Aug. 15, 2014
Estimated Water Depth: 7.7 ft
Rig/Operator: Marooka 300/Austin Fowler

Total Depth: 55.5 ft
Termination Criteria: Target Depth
Cone Size: 1.44"



CPT REPORT - STANDARD - SBT FR 1439-14-021 FLORENCE JUDICIAL CENTER.GPJ S&ME.GDT 9/16/14

C-1

Electronic Filename: Florence Judicial Center C-1.cpt

S&ME

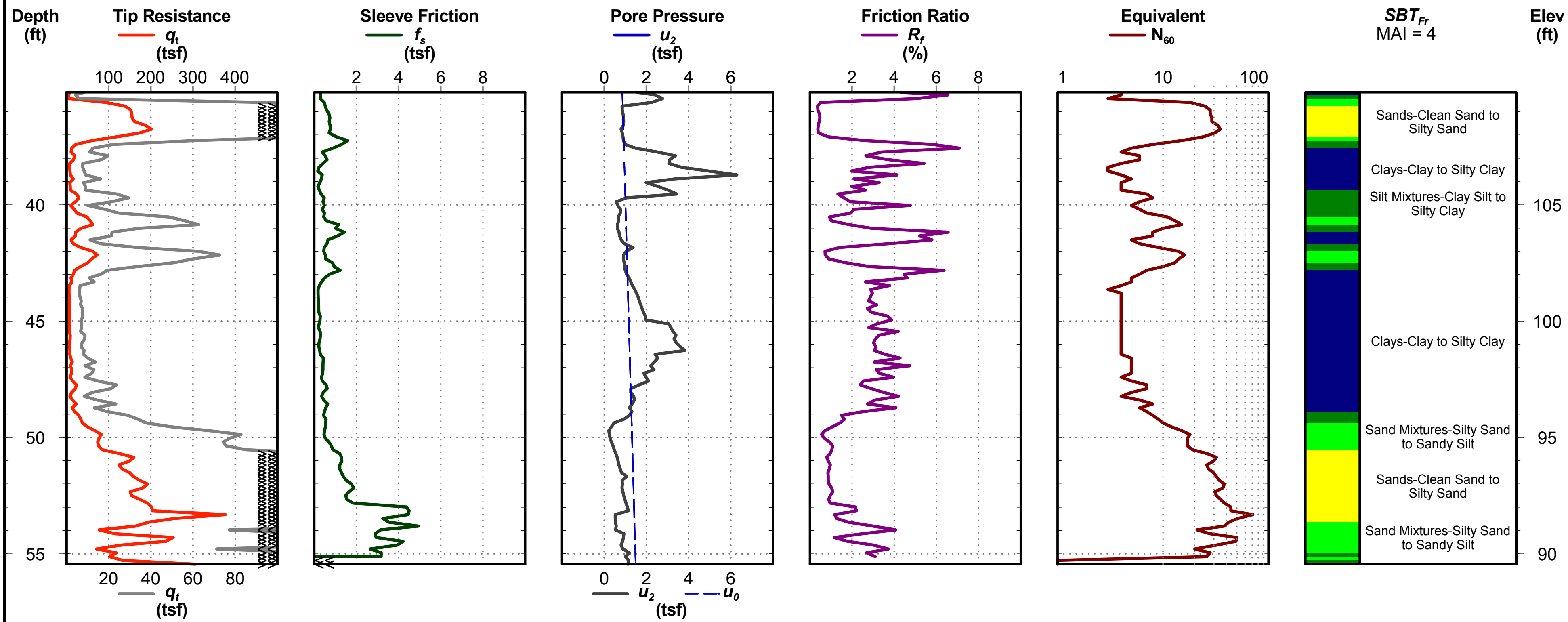
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-1

Date: Aug. 15, 2014
Estimated Water Depth: 7.7 ft
Rig/Operator: Marooka 300/Austin Fowler

Total Depth: 55.5 ft
Termination Criteria: Target Depth
Cone Size: 1.44"



Electronic Filename: Florence Judicial Center C-1.cpt

S&ME

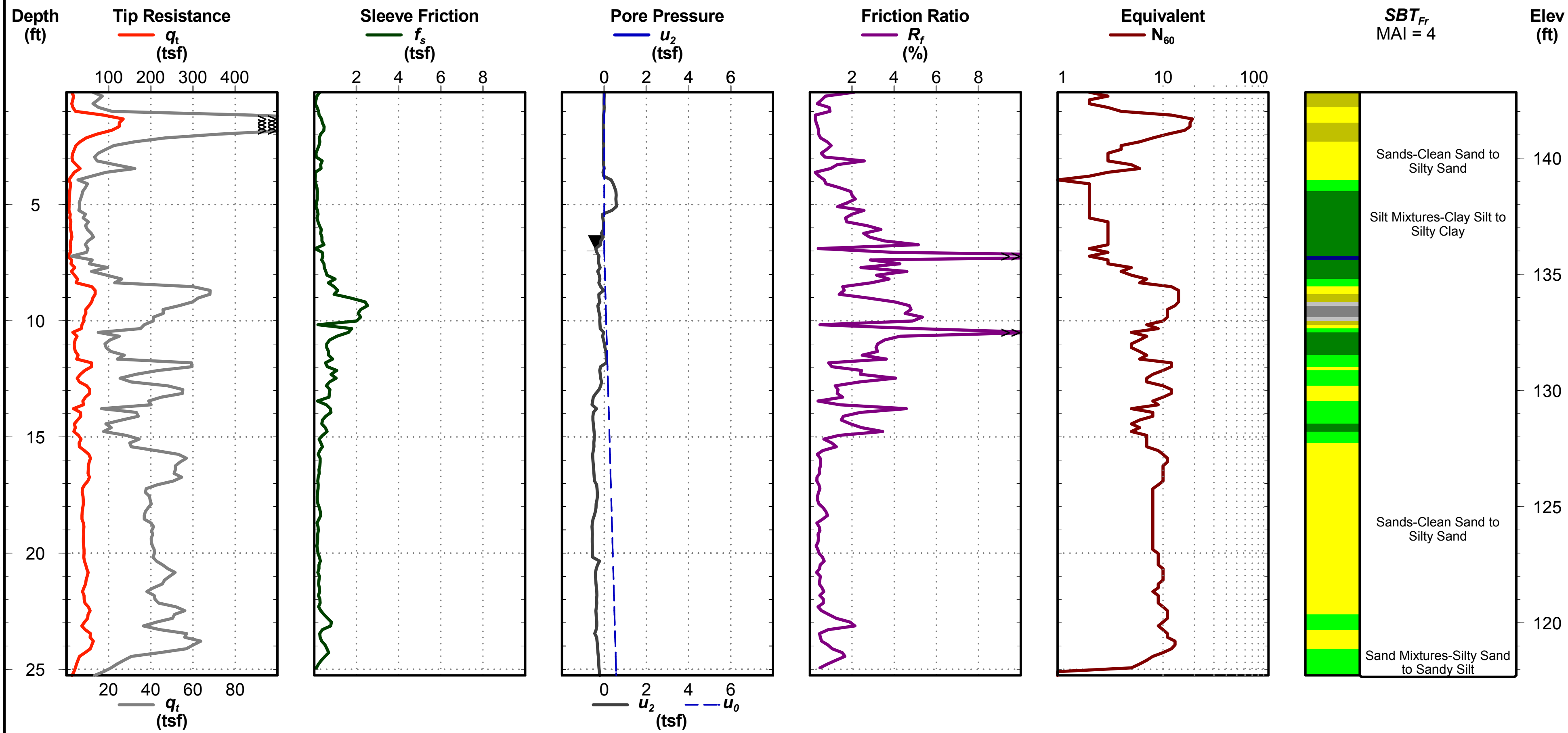
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-2

Date: Aug. 15, 2014
Estimated Water Depth: 7 ft
Rig/Operator: Marooka 300/Austin Fowler

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



CPT REPORT - STANDARD - SBT FR 1439-14-021 FLORENCE JUDICIAL CENTER.GPJ S&ME.GDT 9/16/14

C-2

Electronic Filename: Florence Judicial Center C-2.cpt

S&ME

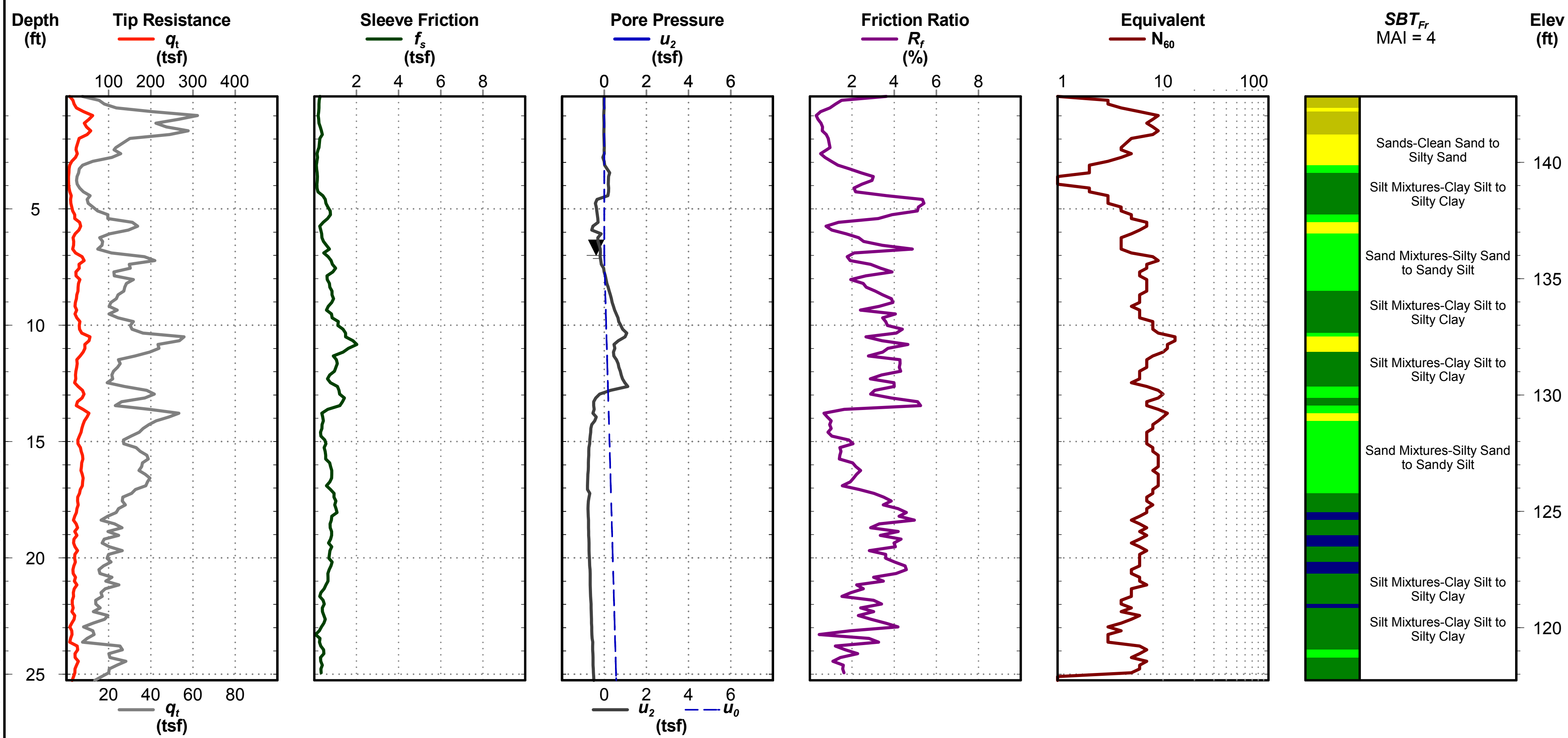
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-3

Date: Aug. 18, 2014
Estimated Water Depth: 7 ft
Rig/Operator: Marooka 300/Austin Fowler

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



C-3

Electronic Filename: Florence Judicial Center C-3.cpt

S&ME

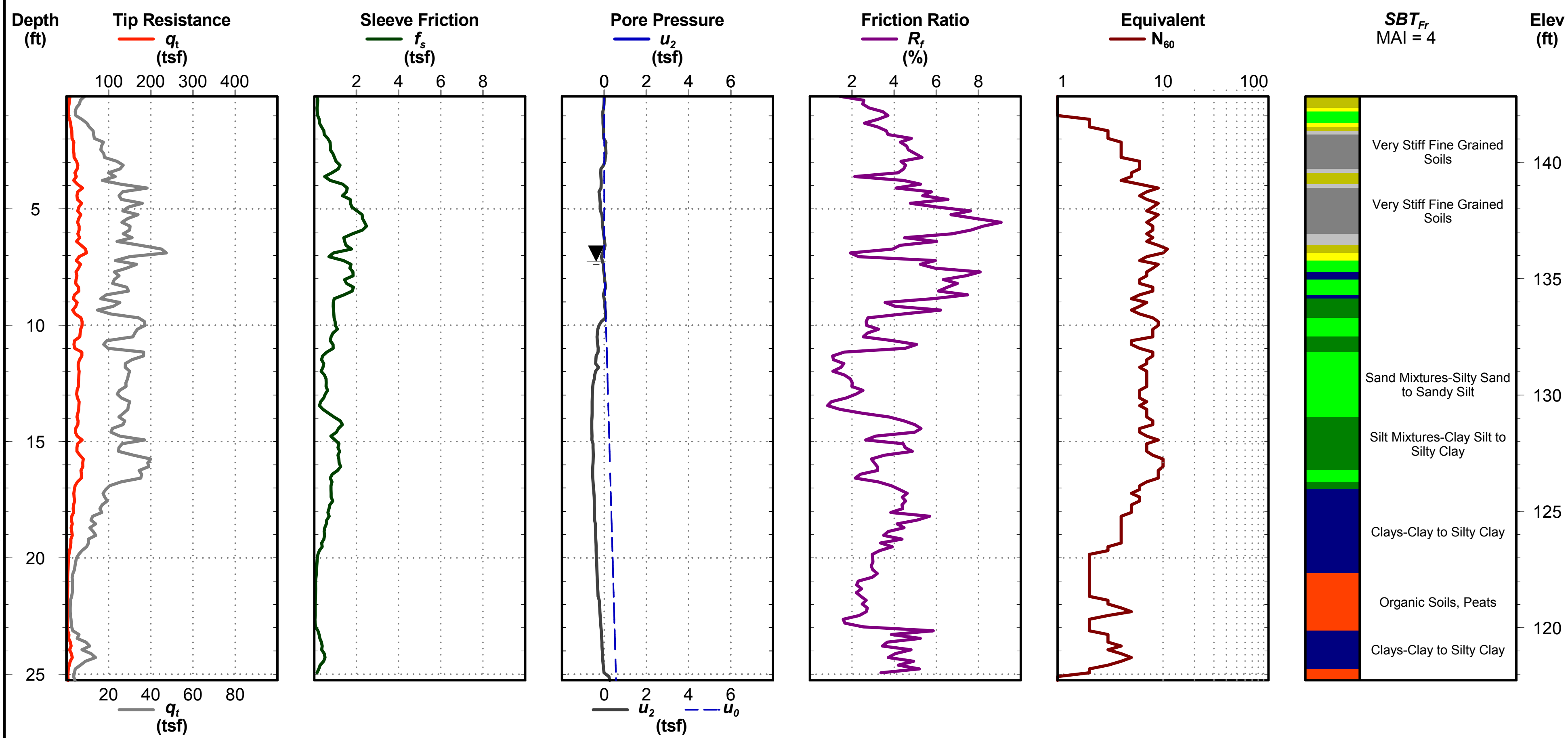
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-4

Date: Aug. 18, 2014
Estimated Water Depth: 7.25 ft
Rig/Operator: Marooka 300/Austin Fowler

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



CPT REPORT - STANDARD - SBT FR 1439-14-021 FLORENCE JUDICIAL CENTER.GPJ S&ME.GDT 9/16/14

C-4

Electronic Filename: Florence Judicial Center C-4.cpt

S&ME

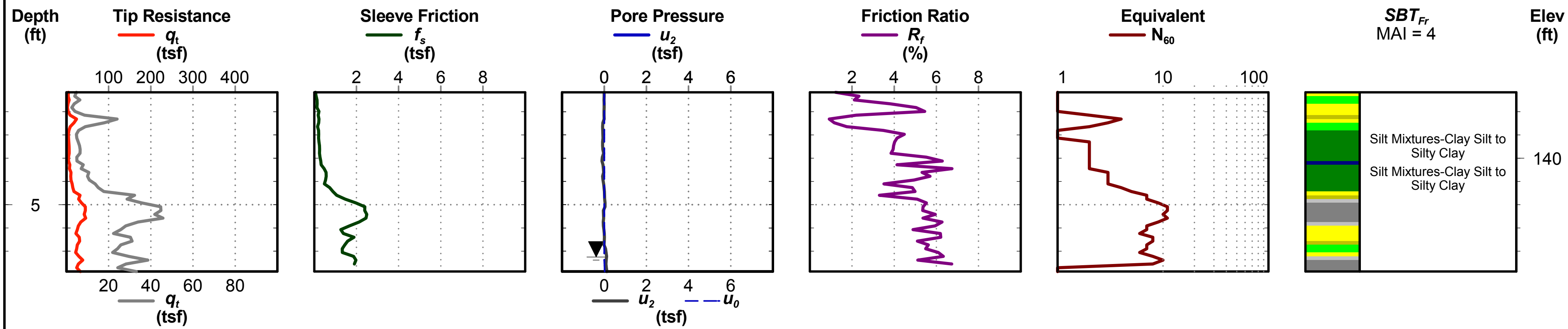
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Cone Penetration Test

C-5

Date: Aug. 18, 2014
Estimated Water Depth: 7.25 ft
Rig/Operator: Marooka 300/Austin Fowler

Total Depth: 7.9 ft
Termination Criteria: Target Depth
Cone Size: 1.44"



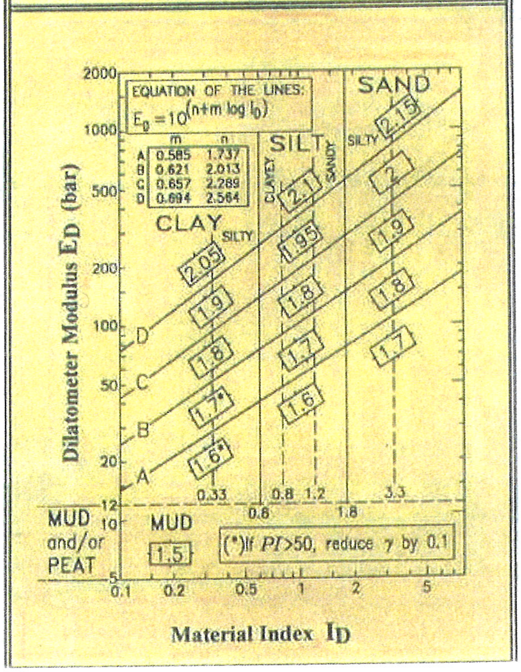
C-5

Electronic Filename: Florence Judicial Center C-5.cpt

DMT Formulas & Soil Classification Legend

SYMB	DESCRIPTION	BASIC DMT REDUCTION FORMULAE	
p_0	Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$	Z_M = Gage reading when vented to atm. However, if ΔA & ΔB are measured with the same gage used for current readings A & B, set Z_M=0 (Z_M is compensated)
p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$	
I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$	u_0 = pre-insertion pore pressure
K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{vo}$	σ'_{vo} = pre-insertion overburden stress
E_D	Dilatometer Modulus	$E_D = 34.7\ (p_1 - p_0)$	E_D is NOT a Young's modulus E. E_D should be used only AFTER combining it with Kd (Stress History). First obtain $M_{DMT} = R_M E_D$, then e.g. $E \approx 0.8\ M_{DMT}$
K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$	for $I_D < 1.2$
OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\ K_D)^{1.56}$	for $I_D < 1.2$
C_u	Undrained Shear Strength	$C_{u,DMT} = 0.22\ \sigma'_{vo}\ (0.5\ K_D)^{1.25}$	for $I_D < 1.2$
φ	Friction Angle	$\varphi_{safe,DMT} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D$	for $I_D > 1.8$
c_h	Coefficient of Consolidation	$C_{h,DMTA} \approx 7\text{cm}^2 / T_{flex}$	T_{flex} from A-log t DMTA-decay curve
k_h	Coefficient of permeability	$k_h = c_h\ \gamma_W / M_h \quad (M_h \approx K_0\ M_{DMT})$	
γ	Unit Weight and Description	(see chart)	
M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M\ E_D$ if $I_D \le 0.6$ $\quad R_M = 0.14 + 2.36 \log K_D$ if $I_D \ge 3$ $\quad R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $\quad R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ where $R_{M,0} = 0.14 + 0.15(I_D - 0.6)$ if $K_D > 10$ $\quad R_M = 0.32 + 2.18 \log K_D$ if $R_M < 0.85$ set $R_M = 0.85$	
U_0	Equilibrium pore pressure	$U_0 = p_2 \approx C - Z_M + \Delta A$	In freely draining soils

SOIL DESCRIPTION and ESTIMATED γ/γ_W



S&ME, Inc
620 Wando Park Boulevard
Mt. Pleasant, South Carolina 29464

(843) 884-0005
(843) 881-6149 fax
www.smeinc.com

S&ME

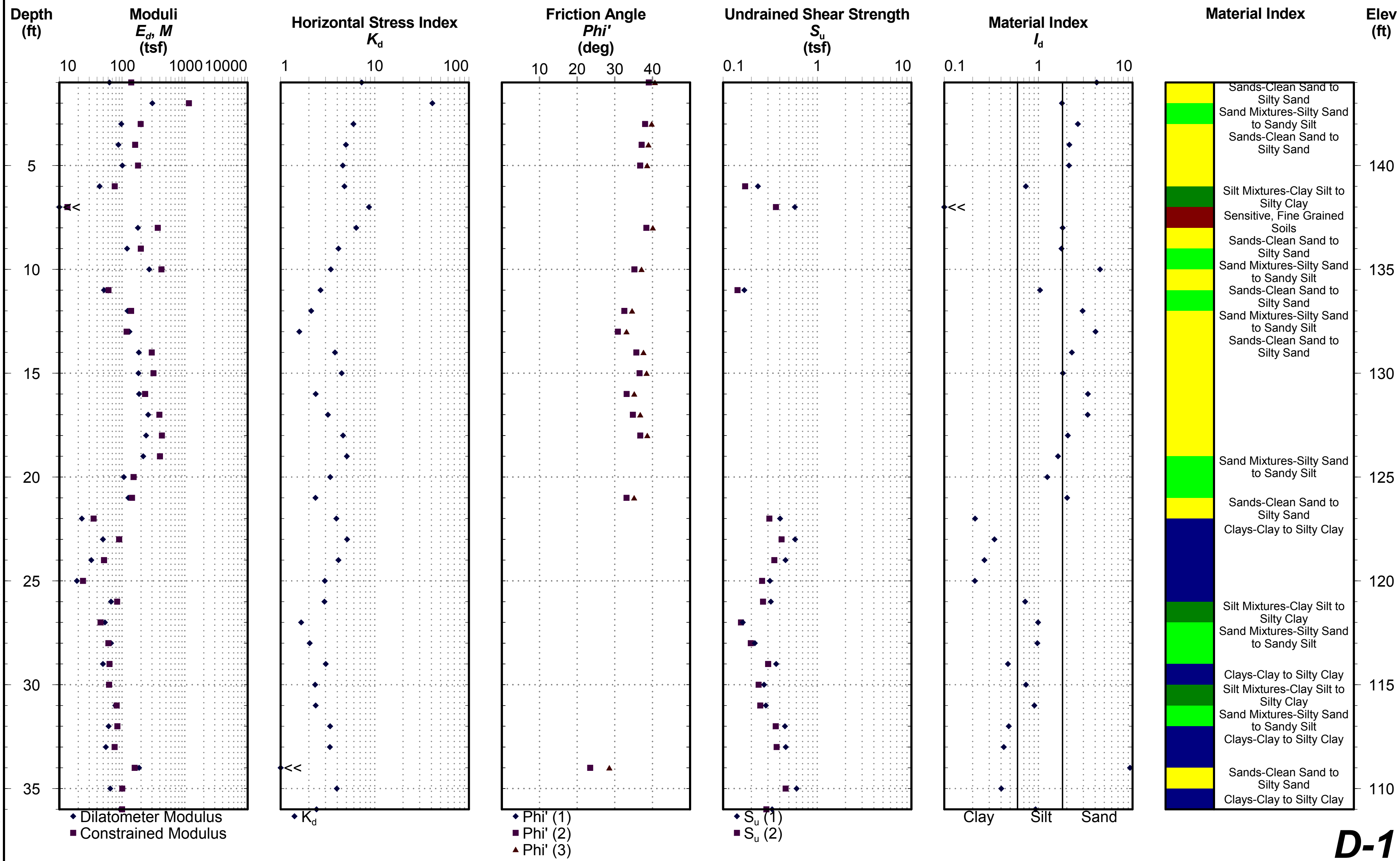
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Dilatometer Test

D-1

Date: Aug. 18, 2014
Estimated Water Depth: 7 ft
Rig/Operator: Marooka 300/Austin Fowler

Total Depth: 50.0 ft
Termination Criteria: Target Depth
Membrane Type: Stiff



D-1

DMT REPORT - DYNAMIC 1439-14-021 FLORENCE JUDICIAL CENTER.GPJ S&ME.GDT 9/16/14

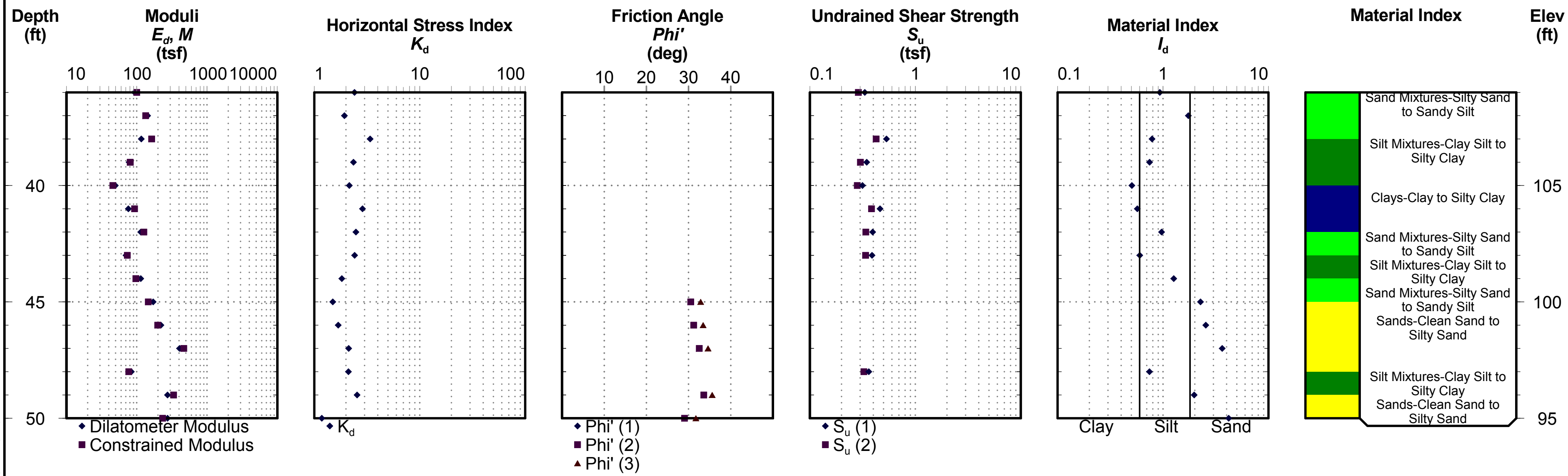
Florence Judicial Center
Florence, SC
S&ME Project No: 1439-14-021

Dilatometer Test

D-1

Date: Aug. 18, 2014
Estimated Water Depth: 7 ft
Rig/Operator: Marooka 300/Austin Fowler

Total Depth: 50.0 ft
Termination Criteria: Target Depth
Membrane Type: Stiff



DMT REPORT - DYNAMIC 1439-14-021 FLORENCE JUDICIAL CENTER.GPJ S&ME.GDT 9/16/14

D-1

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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

HAND AUGER BORING LOG NO. HA-1

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			**FILL** - SILTY CLAYEY SAND (SC-SM) - Dark brown, mostly fine to medium sand, some low plasticity fines, some gravel and concrete fragments, moist, loose.	7-7-7
				1				3-4-5
				2			CLAYEY SAND (SC) - Brown, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	5-6-5
				3				6-7-6
				4			Boring terminated at 4 feet	7-8-7

NOTES:

LOG OF HAND AUGER BORING HA-1

S&ME

HAND AUGER BORING LOG NO. HA-2

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			SILTY SAND (SM) - Dark brown and brown, mostly fine to medium sand, some low plasticity fines, moist, loose.	5-6-4
				1				4-5-4
				2				4-5-4
				3			CLAYEY SAND (SC) - Tan and orange, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	6-5-4
				4			Boring terminated at 4 feet	7-7-6

NOTES:

LOG OF HAND AUGER BORING HA-2

S&ME

HAND AUGER BORING LOG NO. HA-3

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			ASPHALT - 2 inches	4-7-5
							FILL - SILTY SAND (SM) - Red, mostly fine to medium sand, some low plasticity fines, moist, loose.	
				1			SILTY SAND (SM) - Dark brown to brown, mostly fine to medium sand, some low plasticity fines, moist, loose.	4-4-5
				2				5-7-6
				3				7-8-7
				4			Boring terminated at 4 feet	7-8-9

NOTES:

LOG OF HAND AUGER BORING HA-3

S&ME

HAND AUGER BORING LOG NO. HA-4

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			SILTY SAND (SM) - Dark brown and brown, mostly fine to medium sand, some low plasticity fines, moist, loose.	6-6-7
				1				7-6-7
				2				5-8-7
				3			CLAYEY SAND (SC) - Tan and orange, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	7-8-8
				4			Boring terminated at 4 feet	7-8-9

NOTES:

LOG OF HAND AUGER BORING HA-4

S&ME

HAND AUGER BORING LOG NO. HA-5

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			SILTY SAND (SM) - Dark brown and brown, mostly fine to medium sand, some low plasticity fines, moist, loose.	5-6-5
				1				4-7-7
				2				6-5-7
				3			CLAYEY SAND (SC) - Tan and orange, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	7-8-8
				4			Boring terminated at 4 feet	7-8-9

NOTES:

LOG OF HAND AUGER BORING HA-5

S&ME

HAND AUGER BORING LOG NO. HA-6

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			ASPHALT - 2 inches	
							Gravel - 6 inches	
				1			**FILL** - CLAYEY SAND (SC) - Red, mostly fine to medium sand, some low to medium plasticity fines, some dark clayey inclusions, moist, loose.	6-8-8
				2			CLAYEY SAND (SC) - Brown, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	7-8-9
				3				10-10-11
				4			Boring terminated at 4 feet	9-8-8

NOTES:

LOG OF HAND AUGER BORING HA-6

HAND AUGER BORING LOG NO. HA-7

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/14/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **P. Moody**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			ASPHALT - 2 inches	
							Gravel - 7 1/2 inches	
				1			**FILL** - SILTY SAND (SM) - Red, mostly fine to medium sand, some low plasticity fines, some dark clayey inclusions, moist, loose.	8-9-9
				2			CLAYEY SAND (SC) - Brown, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	10-10-10
				3				9-7-8
				4			Boring terminated at 4 feet	8-8-7

NOTES:

LOG OF HAND AUGER BORING HA-7

S&ME

HAND AUGER BORING LOG NO. HA-8

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/15/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **S. Herring**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			ASPHALT - 2 inches	7-6-7
							Gravel - 4 inches	
				1			**FILL** - CLAYEY SAND (SC) - Red, mostly fine to medium sand, some low to medium plasticity fines, some dark clayey inclusions, moist, loose.	8-7-7
				2			CLAYEY SAND (SC) - Brown, mostly fine to medium sand, some low to medium plasticity fines, moist, loose.	10-7-8
				3				7-8-7
				4			Boring terminated at 4 feet	8-7-6

NOTES:

LOG OF HAND AUGER BORING HA-8

S&ME

HAND AUGER BORING LOG NO. HA-9

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **TOB = 8.5'**

DATE DRILLED: **8/15/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Hand Auger**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **W. Kannon**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			See Test Pit Log TP-1	
				5			SANDY LEAN CLAY (CL) - Yellow-brown, mostly low to medium plasticity fines, some fine sand, moist.	
				7			CLAYEY SAND (SC) - Yellow-brown to orange, mostly fine to medium sand, some low to medium plasticity fines, moist.	
				8			CLAYEY SAND (SC) - Tan and pale orange, mostly fine to medium sand, some low to medium plasticity fines, wet.	∇
				9				

NOTES:

LOG OF HAND AUGER BORING HA-9

S&ME

HAND AUGER BORING LOG NO. TP-1

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/15/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Backhoe**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **W. Kannon**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			**FILL** - POORLY GRADED SAND WITH SILT (SP-SM) - Tan, mostly fine to medium sand, few low plasticity fines, moist.	
				1			COAL	
				2			SILTY SAND (SM) - Brown, mostly fine to medium sand, some low plasticity fines, moist.	
				3			CLAYEY SAND (SC) - Yellow-brown, mostly fine to medium sand, some low to medium plasticity fines, moist.	
				4				
							SANDY LEAN CLAY (CL) - Yellow-brown, mostly low to medium plasticity fines, some fine sand, moist.	
				5			Test pit terminated at 5 feet	

NOTES:

HAND AUGER LOG 1439-14-021 FLORENCE JUDICIAL CENTER HAB.GPJ WITH CPT.GDT 9/16/14

LOG OF HAND AUGER BORING TP-1

S&ME

HAND AUGER BORING LOG NO. TP-2

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/15/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Backhoe**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **W. Kannon**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0			**FILL** - POORLY GRADED SAND WITH SILT (SP-SM) - Tan, mostly fine to medium sand, few low plasticity fines, moist.	
				1			COAL	
				2			SILTY SAND (SM) - Brown, mostly fine to medium sand, some low plasticity fines, moist.	
				3			CLAYEY SAND (SC) - Yellow-brown, mostly fine to medium sand, some low to medium plasticity fines, moist.	
				4				
							SANDY LEAN CLAY (CL) - Yellow-brown, mostly low to medium plasticity fines, some fine sand, moist.	
				5			Test pit terminated at 5 feet	

NOTES:

LOG OF HAND AUGER BORING TP-2

S&ME

HAND AUGER BORING LOG NO. TP-3

PROJECT: **Florence Judicial Center**
PROJECT NO: **1439-14-021**
PROJECT LOCATION: **Florence, SC**

WATER LEVEL: **Not encountered**

DATE DRILLED: **8/15/14**
DRILLING CONTRACTOR: **S&ME**
DRILLING METHOD: **Backhoe**

GROUND SURFACE ELEVATION: **Unknown**
LOGGED BY: **W. Kannon**

This log is part of the report prepared for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE NUMBER	SAMPLE ADVANCE (ft.)	OVA (ppm)	ELEVATION (ft.)	DEPTH (ft.)	USCS	GRAPHIC SYMBOL	DESCRIPTION	DCP (blows per increment)
				0–1			Concrete - Thin soil layer about 1/2 to 1 inch thick	
				1–3			**FILL** - SILTY SAND (SM) - Dark brown-gray, mostly fine to medium sand, few low plasticity fines, some brick and concrete fragments, glass, and wood, wet.	
				3–4			SILTY SAND (SM) - Brown, mostly fine to medium sand, some low plasticity fines, moist.	
				4–5			SANDY LEAN CLAY (CL) - Yellow-brown, mostly low to medium plasticity fines, some fine sand, moist.	
				5			Test pit terminated at 5 feet	

NOTES:

LOG OF HAND AUGER BORING TP-3

S&ME

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

JOB NAME :	Florence Judicial Center						
JOB NO. :	1439-14-021	**REPORT NO. :**		**TEST DATE :**	08/15/14	**INVESTIGATOR :**	WR/SM
TEST PIT NO. :	DRI-1	**DEPTH / ELEV. :**	- 5 feet			**REVIEWED BY :**	WK
TEST PIT LOCATION :	Detention Pond Area						
SOIL DESCRIPTION :	Yellow-brown, Sandy Lean Clay (CL)						

CONSTANTS	AREA CM^2	DEPTH OF LIQUID CM	MARIOTTE TUBE NO.	ΔVOLUME / ΔH CM^3 / CM
INNER RING	699.1	10.16	**1**	1
ANNULAR SPACE	2105.0	10.16	**2**	1

READING NO.		DATE	TIME	ELAPSED TIME	FLOW READINGS				LIQUID TEMP.	INFILTRATION RATE		REMARKS
					INNER RING		ANNULAR SPACE			INNER	ANNULAR	
			HR:MIN:SEC	MINUTES	READING CM	FLOW CM^3	READING CM	FLOW CM^3	^{O}C	IN. / HOUR	IN. / HOUR	GROUND TEMP. 28 ^{O}C
1	S	08/15/14	10:00						24			
	E		12:30	150		150		400		0.03	0.03	
2	S		12:35									
	E		3:05	150		125		330		0.03	0.02	
3	S		3:10									
	E		5:40	150		100		300		0.02	0.02	

APPENDIX C

SUMMARY OF LABORATORY PROCEDURES

LABORATORY TEST RESULTS

SUMMARY OF LABORATORY PROCEDURES

Examination of Recovered Soil Samples

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

Moisture Content Testing of Soil Samples by Oven Drying

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

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

Liquid and Plastic Limits Testing

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

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

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

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

Percent Fines Determination of Samples

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

Compaction Tests of Soils Using Modified Effort

Soil placed as engineering fill is compacted to a dense state to obtain satisfactory engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure the required compaction and water contents are achieved. Test procedures generally followed those described by ASTM D 1557, "*Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 lbf/ft*^{3}*).*"

The relationship between water content and the dry unit weight is determined for soils compacted in a 4 inch diameter molds with a 10 lbf rammer dropped from a height of 12 inches, producing a compactive effort of 56,000 lbf/ft^3.

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

Laboratory California Bearing Ratio Tests of Compacted Samples

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

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

Form No: TR-D2216-T265-1

Revision No. 0

Revision Date: 02/22/08

Laboratory Determination of Water Content

S&ME

ASTM D 2216 ☑ *AASHTO T 265* ☐ ***Quality Assurance***

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

Project #:	**1439-14-021**	Report Date:	08/20/14
Project Name:	Florence Co. Judicial Center	Test Date(s):	08/19/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		
Sample by:	PM & SH	Sample Date(s):	08/15/14
Sampling Method:	Grab	Drill Rig :	--

Method: **A** (1%) ☐ **B** (0.1%) ☑ *Balance ID.* ***24496*** *Calibration Date:* ***11/5/13***

Boring No.	Sample No.	Sample Depth	Tare #	Tare Weight	Tare Wt.+ Wet Wt	Tare Wt. + Dry Wt	Water Weight	Percent Moisture	Note
		ft. or m.		grams	grams	grams	grams	%	
HA-1,2,4-7	BULK 1	0.5 - 2	54	0.00	677.40	597.80	79.60	**13.3%**	
HA-3, 8	BULK 2	1 - 2	30	0.00	321.50	297.70	23.80	**8.0%**	
HA-1	2	1 - 2	69	0.00	325.60	289.90	35.70	**12.3%**	
HA-5	2	1 - 2	00	0.00	389.10	342.60	46.50	**13.6%**	
HA-9	3	8 - 9	89	0.00	178.10	168.10	10.00	**5.9%**	

Notes / Deviations / References

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

W. Kannon	WDK	Project Engineer	8/29/2014
Technical Responsibility	*Signature*	*Position*	*Date*

Form No: TR-D1140-1
Revision No. 0
Revision Date: 10/26/07

Material Finer than the #200 Sieve

ASTM D1140

Quality Assurance

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

Project #:	**1439-14-021**	Report Date:	08/20/14
Project Name:	Florence Judical Center	Test Date(s):	08/19/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		
Sample by:	PM & SH	Sample Date(s):	08/15/14
Sampling Method:	Grab	Drill Rig :	Hand Auger

Method; A ☐ **B** ☑ Soaked ☑ Soak Time 16 hrs.

Boring #	Sample #	Sample Depth	Tare #	Tare Weight	Tare Wt.+ Wet Wt	Tare Wt. + Dry Wt	Tare Wt. + Dry Wt. after Wash	% Passing #200
		ft.		grams	grams	grams	grams	%
BULK	1	0.5 - 2	2	0.00	0.00	117.40	82.90	**29.4%**
BULK	2	0.5 - 2	89	0.00	0.00	115.70	88.70	**23.3%**
HA-1	2	1 - 2	69	0.00	0.00	124.30	94.80	**23.7%**
HA-5	2	1 - 2	00	0.00	0.00	132.70	98.30	**25.9%**
HA-9	3	8 - 9	89	0.00	0.00	168.10	143.90	**14.4%**

Balance ID. ***24496*** *Calibration Date:* ***11/5/13*** *#200 Sieve* ***24527*** *Calibration Date:* ***1/10/14***

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

W. Kannon	WDK	Project Engineer	8/29/2014
Technical Responsibility	*Signature*	*Position*	*Date*

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

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

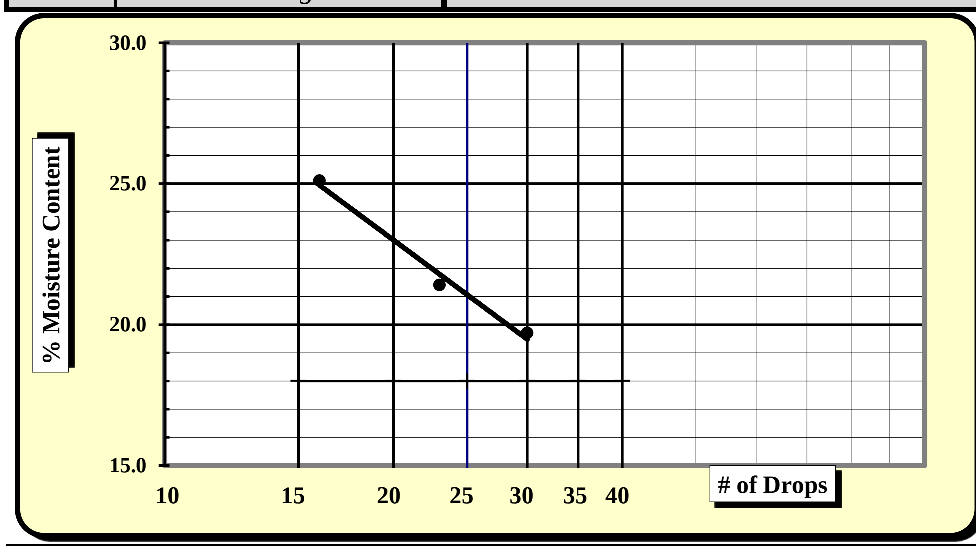
ASTM D 4318 ☒ *AASHTO T 89* ☐ *AASHTO T 90* ☐ ***Quality Assurance***

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

Project #:	**1439-14-021**	Report Date:	08/27/14
Project Name:	Florence Co. Judicial Center	Test Date(s)	08/25/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		
Boring #:	HA-1 Sample #:	Sample Date:	08/15/14
Location:	HA-1	Depth:	1'-2'
Sample Description:	Dark brown Silty-Clayey Sand (SC-SM)		

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	24496	11/5/2013	Grooving tool	24511	1/4/2014
LL Apparatus	24510	1/4/2014			
Oven	24456	5/10/2014			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	50	51	52				53	54	
A	Tare Weight	15.13	15.26	15.42				7.07	7.20	
B	Wet Soil Weight + A	22.31	21.90	21.01				11.21	11.54	
C	Dry Soil Weight + A	21.13	20.73	19.89				10.61	10.89	
D	Water Weight (B-C)	1.18	1.17	1.12				0.60	0.65	
E	Dry Soil Weight (C-A)	6.00	5.47	4.47				3.54	3.69	
F	% Moisture (D/E)*100	19.7%	21.4%	25.1%				16.9%	17.6%	
N	# OF DROPS	30	23	16						
LL	LL = **F** * FACTOR									
Ave.	*Average*							**17.3%**		



One Point Liquid Limit

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

NP, Non-Plastic	☐
Liquid Limit	**21**
Plastic Limit	**17**
Plastic Index	**4**
Group Symbol	**SC-SM**
Multipoint Method	☑
One-point Method	☐

Wet Preparation ☐ Dry Preparation ☑ Air Dried ☐

Notes / Deviations / References:

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

W. Kannon	WDK	Project Engineer	9/8/2014
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D4318-T89-90

Revision No. 0

Revision Date: 11/20/07

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

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

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

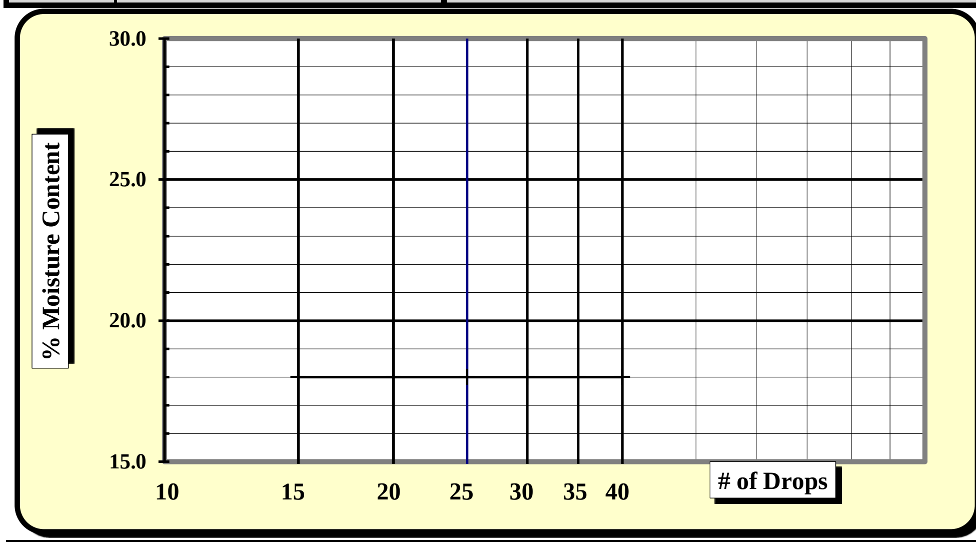
Project #:	**1439-14-021**	Report Date:	08/27/14
Project Name:	Florence Co. Judicial Center	Test Date(s)	08/25/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		

Boring #:	HA-5	Sample #:		Sample Date:	08/15/14
Location:	HA-5			Depth:	1'-2'

Sample Description: Dark brown SiltySand (SM)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	24496	11/5/2013	Grooving tool	24511	1/4/2014
LL Apparatus	24510	1/4/2014			
Oven	24456	5/10/2014			

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



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

NP, Non-Plastic	☑
Liquid Limit	**--**
Plastic Limit	**NP**
Plastic Index	**--**
Group Symbol	**SM**

Multipoint Method ☐

One-point Method ☐

Wet Preparation ☐ Dry Preparation ☑ Air Dried ☐

Notes / Deviations / References:

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

W. Kannon	WDK	Project Engineer	9/8/2014
Technical Responsibility	*Signature*	*Position*	*Date*

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

Sieve Analysis of Soils

S&ME

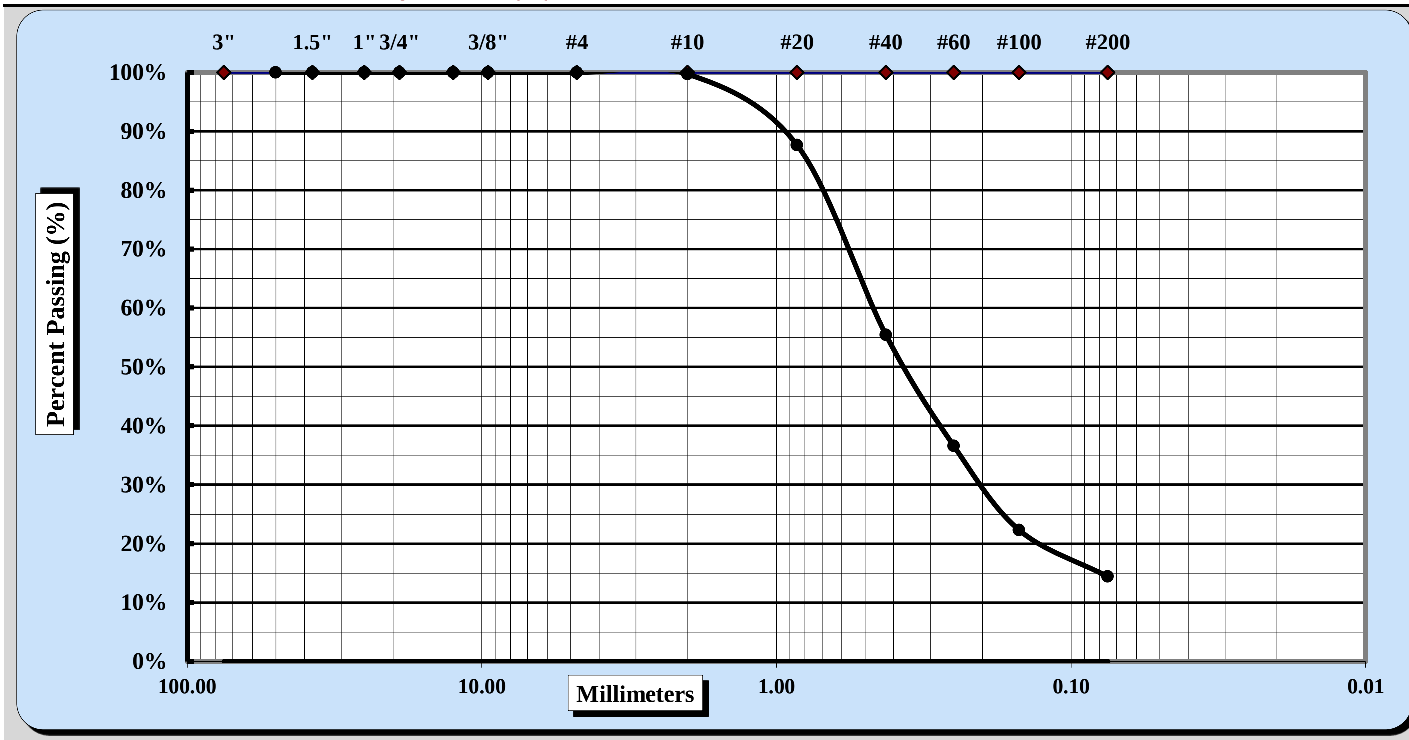
ASTM D 422

Quality Assurance

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

Project #:	**1439-14-021**	Report Date:	9/16/14
Project Name:	Florence Judical Center	Test Date(s):	9/15/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Maint Street, Columbia, SC		
Lab #:	2068	Sample Date:	8/15/14
Location:	HA-9	Depth:	8'-9'

Sample Description: Orange/tan Clayey Sand (SC)



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

Maximum Particle Size	2.000	Coarse Sand	0.3%	Fine Sand	41.0%
Gravel	0.0%	Medium Sand	44.3%	Silt & Clay	14.4%
Liquid Limit	ND	Plastic Limit	ND	Plastic Index	ND
Specific Gravity	N/D			Moisture Content	5.9%
Coarse Sand	0.3%	Medium Sand	44.3%	Fine Sand	41.0%

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

Notes / Deviations / References: Debris was removed from the sample prior to testing.

W. Kannon	WDK	Project Engineer	9/15/2014
Technical Responsibility	*Signature*	*Position*	*Date*

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

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

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

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

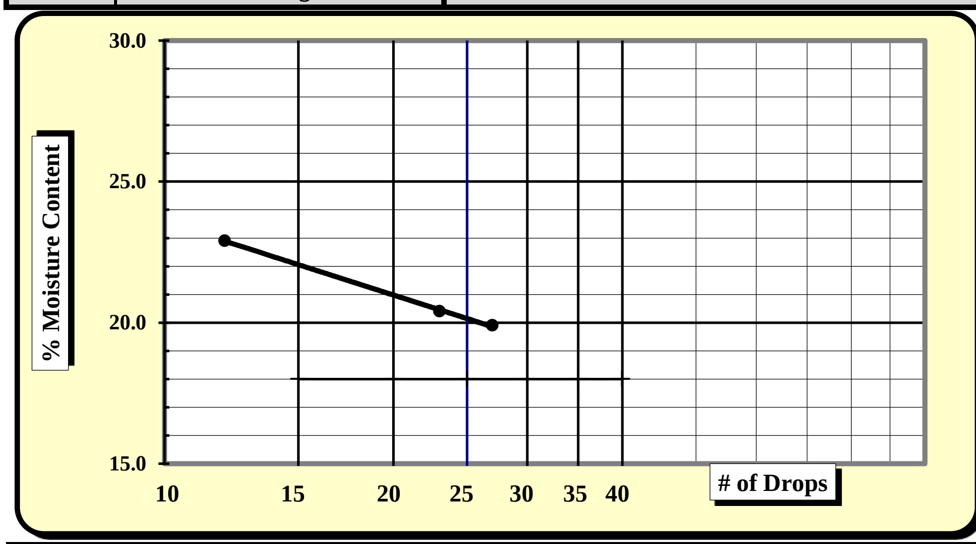
Project #:	**1439-14-021**	Report Date:	08/20/14
Project Name:	Florence Judical Center	Test Date(s)	08/19/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		

Boring #:	HA-1,2,4,5,6,7	Sample #:	2053	Sample Date:	08/15/14
Location:	BULK #1			Depth:	1'-2'

Sample Description: Dark Grey-brown Silty Sand (SM)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	24496	11/5/2013	Grooving tool	24511	1/4/2014
LL Apparatus	24510	1/4/2014			
Oven	24456	5/10/2014			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	50	51	52				53	54	
A	Tare Weight	13.78	15.48	15.06				19.40	15.13	
B	Wet Soil Weight + A	21.91	22.55	21.13				23.06	18.98	
C	Dry Soil Weight + A	20.56	21.35	20.00				22.55	18.43	
D	Water Weight (B-C)	1.35	1.20	1.13				0.51	0.55	
E	Dry Soil Weight (C-A)	6.78	5.87	4.94				3.15	3.30	
F	% Moisture (D/E)*100	19.9%	20.4%	22.9%				16.2%	16.7%	
N	# OF DROPS	27	23	12				*Moisture Contents determined by ASTM D 2216*		
LL	LL = **F** * FACTOR									
Ave.	*Average*							**16.5%**		



One Point Liquid Limit

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

NP, Non-Plastic	☐
Liquid Limit	**20**
Plastic Limit	**17**
Plastic Index	**3**
Group Symbol	**SM**
Multipoint Method	☑
One-point Method	☐

Wet Preparation ☐ Dry Preparation ☑ Air Dried ☐

Notes / Deviations / References:

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

W. Kannon	WDK	Project Engineer	8/29/2014
Technical Responsibility	*Signature*	*Position*	*Date*

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

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

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

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

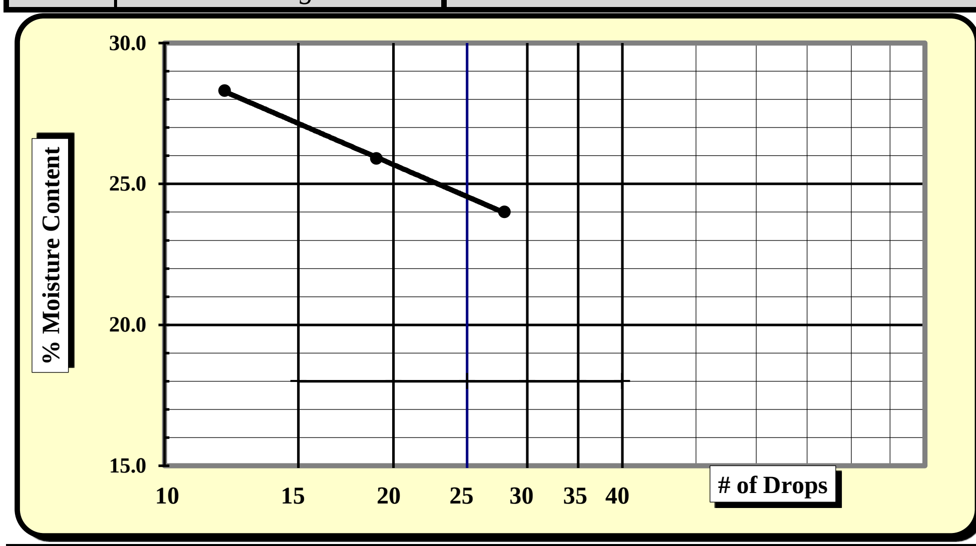
Project #:	**1439-14-021**	Report Date:	08/20/14
Project Name:	Florence Judical Center	Test Date(s)	08/19/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		

Boring #:	BULK #2	Sample #:	2054	Sample Date:	08/15/14
Location:				Depth:	1'-2'

Sample Description: Red-brown Clayey Sand (SC)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	24496	11/5/2013	Grooving tool	24511	1/4/2014
LL Apparatus	24510	1/4/2014			
Oven	24456	5/10/2014			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	50	51	52				53	54	
A	Tare Weight	13.83	13.70	13.83				13.82	15.09	
B	Wet Soil Weight + A	21.93	19.09	20.76				17.44	18.79	
C	Dry Soil Weight + A	20.36	17.98	19.23				16.99	18.32	
D	Water Weight (B-C)	1.57	1.11	1.53				0.45	0.47	
E	Dry Soil Weight (C-A)	6.53	4.28	5.40				3.17	3.23	
F	% Moisture (D/E)*100	24.0%	25.9%	28.3%				14.2%	14.6%	
N	# OF DROPS	28	19	12				*Moisture Contents determined by ASTM D 2216*		
LL	LL = **F** * FACTOR									
Ave.	*Average*							**14.4%**		



One Point Liquid Limit

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

NP, Non-Plastic	☐
Liquid Limit	**25**
Plastic Limit	**14**
Plastic Index	**11**
Group Symbol	**SC**
Multipoint Method	☑
One-point Method	☐

Wet Preparation ☐ Dry Preparation ☑ Air Dried ☐

Notes / Deviations / References:

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

W. Kannon	WDK	Project Engineer	8/29/2014
Technical Responsibility	*Signature*	*Position*	*Date*

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

Moisture - Density Report

S&ME

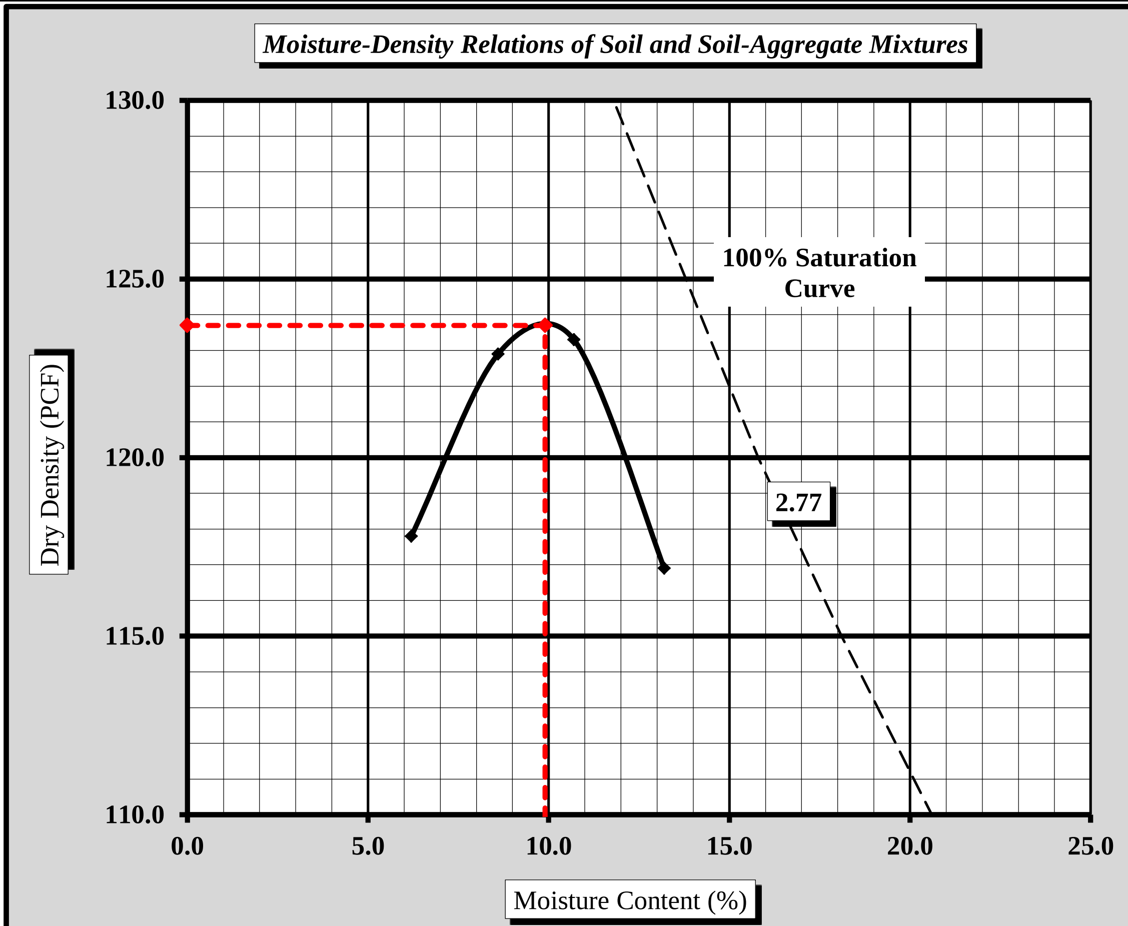
Quality Assurance

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

S&ME Project #:	**1439-14-021**	Report Date:	8/28/14
Project Name:	Florence County Judicial Center	Test Date(s):	8/21/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		
Lab #	2054	Sample Date:	8/15/2014
Location:	HA-3, 8: BULK #2	Depth:	1-2'
Sample Description:	Reddish Brown Clayey Sand (SC)		

Maximum Dry Density 123.7 PCF. **Optimum Moisture Content 9.8%**

ASTM D 698 - - Method A



Soil Properties

Natural Moisture Content	**8.0%**
Specific Gravity of Soil	**--**
Liquid Limit	**25**
Plastic Limit	**14**
Plastic Index	**11**

% Passing

3/4"	100.0%
3/8"	100.0%
#4	
#10	
#40	
#60	
#200	23.3%

Oversize Fraction

Bulk Gravity	
% Moisture	--
% Oversize	#DIV/0!
MDD	
Opt. MC	

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

References / Comments / Deviations:

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 698: Laboratory Compaction Characteristics of Soil Using Standard Effort

W. Kannon	WDK	Dept. Supervisor	8/29/2014
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D1833-T193-3
Revision No. 0
Revision Date: 2/6/08
Modified

CBR (California Bearing Ratio) of Laboratory Compacted Soil

ASTM D 1883

Quality Assurance

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

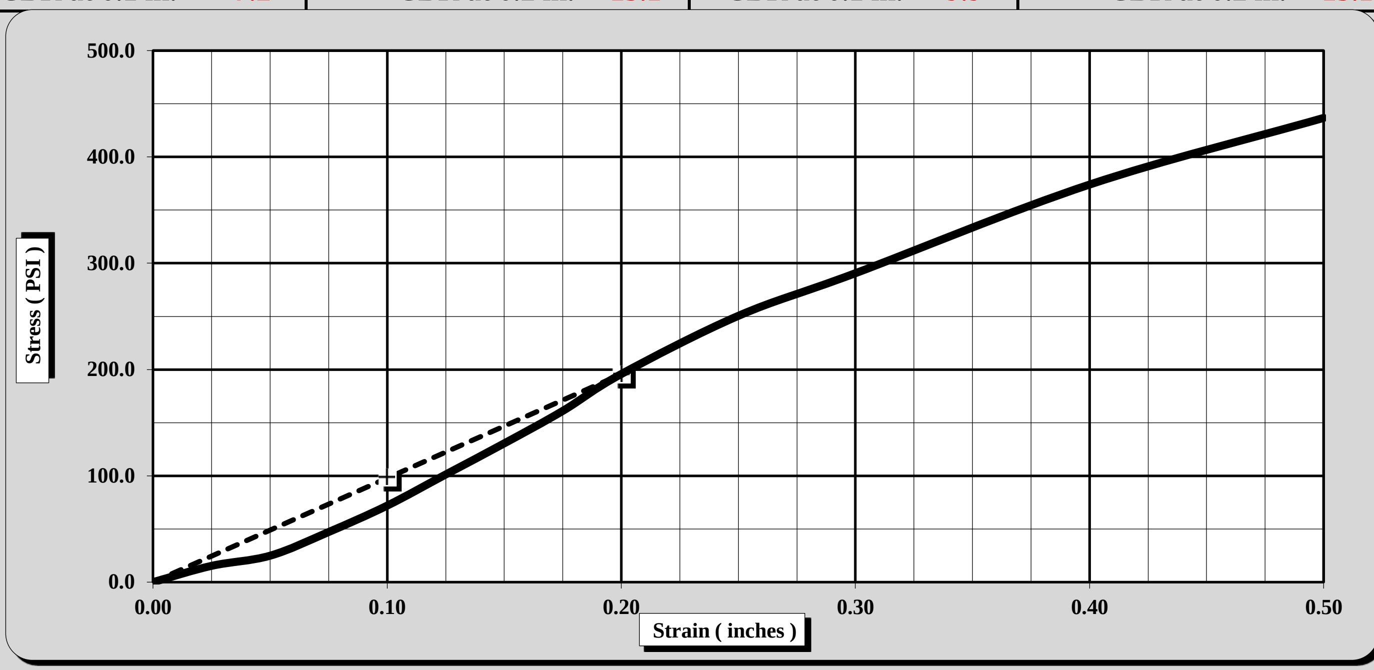
Project #:	**1439-14-021**	Report Date:	8/27/14
Project Name:	Florence Judical Center	Test Date(s)	8/26/14
Client Name:	Stevens & Wilkinson		
Client Address:	1501 Main Street; Columbia, SC		
Boring #	HA-3, 8	Lab# 2054	Sample Date: 8/15/14
Location:	BULK 2		Depth: 1' - 2'

Sample Description: Red-brown Clayey Sand (SC)

ASTM D 698 *Method A* Maximum Dry Density: **123.7** **PCF** Optimum Moisture Content: **9.8%**

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

Uncorrected CBR Values				**Corrected CBR Values**			
CBR at 0.1 in.	**7.2**	**CBR at 0.2 in.**	**13.1**	**CBR at 0.1 in.**	**9.9**	**CBR at 0.2 in.**	**13.1**



CBR Sample Preparation:

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

Before Soaking		*After Soaking*	
Compactive Effort (Blows per Layer)	28	Final Dry Density (PCF)	-3.8
Initial Dry Density (PCF)	117.5	Average Final Moisture Content	-3488.4%
Moisture Content of the Compacted Specimen	10.0%	Moisture Content (top 1" after soaking)	-2624.4%
Percent Compaction	95.0%	Percent Swell	0.1%

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

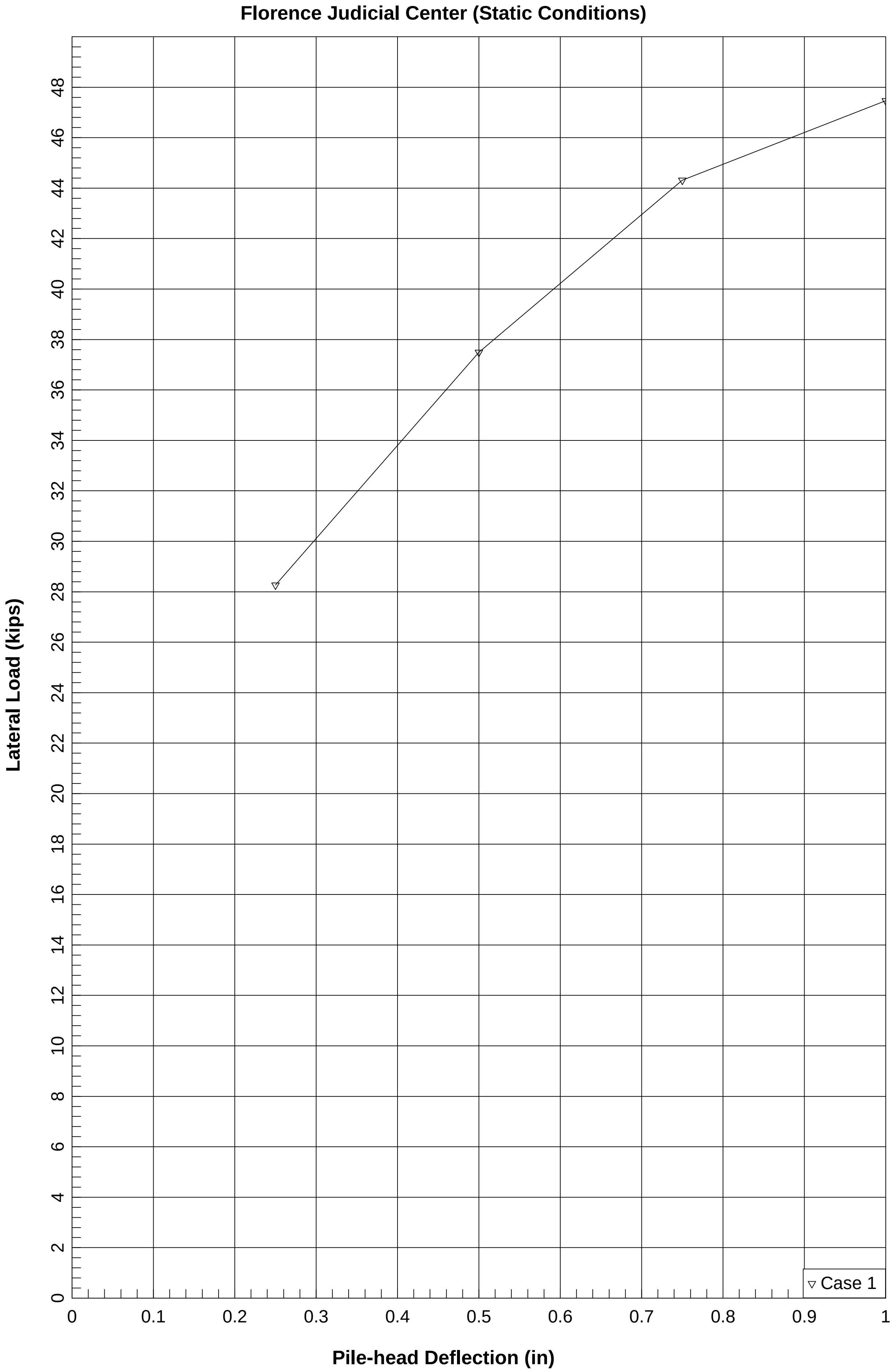
Notes/Deviations/References: Liquid Limit: ASTM D 4318, Classification: ASTM D 2487

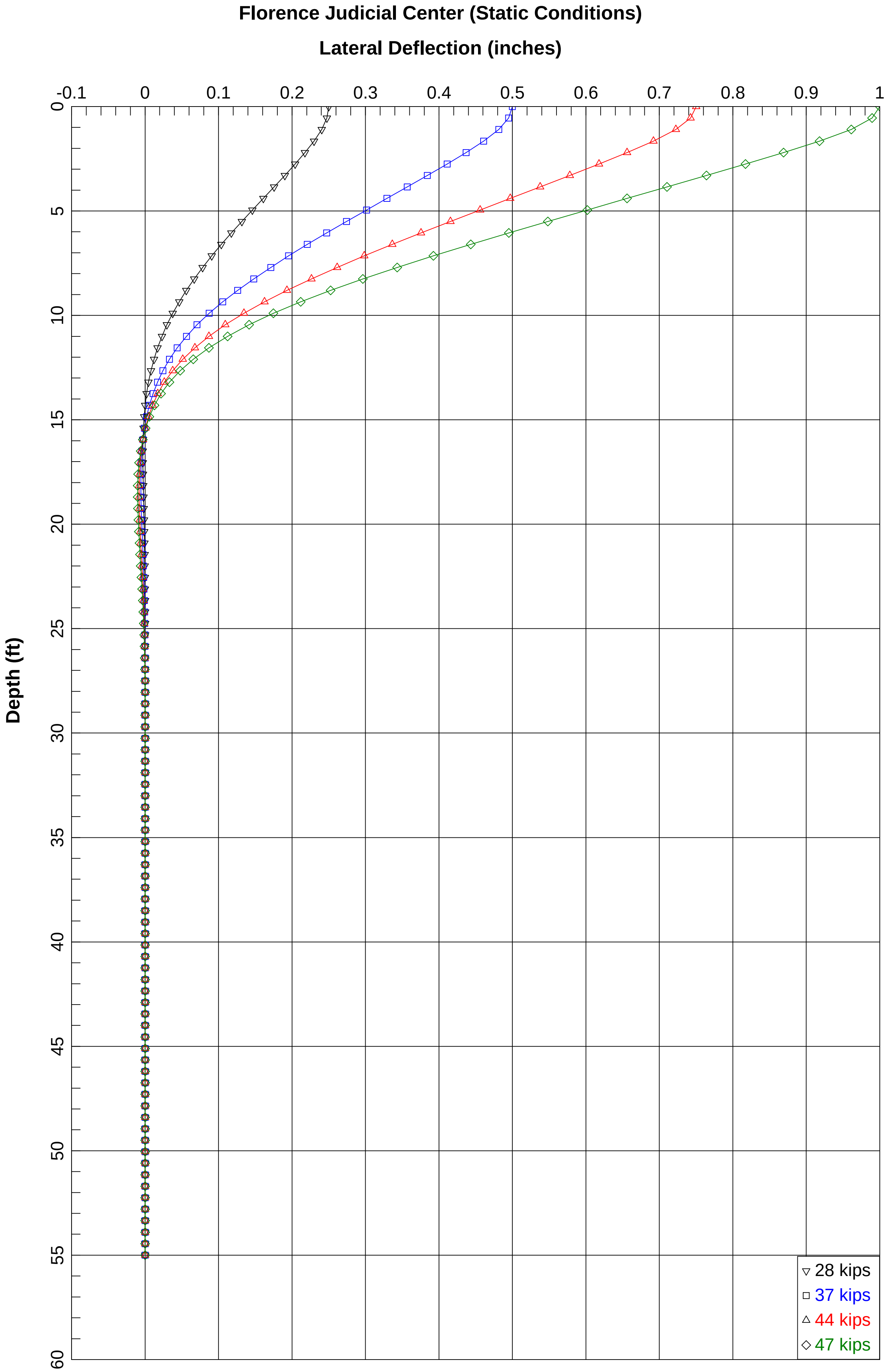
W. Kannon	WDK	Project Engineer	9/8/2014
Technical Responsibility	*Signature*	*Position*	*Date*

-

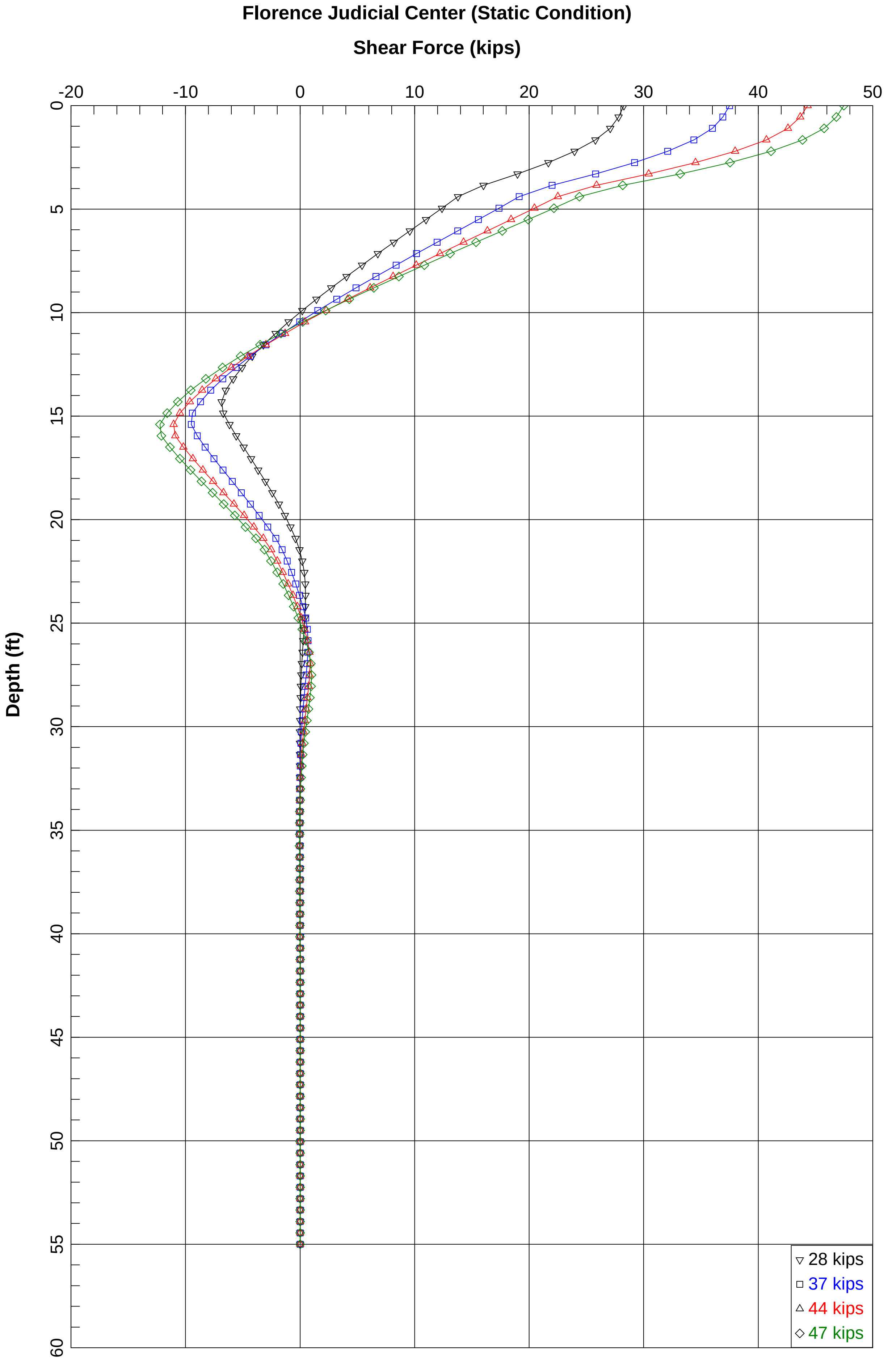
APPENDIX D

L-PILE RESULTS

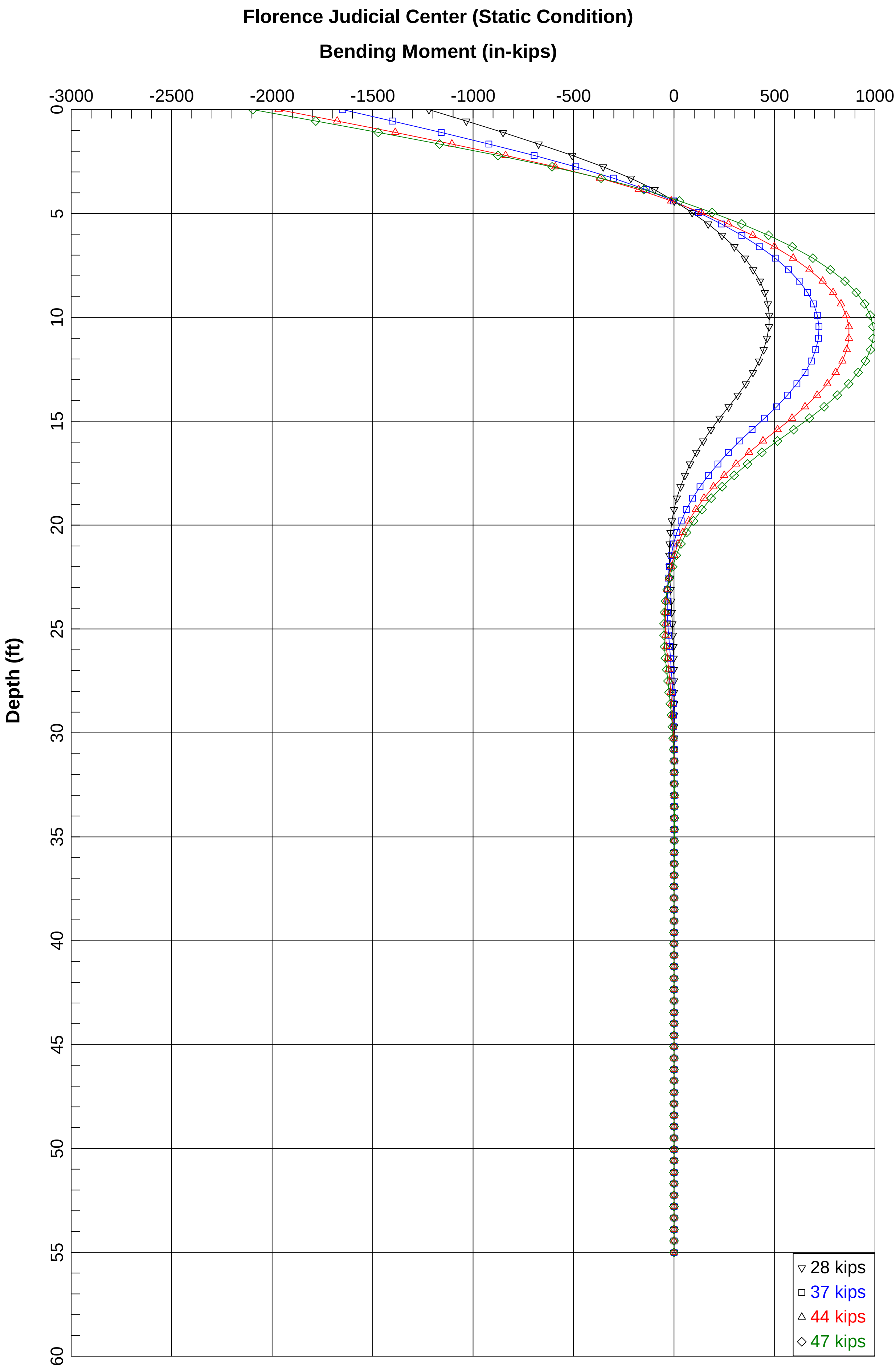

Florence Judicial Center (Static Conditions)
Lateral Load (kips)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Pile-head Deflection (in)
Case 1



Florence Judicial Center (Static Conditions)
Lateral Deflection (inches)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0
5
10
15
20
25
30
35
40
45
50
55
60
Depth (ft)
28 kips
37 kips
44 kips
47 kips



Florence Judicial Center (Static Condition)
Shear Force (kips)
-20
-10
0
10
20
30
40
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
28 kips
37 kips
44 kips
47 kips



Florence Judicial Center (Static Condition)
Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
28 kips
37 kips
44 kips
47 kips

October 10, 2015

Florence County
180 N. Irby Street, MSC-G
Florence, South Carolina 29501

Attention: Ms. Suzanne King

Reference: **Report of Supplemental Geotechnical Exploration – Stage 1**
Florence County Judicial Center
Florence, South Carolina
S&ME Project No. 1439-15-029

Dear Ms. King:

S&ME, Inc. has completed this *Report of Supplemental Geotechnical Exploration – Stage 1* for the above referenced project after receiving authorization to proceed from Mr. Patrick Fletcher, Procurement Officer in the form of a Purchase Order No. 098739, dated September 14, 2015. Our supplemental analysis was conducted in general accordance with our Proposal No. 14-1500693, dated September 10, 2015.

Updated Project Information

Updated project information was provided during a telephone conversation between Ms. Michelle Motchos, P.E. (Stevens & Wilkinson) and Mr. Will Kannon, P.E., (S&ME) on September 3, 2015. Additional information was provided to Mr. Kannon by Mr. Patrick Fletcher (Florence County Procurement Officer) during a conversation on September 8, 2015. During this conversation Mr. Fletcher requested that S&ME submit a proposal to Florence County for supplemental exploration.

The proposed project site is located on North Irby Street across the street (west of) the existing Florence City-County Complex in Florence, South Carolina. The project site is currently developed with several commercial buildings fronting N. Irby Street in the eastern portion of the site. The remainder of the site is mostly covered in asphaltic pavements. As of the date of this report, demolition of the existing structures and pavements has not yet been completed and the site conditions remain as they were during our previous exploration.

Previous Analysis

We previously performed down-hole seismic testing within the CPT boreholes to measure the shear wave velocity to a maximum depth of about 55 feet. To determine the average shear wave velocity to a depth of 100 feet, as is required by ASCE 7-10, Section 20.3.3, we interpolated the data between depths of 55 feet to 100 feet. An average shear wave velocity of 1,000 feet per second (fps) was obtained using this methodology, so it was determined that seismic Site Class D applies. This resulted in short period (0.2 sec) response acceleration S_{DS} = 0.51g, and long (1-second) period response acceleration S_{D1} = 0.26g. Using these coefficients, it was determined that Seismic Design Category D was appropriate.

Based on a telephone conversation with Ms. Michelle Motchos (Stevens & Wilkinson) on September 3, 2015, we understand that the design team desired to see if a Design Category C would be possible to obtain with additional testing.

To check the feasibility of obtaining a Design Category C, we performed preliminary calculations using the "general procedure" defined in the Code, and we found that *only if* Site Class C can be applied, S_{DS} would improve to 0.41g (which falls within the Design Category C range); however, S_{D1} would be 0.21g, which is still higher than the maximum value of 0.20g that is allowed for Design Category C. Therefore, in order to have the possibility of achieving a Seismic Design Category C, a Site-Specific Seismic Response Analysis (SSRA) would have to be performed. But this step would only be taken if in fact the Site Class could be upgraded from D to C. Therefore, we proposed a two-stage process of further evaluation; stage one was to measure shear wave velocities at the site to see if 1,200 fps could be achieved, thereby upgrading the Site Class to C; if not, we planned to stop the process after this stage of analysis. If 1,200 fps shear wave velocity was achieved, then stage two would be to perform the SSRA.

❖ Field Exploration

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. Two separate arrays (SW-1 and SW-2) were laid out by our field personnel and test locations are noted on Figure 1 in the appendix.

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 5 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 at each test location. 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.

❖ Results

Based on the results of the shear wave velocity testing, test location SW-1 measured an average velocity of 802 feet per second (fps) over a depth of about 100 feet and SW-2 measured an average velocity of 840 fps over a depth of about 100 feet. Therefore, the average velocity of the site was estimated to be about 820 fps. This is slightly lower than the shear wave velocity of 1,000 fps that was obtained from the interpolated values during the original exploration, but is still well within the range designated for Site Class D. The shear wave velocity profiles are presented in the appendix as Figures 2 and 3.

Based on the information presented in Table 20.3-1 of ASCE 7-10, a velocity of at least 1,200 fps must be obtained in order for the soil profile to be upgraded from seismic Site Class D to Site Class C. Therefore the seismic Site Class for this site remains Site Class D, as was originally determined, and the seismic parameters presented in our original report still apply.

Since we were unable to obtain an upgraded site classification, Stage 2 of our proposed scope of services, the site-specific seismic response analysis (SSRA), will not be performed, and the Seismic Design Category for this project remains D.

❖ Limitations of Report

This supplemental 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 supplemental subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our test 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; this may result in an additional fee for services.

In the event that any changes in the nature, design, or location of the structures, pavements, or other appurtenances are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions are modified or verified in writing by the submitting engineers.

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

Closure

S&ME, Inc. appreciates the opportunity to provide geotechnical consultation services on this project. If you have any questions concerning this report, please do not hesitate to contact us.

Very truly yours,

S&ME, Inc.

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION S&ME, INC. No. C00473

William D. Kannon, P.E.
Project Engineer

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

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 21248 10/10/15 RON FOREST JR.

Appendix:
- Figure 1 – Test Location Sketch
- Figure 2 – Shear Wave Velocity Profile SW-1
- Figure 3 – Shear Wave Velocity Profile SW-2

Appendix

SCALE:	NTS	
CHECKED BY:	RPF	
DRAWN BY:	WDK	
DATE:	October 2015	

S&ME

TEST LOCATION SKETCH
Florence Judicial Center
Florence, South Carolina

JOB NO. 1439-15-029

FIGURE NO 1

FIGURE 2: Shear Wave Velocity Profile SW-1
Florence Judicial Building
Florence, South Carolina
1439-15-029

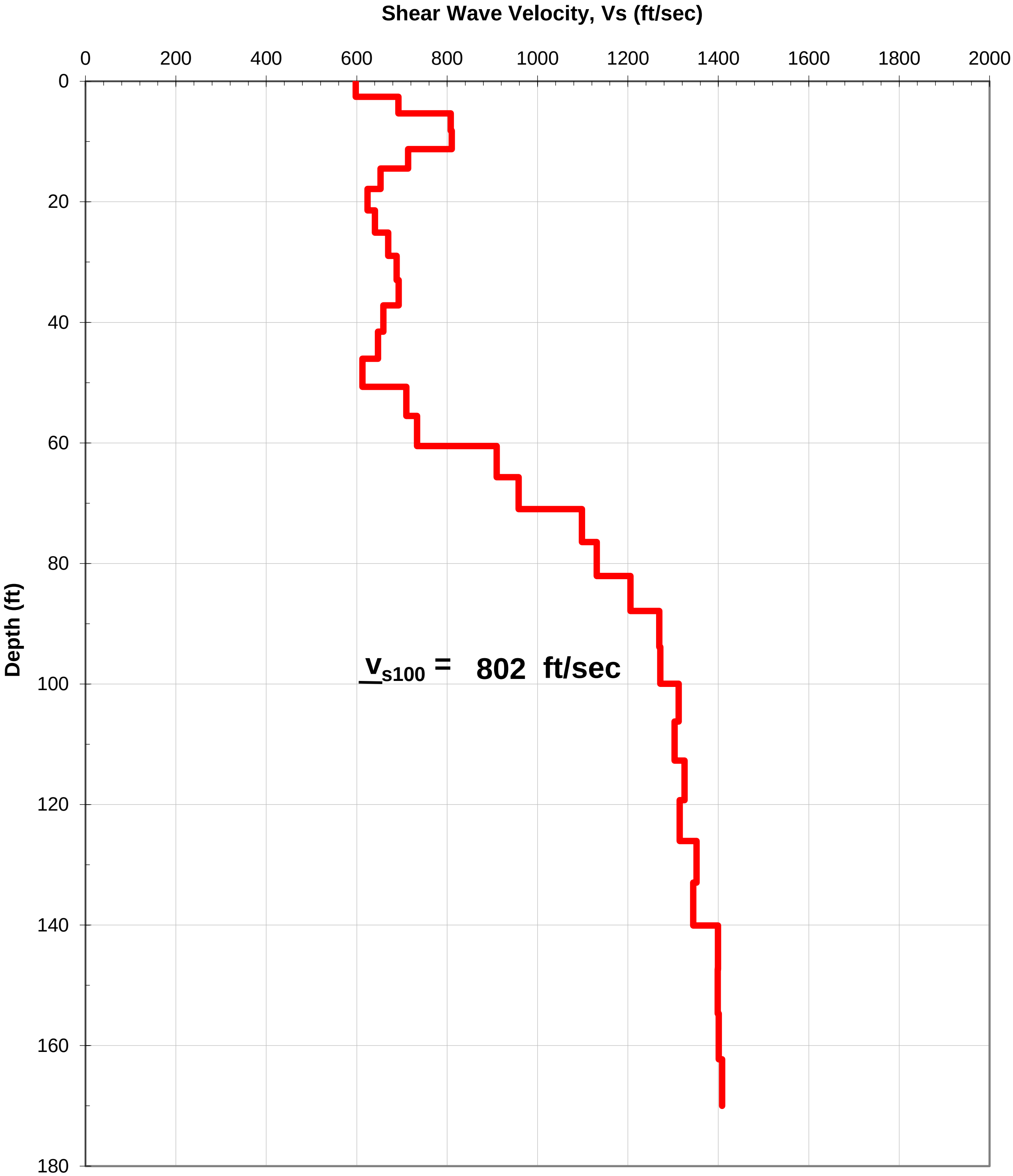
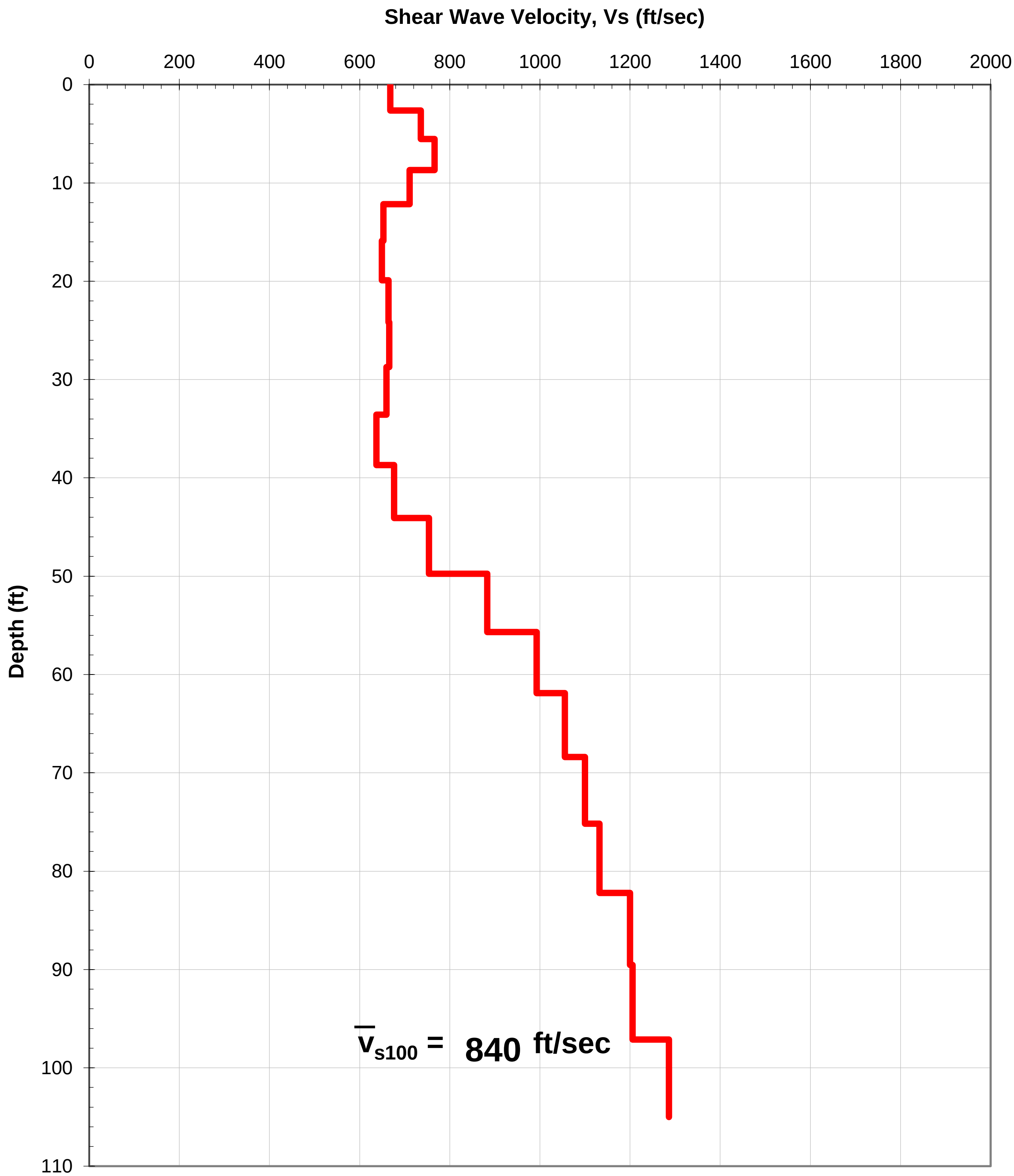
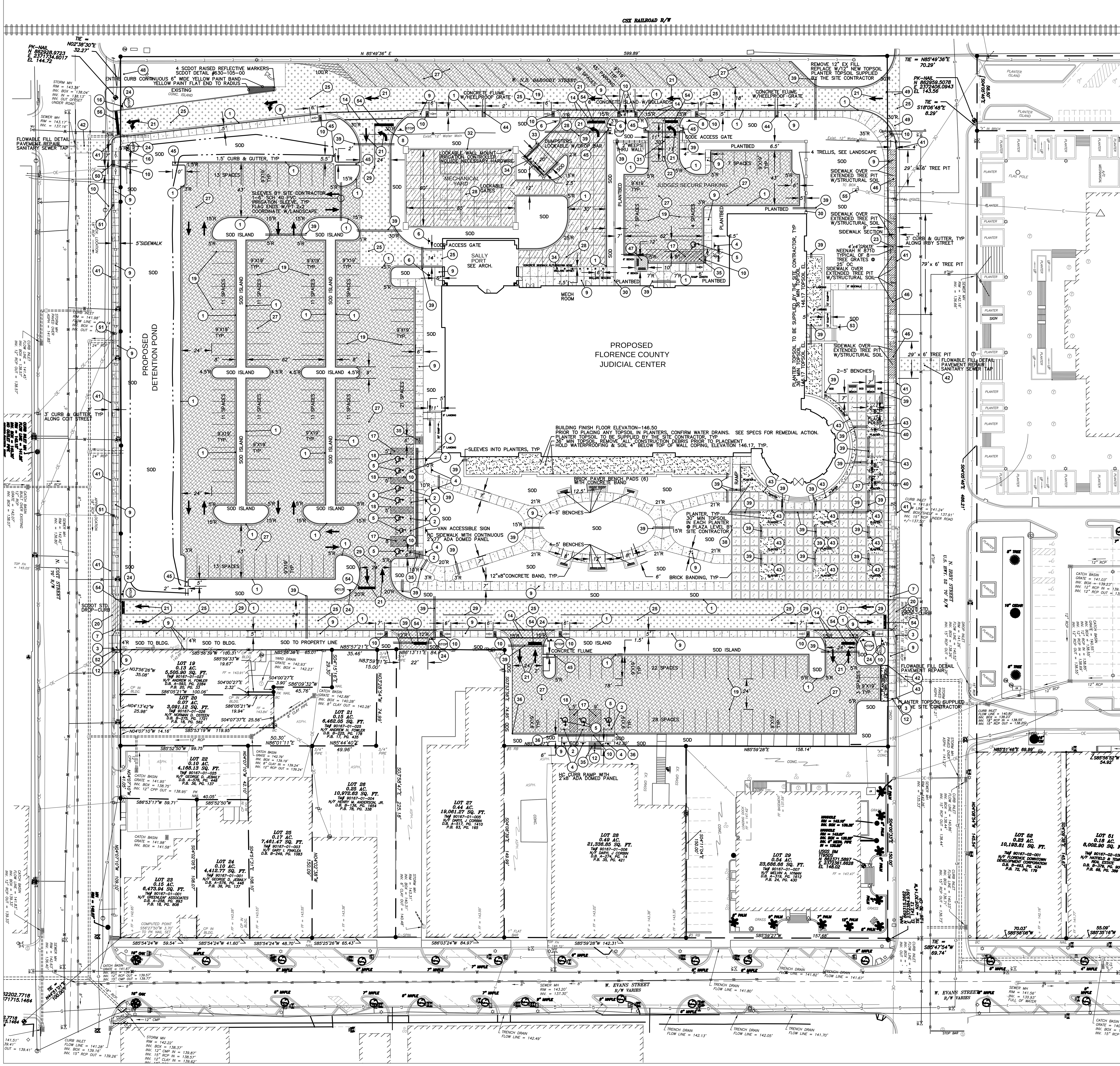


FIGURE 3: Shear Wave Velocity Profile SW-2
Florence Judicial Building
Florence, South Carolina
1439-15-029





REFERENCES

1) BOUNDARY & TOPOGRAPHIC SURVEY, DATED JULY 17, 2014, LAST REVISED SEPTEMBER 11, 2014 & WAS PROVIDED BY NESBITT SURVEYING CO., INC.
2) UTILITY INFORMATION PROVIDED BY SCDOT AND CITY OF FLORENCE.

GENERAL NOTES

1) THE SITE IS IDENTIFIED AS FLORENCE COUNTY TAX PARCELS 90167–01–029, –030, –020, –021, –019, –018, –017, –016, –015, –014, –013, –012, –011, –028, –010, –009, –008, –022.
2) THE PROJECT SITE IS WITHIN THE CITY LIMITS OF FLORENCE.
3) THE PROJECT SITE CURRENTLY CONSISTS OF SURFACE PARKING, COMMERCIAL BUILDINGS & VACANT COMMERCIAL LOTS.
5) THE CONTRACTOR IS RESPONSIBLE FOR DETERMINING THE LOCATIONS AND ELEVATIONS OF ALL UNDERGROUND UTILITIES PRIOR TO CONSTRUCTION.
6) WE HAVE CONSULTED FIRM MAP NUMBER 45041C0135D, PANEL 135 OF 565, EFFECTIVE DATE: DECEMBER 16, 2004, AND TO THE BEST OF OUR KNOWLEDGE AND BELIEF, THE PROPERTY IS NOT IN A SPECIAL FLOOD HAZARD ZONE.
7) ALL DIMENSIONS REFERENCED TO THE CURB INCLUDING RADII ARE TO THE FACE OF CURB (PARKING LOT SIDE).
8) INSTALL EXPANSION JOINTS AROUND UTILITY POLES LOCATED IN NEW SIDEWALKS.
9) ALL PAVEMENT MARKINGS WITHIN SCDOT R/W SHALL BE THERMOPLASTIC & INSTALLED ACCORDING TO CURRENT SCDOT STANDARDS & SPECIFICATIONS.
10) ANY TRAFFIC LOOPS THAT ARE DAMAGED DURING CONSTRUCTION SHALL BE REPAIRED BY THE CONTRACTOR UNDER THE SUPERVISION OF SCDOT. THIS INCLUDES THE PEDESTRIAN SIGNAL AT BAROODY STREET.
11) NO PAVERS ARE TO BE WITHIN 5' OF ANY NEW SIDEWALKS WITHIN SCDOT R/W.

CODED SITE NOTES

1 NEW 18" WIDE CONCRETE "L TYPE" CURB AND GUTTER.
2 NEW CONCRETE WHEELSTOP, TYP.
3 TIE NEW CURB AND GUTTER TO EXISTING.
4 NEW FREE STANDING HANDICAP PARKING SIGNAGE, TYP.
5 PAINTED ACCESSIBLE SYMBOL (SEE DETAIL). DENOTE 1 SPACE AS VAN ACCESSIBLE ON PAVEMENT AND SIGNAGE.
6 NEW FREE STANDING "AUTHORIZED VEHICLES ONLY" SIGNAGE.
7 NEW 24' WIDE DROP–CURB TWO–WAY ACCESS DRIVE PER SCDOT SPECS.
8 NEW FREE STANDING LEFT TURN ONLY SIGN.
9 NEW 4" DEPTH CONCRETE SIDEWALK. RAISE EX. VALVE COVERS, MH LIDS AS NECESSARY. SEE DET.
10 NEW ACCESSIBLE CURB RAMPS WITH CONTINUOUS DETECTABLE WARNING SURFACE MEETING ADA REQUIREMENTS, SAFETY YELLOW. NOTE–PANELS ARE CONTINUOUS ACROSS DEPRESSED WALKS.
11 NEW CONCRETE SIDEWALK. WIDTH AS SHOWN. SEE DETAIL.
12 TIE NEW CONCRETE SIDEWALK TO EXISTING.
13 PROPOSED CONCRETE ACCESSIBLE RAMP WITH HANDRAILS MEETING ADA REQUIREMENTS.
14 6' WIDE PAINTED WHITE CROSSWALK STRIPING (8" STRIPING), 125 MIL THERMO.
15 FREESTANDING ACCESSIBLE SIGNAGE (SEE DETAIL).
16 ONE WAY DO NOT ENTER SIGN, BACK TO BACK W/STOP SIGN ON SAME POST.
17 4" WIDE PAINTED BLUE ACCESSIBLE STRIPING @ 45°, 2' O.C. (SEE STRIPING NOTES).
18 5' WIDE PAINTED BLUE ACCESSIBLE CROSSWALK. 4" WIDE STRIPING @ 45°, 2' O.C.
19 4" WIDE PAINTED WHITE PARKING LOT STRIPING (TYPICAL) (SEE STRIPING NOTES).
20 5' WIDE HEAVY DUTY CONCRETE CROSSWALK @ CURB DROPS.
21 PAINTED WHITE TRAFFIC DIRECTIONAL FLOW ARROW (TYP), 125 MIL THERMO. (SEE DETAILS).
22 2'WIDE X 12" DEEP (6" AFG) W/#3 @ 6"oc(LONGITUDINAL) CONCRETE ISLAND W/KEY PUNCH BOX.
23 TREE GRATES SUPPLIED & INSTALLED BY THE SITE CONTRACTOR.
24 SCDOT STANDARD STOP SIGN (SEE SIGNAGE NOTES).
25 NEW HEAVY DUTY ASPHALT PAVEMENT (SEE DETAILS).
26 NEW ASPHALT PAVEMENT PER SCDOT (SEE DETAILS).
27 NEW LIGHT DUTY ASPHALT PAVEMENT (SEE DETAILS).
28 NEW HEAVY DUTY CONCRETE PAVEMENT (SEE DETAILS).
29 4" WIDE DOUBLE YELLOW PAINTED STRIPING, 90 MIL THERMO.
30 NEW BRICK SECURITY WALL (SEE ARCHITECTURALS FOR DETAILS).
31 NEW 24' WIDE ELECTRICAL DOUBLE GATE, LOOP DETECTOR (SEE ARCHITECTURALS FOR DETAILS).
32 NEW BRICK ENCLOSURE WALL FOR MECHANICAL YARD (SEE ARCHITECTURALS FOR DETAILS).
33 NEW DUMPSTER ENCLOSURE W/SWING GATES & DROP PIN (SEE ARCHITECTURALS FOR DETAILS).
34 NEW STEEL BOLLARDS (7) w/TOPGUARD or EQUAL CAPS, 2 COATS HIGH VISIBILITY YELLOW.
35 NEW 6'x6' CONCRETE RAMP DOWN TO GRADE.
36 TAPERED CURB (SEE DETAILS).
37 3' WIDE x 8' LONG x 4" DEPTH CONCRETE WALK @ FLAGPOLES.
38 NEW BRICK PAVER BANDING (SEE ARCHITECTURALS).
39 1–6" SCH 40 PVC IRRIGATION SLEEVE @ EA. LOCATION.(TYPICAL SYMBOL ═══) PLACE 2x2 PRESSURE TREATED STAKE @ EA. END. PAINT TOPS ORANGE.
40 4–24'TALL–GALVANIZED STEEL FLAG POLES W/CONC FOOTINGS & LIGHTENING ARRESTORS, PER MGF.
41 3' WIDE CURB AND GUTTER PER CITY OF FLORENCE ON N. IRBY AND N. COIT STREETS.
42 SEE STANDARD DETAIL FOR UTILITY WORK IN EXISTING ROADWAY.
43 4"x 1/4" BLACK STEEL EDGING ALONG INSIDE EDGE OF ALL PLAZA PLANTERS.
44 2' WIDE CONCRETE CURBED FLUME W/ATTACHED 2'X6' ADA STAINLESS STL GRATE COVER.
45 2' WIDE CONCRETE CURBED FLUME.
46 4' WIDE TREE PLANTING PIT (3 TOTAL) W/STRUCTURAL PLANTING SOIL, SEE DETAIL.
47 1' WIDE CONC. BAND W/ CONCRETE PAVER, HERRINGBONE PATTERN ON SAND INFILL.
48 NOT USED
49 REPAINT CROSSWALK STRIPES DELETED BY NEW PAVING, 125 MIL THERMO.
50 INSTALL SCDOT PARALLEL PEDESTRIAN RAMP, #720–951–11.
51 INSTALL EXPANSION JOINT AROUND ALL EX. POWER/LIGHT POLES TO RECEIVE NEW CONCRETE WALK.
52 PROTECT ALL EXISTING POWER POLES DURING CONSTRUCTION ACTIVITY.
53 SIGN–'EMERGENCY EGRESS ONLY'.
54 24" WIDE PAINTED WHITE STOP BAR
55 SCDOT TRAFFIC LIGHT CABINET/POLE TO BE RELOCATED
56 24" WIDE WHITE THERMOPLASTIC STOP BAR

SIGNAGE NOTES

1. THE CONTRACTOR SHALL FURNISH AND INSTALL "HANDICAPPED PARKING" SIGNS WITH IDENTIFYING HANDICAPPED SYMBOL AT ALL HANDICAPPED PARKING SPACES. SIGNS SHALL BE 12"x18", 0.080 ALUMINUM WITH BAKED ENAMEL FINISH. SIGNS SHALL HAVE WHITE LETTERING ON BLUE BACKGROUND, WITH INSTALLATION AS SHOWN ON DETAIL SHEET. SIGNS SHALL CONFORM TO FEDERAL HIGHWAY ADMINISTRATION AND/OR SCDOT SPECIFICATIONS.
2. THE CONTRACTOR SHALL FURNISH AND INSTALL "STOP" SIGNS IN THE LOCATIONS INDICATED. "STOP" SIGNS SHALL HAVE RED REFLECTORIZED BACKGROUND AND WHITE REFLECTORIZED LEGEND AND BORDER CONFORMING TO SCDOT SPECIFICATIONS. "STOP" SIGNS SHALL BE 36"x36" (R1–1–36) AT DRIVES ENTERING IRBY AND COIT AND 30"x30" ELSEWHERE ON–SITE.

PARKING LOT STRIPING NOTES

1. ALL PARKING LOT STRIPING SHOWN SHALL BE PERFORMED BY THE CONTRACTOR USING WHITE TRAFFIC MARKING PAINT UNLESS NOTED OTHERWISE BELOW. PAINT SHALL BE SHERWIN–WILLIAMS "PRO–MAR" TRAFFIC MARKING PAINT OR GLIDDEN TRAFFIC PAINT AND SHALL BE APPLIED IN TWO COATS & IN STRICT ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS.
2. ITEMS TO BE PAINTED WHITE INCLUDE: PARKING STALLS; DIRECTIONAL ARROWS OF SIZE AND LOCATIONS INDICATED; STOP BARS; STRIPED ISLANDS; RAISED DIVIDERS AND/OR MEDIANS; ITEMS TO BE PAINTED YELLOW INCLUDE: ALL SITEWORK PROTECTIVE GUARD POSTS BEYOND (5) FIVE FEET OF THE BUILDING; STEEL PIPE BASE OF HANDICAP PARKING SIGNS; AND ANY MARKINGS THAT WILL BE LOCATED ON CONCRETE SURFACE.
3. ITEMS REQUIRING BLUE PAINT INCLUDE HANDICAP PARKING STALLS AND SYMBOLS; ALL STRIPED ISLANDS ADJACENT TO HANDICAP PARKING SPACES.

HEAVY DUTY ASPHALT PAVEMENT (SEE DETAILS FOR COMPOSITION & THICKNESS)

LIGHT DUTY ASPHALT PAVEMENT (SEE DETAILS FOR COMPOSITION & THICKNESS)

NEW HEAVY DUTY CONCRETE PAVEMENT (SEE DETAILS FOR COMPOSITION & THICKNESS)

NEW NORMAL DUTY CONCRETE PAVEMENT (SEE DETAILS FOR COMPOSITION & THICKNESS)

STRUCTURAL SOIL (SEE DETAILS)

PARKING SUMMARY

PARKING SPACES PROVIDED

- MAIN PARKING LOT:	135 SPACES
- SOUTHEAST PARKING LOT:	53 SPACES
- BAROODY PARKING LOT:	28 SPACES
- JUDGES' SECURE PARKING LOT:	18 SPACES
TOTAL PARKING SPACES:	234 SPACES
ACCESSIBLE PARKING PROVIDED:	11 SPACES
ACCESSIBLE PARKING PERCENTAGE OF TOTAL:	4.7%

0 30 60 90 120
GRAPHIC SCALE IN FEET

SC GRID

S&W
ARCHITECTURE ENGINEERING INTERIORS
STEVENS & WILKINSON
1501 MAIN STREET, SUITE 730
COLUMBIA, SC 29201
P 803.765.0320 F 803.254.6209
WWW.STEVENS-WILKINSON.COM

Goforth Brown & Associates
architects and planners
1413 West Evans Street
Florence, South Carolina

Justice Planning Associates, Inc.

SUBMITTALS (BY LETTER)/REVISIONS (BY NUMBER)

NO.	DATE	DESCRIPTION
A	12.16.15	PERMIT SET

☐ APPROVED FOR CONSTRUCTION
■ NOT APPROVED FOR CONSTRUCTION

S&W PROJECT NUMBER: 13022.00

☐ DATE: 12.16.15

FLORENCE COUNTY

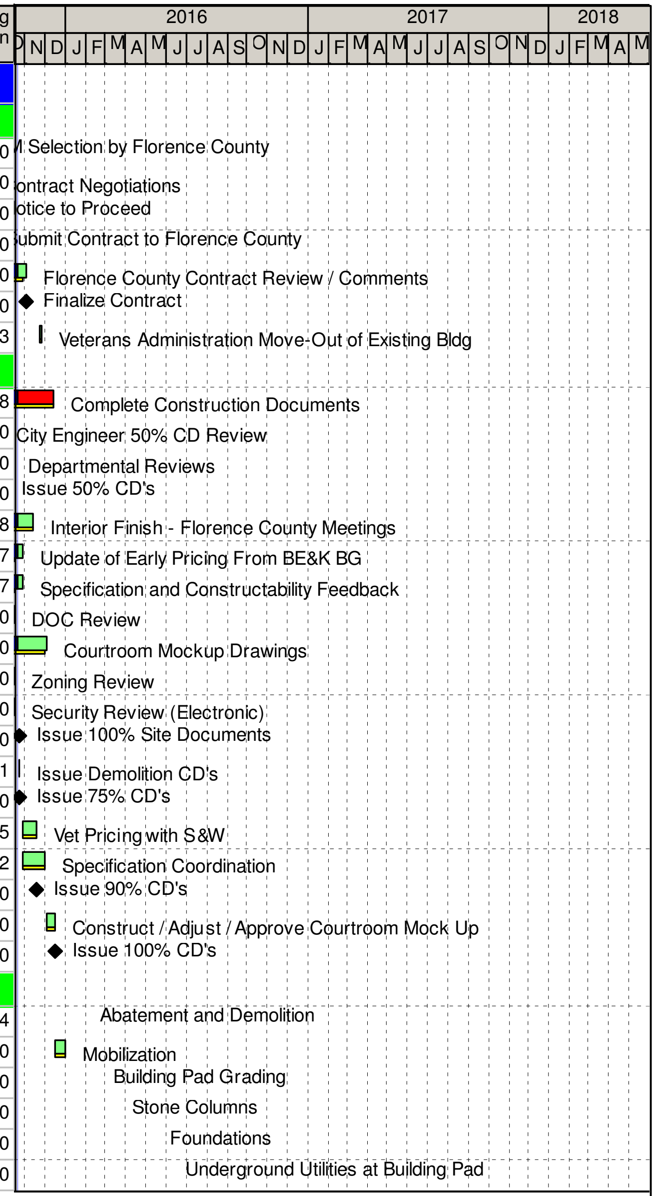
FLORENCE COUNTY SOUTH CAROLINA 1888

FLORENCE COUNTY JUDICIAL CENTER

SITE PLAN

C3.00

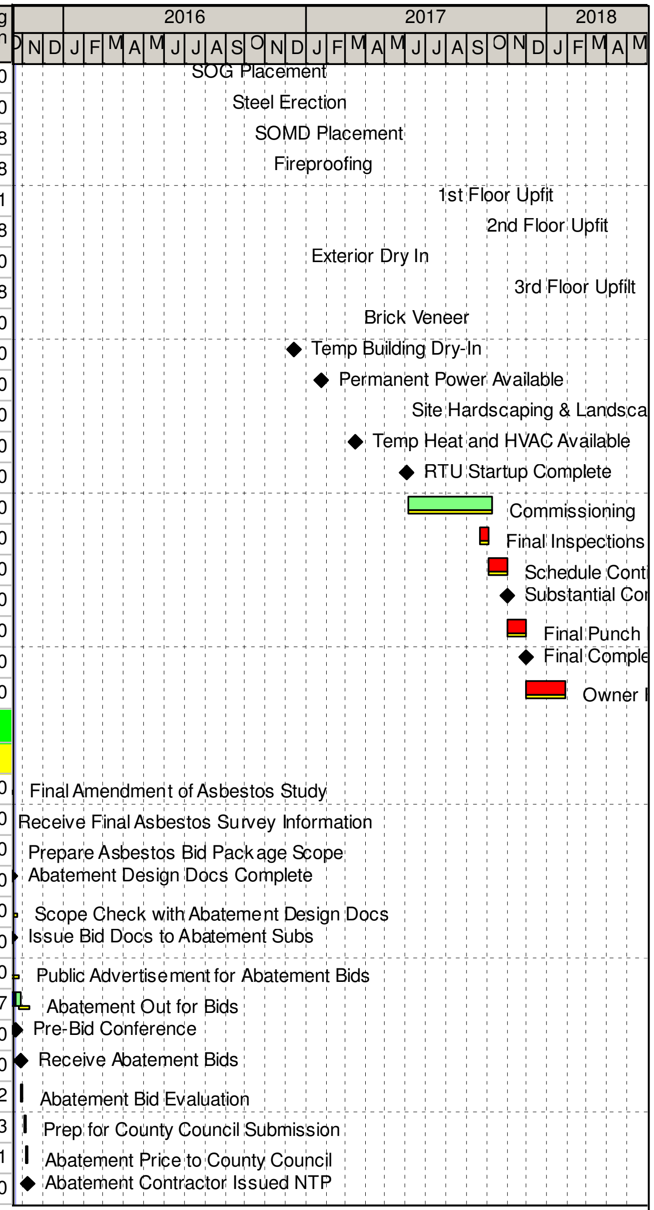
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
Florence County Judicial Center							
General Milestones							
100	CM Selection by Florence County	0	8/6/15	8/21/15			0
110	Contract Negotiations	10	8/6/15	8/21/15	8/19/15	9/2/15	0
120	Notice to Proceed	0	8/20/15	9/2/15			0
130	Submit Contract to Florence County	0	9/9/15	9/8/15			0
140	Florence County Contract Review / Comments	36	9/10/15	10/8/15	10/29/15		10
150	Finalize Contract	0	11/3/15				0
160	Veterans Administration Move-Out of Existing Bldg	3	11/23/15		11/25/15		3
Design Milestones							
20000	Complete Construction Documents	79	8/21/15	8/21/15	12/14/15		38
20160	City Engineer 50% CD Review	1	9/21/15	9/21/15	9/21/15	9/21/15	0
20130	Departmental Reviews	10	9/28/15	9/28/15	10/9/15	10/9/15	0
20020	Issue 50% CD's	0	9/23/15	10/1/15			0
20170	Interior Finish - Florence County Meetings	32	10/1/15	10/1/15	11/13/15		18
20090	Update of Early Pricing From BE&K BG	17	9/23/15	10/2/15	10/15/15		7
20110	Specification and Constructability Feedback	17	9/23/15	10/2/15	10/15/15		7
20140	DOC Review	10	10/5/15	10/5/15	10/16/15	10/16/15	0
20120	Courtroom Mockup Drawings	40	10/5/15	10/5/15	12/1/15		30
20150	Zoning Review	5	10/12/15	10/12/15	10/16/15	10/16/15	0
20180	Security Review (Electronic)	5	10/12/15	10/12/15	10/16/15	10/16/15	0
20040	Issue 100% Site Documents	0	10/22/15				0
20030	Issue Demolition CD's	1	10/22/15		10/22/15		1
20050	Issue 75% CD's	0	10/23/15				0
20100	Vet Pricing with S&W	15	10/29/15		11/18/15		15
20190	Specification Coordination	22	10/29/15		12/1/15		22
20060	Issue 90% CD's	0	11/19/15				0
20200	Construct / Adjust / Approve Courtroom Mock Up	10	12/3/15		12/16/15		10
20070	Issue 100% CD's	0	12/15/15				0
Construction Milestones							
70110	Abatement and Demolition	34	12/7/15		1/26/16		34
70000	Mobilization	10	12/16/15		12/31/15		10
70120	Building Pad Grading	20	1/20/16		2/16/16		20
70130	Stone Columns	20	2/17/16		3/15/16		20
70140	Foundations	30	3/31/16		5/11/16		30
70150	Underground Utilities at Building Pad	40	4/7/16		6/2/16		40



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
A Pernix Group Company

Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
70160	SOG Placement	10	6/3/16		6/16/16		10
70170	Steel Erection	40	6/21/16		8/16/16		40
70180	SOMD Placement	28	8/10/16		9/19/16		28
70190	Fireproofing	28	9/8/16		10/17/16		28
70200	1st Floor Upfit	191	9/22/16		6/23/17		191
70210	2nd Floor Upfit	238	9/27/16		9/5/17		238
70230	Exterior Dry In	50	10/4/16		12/14/16		50
70220	3rd Floor Upfit	258	10/11/16		10/17/17		258
70240	Brick Veneer	70	11/22/16		3/3/17		70
70010	Temp Building Dry-In	0			12/14/16		0
70020	Permanent Power Available	0	1/23/17				0
70250	Site Hardscaping & Landscaping	50	3/6/17		5/15/17		50
70260	Temp Heat and HVAC Available	0	3/16/17				0
70030	RTU Startup Complete	0			6/2/17		0
70040	Commissioning	90	6/5/17		10/10/17		90
70050	Final Inspections	10	9/20/17		10/3/17		10
70090	Schedule Contingency	20	10/4/17		10/31/17		20
70060	Substantial Completion	0			10/31/17		0
70070	Final Punch List / Final Clean	20	11/1/17		11/30/17		20
70080	Final Completion	0			11/30/17		0
70100	Owner FFE & Move-In	40	12/1/17		1/29/18		40
Preconstruction							
Abatement Preconstruction and Design							
11170	Final Amendment of Asbestos Study	13	9/28/15	9/17/15	10/14/15	9/28/15	0
11160	Receive Final Asbestos Survey Information	1	9/25/15	9/28/15	9/25/15	9/28/15	0
11010	Prepare Asbestos Bid Package Scope	6	10/2/15	10/7/15	10/9/15	10/12/15	0
11050	Abatement Design Docs Complete	0			10/16/15	10/12/15	0
11070	Scope Check with Abatement Design Docs	5	10/19/15	10/12/15	10/23/15	10/12/15	0
11080	Issue Bid Docs to Abatement Subs	0	10/26/15	10/12/15			0
11000	Public Advertisement for Abatement Bids	14	10/12/15	10/13/15	10/25/15	10/13/15	0
11090	Abatement Out for Bids	12	10/26/15	10/13/15	11/10/15		7
11100	Pre-Bid Conference	0	10/21/15				0
11110	Receive Abatement Bids	0			10/28/15		0
11120	Abatement Bid Evaluation	2	10/29/15		10/30/15		2
11130	Prep for County Council Submission	3	11/2/15		11/4/15		3
11180	Abatement Price to County Council	1	11/6/15		11/6/15		1
11150	Abatement Contractor Issued NTP	0			11/6/15		0

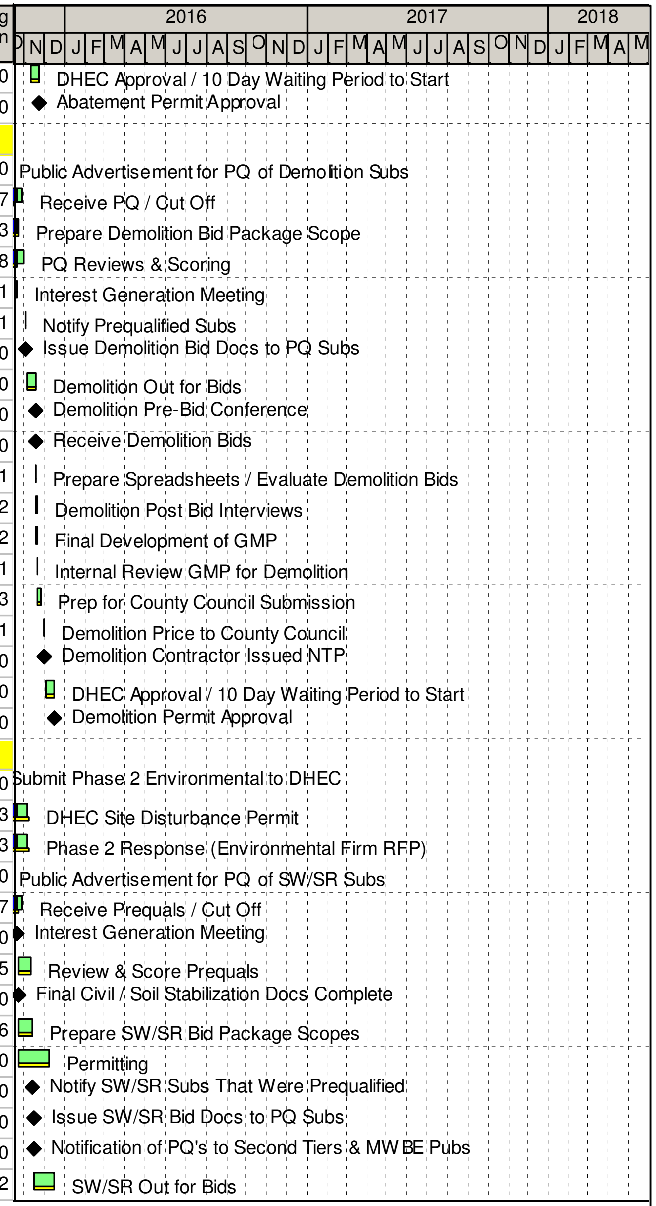


FCJC- 151004
10/20/15

Florence County Judicial Center
Preliminary Schedule

Building Group
A Pernix Group Company

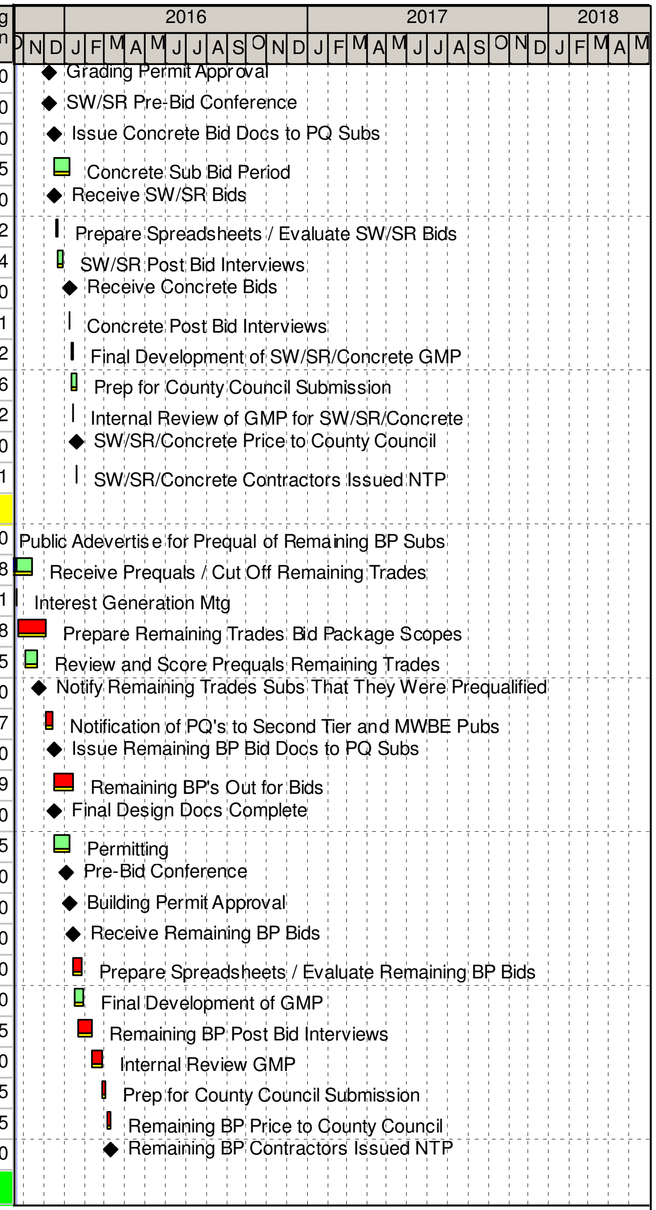
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
11060	DHEC Approval / 10 Day Waiting Period to Start	10	11/10/15		11/23/15		10
11140	Abatement Permit Approval	0			11/23/15		0
Demolition Preconstruction							
10510	Public Advertisement for PQ of Demolition Subs	5	9/24/15	9/24/15	9/28/15	9/28/15	0
10540	Receive PQ / Cut Off	15	9/28/15	9/28/15	10/16/15		7
10520	Prepare Demolition Bid Package Scope	10	10/12/15	10/12/15	10/23/15		3
10550	PQ Reviews & Scoring	7	10/12/15	10/12/15	10/20/15		8
10530	Interest Generation Meeting	1	10/20/15		10/20/15		1
10560	Notify Prequalified Subs	1	11/2/15		11/2/15		1
10590	Issue Demolition Bid Docs to PQ Subs	0	11/3/15				0
10600	Demolition Out for Bids	10	11/4/15		11/17/15		10
10610	Demolition Pre-Bid Conference	0	11/17/15				0
10620	Receive Demolition Bids	0			11/17/15		0
10630	Prepare Spreadsheets / Evaluate Demolition Bids	1	11/18/15		11/18/15		1
10670	Demolition Post Bid Interviews	2	11/18/15		11/19/15		2
10640	Final Development of GMP	2	11/18/15		11/19/15		2
10680	Internal Review GMP for Demolition	1	11/20/15		11/20/15		1
10700	Prep for County Council Submission	3	11/20/15		11/24/15		3
10710	Demolition Price to County Council	1	11/30/15		11/30/15		1
10660	Demolition Contractor Issued NTP	0			12/1/15		0
10690	DHEC Approval / 10 Day Waiting Period to Start	10	12/2/15		12/15/15		10
10650	Demolition Permit Approval	0			12/15/15		0
Sitework / Soil Stabalization / Conrete Preconstruction							
12160	Submit Phase 2 Environmental to DHEC	0	9/15/15	9/15/15			0
12060	DHEC Site Disturbance Permit	35	9/21/15	9/21/15	11/6/15		13
12170	Phase 2 Response (Environmental Firm RFP)	35	9/21/15	9/21/15	11/6/15		13
12000	Public Advertisement for PQ of SW/SR Subs	5	9/24/15	9/24/15	9/28/15	9/28/15	0
12030	Receive Prequals / Cut Off	20	9/28/15	9/28/15	10/23/15		7
12020	Interest Generation Meeting	0	10/20/15				0
12040	Review & Score Prequals	15	10/22/15		11/11/15		15
12050	Final Civil / Soil Stabilization Docs Complete	0			10/22/15		0
12010	Prepare SW/SR Bid Package Scopes	16	10/23/15		11/13/15		16
12180	Permitting	30	10/23/15		12/7/15		30
12190	Notify SW/SR Subs That Were Prequalified	0	11/12/15				0
12080	Issue SW/SR Bid Docs to PQ Subs	0	11/16/15				0
12200	Notification of PQ's to Second Tiers & MWBE Pub	0	11/16/15				0
12090	SW/SR Out for Bids	22	11/16/15		12/17/15		22



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
A Pernix Group Company

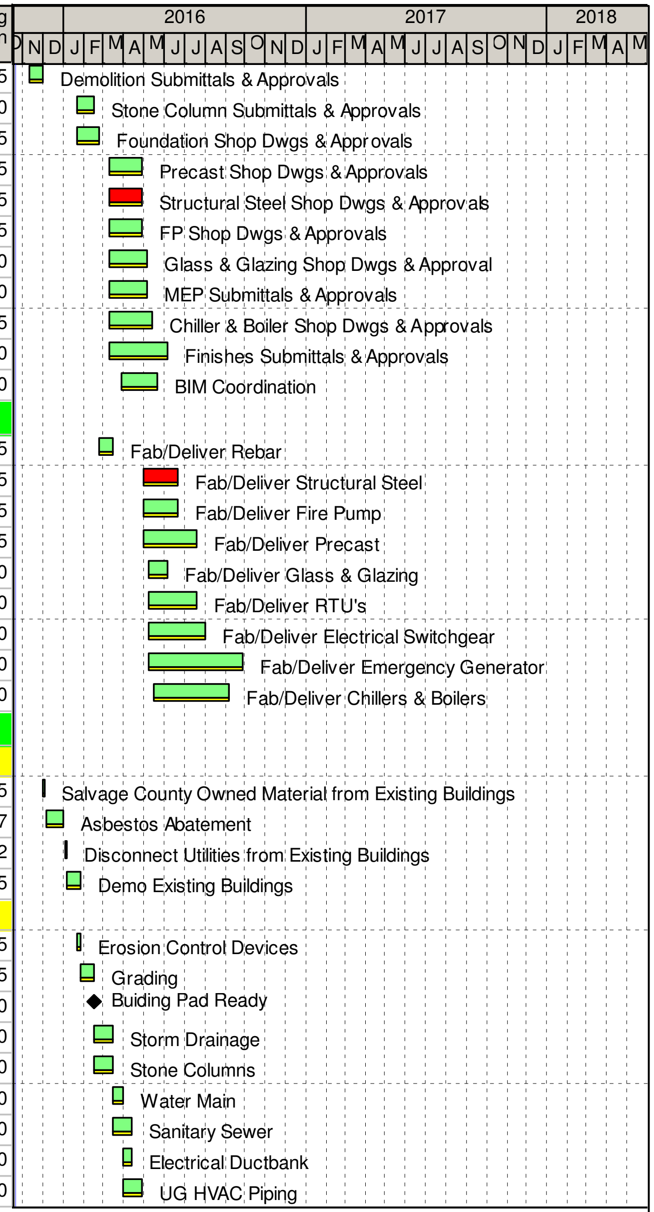
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
12140	Grading Permit Approval	0			12/7/15		0
12100	SW/SR Pre-Bid Conference	0	12/8/15				0
12270	Issue Concrete Bid Docs to PQ Subs	0	12/15/15				0
12280	Concrete Sub Bid Period	15	12/15/15		1/7/16		15
12110	Receive SW/SR Bids	0			12/17/15		0
12120	Prepare Spreadsheets / Evaluate SW/SR Bids	2	12/18/15		12/21/15		2
12210	SW/SR Post Bid Interviews	4	12/22/15		12/29/15		4
12250	Receive Concrete Bids	0			1/7/16		0
12260	Concrete Post Bid Interviews	1	1/8/16		1/8/16		1
12130	Final Development of SW/SR/Concrete GMP	2	1/11/16		1/12/16		2
12230	Prep for County Council Submission	6	1/11/16		1/18/16		6
12220	Internal Review of GMP for SW/SR/Concrete	2	1/13/16		1/14/16		2
12240	SW/SR/Concrete Price to County Council	0	1/19/16				0
12150	SW/SR/Concrete Contractors Issued NTP	1	1/19/16		1/19/16		1
Remaining Trades Preconstruction							
13000	Public Adevertise for Prequal of Remaining BP Sul	5	9/24/15	9/24/15	9/28/15	9/28/15	0
13030	Receive Prequals / Cut Off Remaining Trades	35	9/28/15	9/28/15	11/13/15		18
13020	Interest Generation Mtg	1	10/20/15		10/20/15		1
13010	Prepare Remaining Trades Bid Package Scopes	28	10/23/15		12/3/15		28
13040	Review and Score Prequals Remaining Trades	15	11/2/15		11/20/15		15
13160	Notify Remaining Trades Subs That They Were Pı	0	11/23/15				0
13170	Notification of PQ's to Second Tier and MWBE Pu	7	12/4/15		12/14/15		7
13080	Issue Remaining BP Bid Docs to PQ Subs	0	12/15/15				0
13090	Remaining BP's Out for Bids	19	12/15/15		1/13/16		19
13050	Final Design Docs Complete	0			12/15/15		0
13060	Permitting	15	12/16/15		1/8/16		15
13100	Pre-Bid Conference	0	1/4/16				0
13140	Building Permit Approval	0			1/8/16		0
13110	Receive Remaining BP Bids	0			1/13/16		0
13120	Prepare Spreadsheets / Evaluate Remaining BP E	10	1/14/16		1/27/16		10
13130	Final Development of GMP	10	1/15/16		1/28/16		10
13180	Remaining BP Post Bid Interviews	15	1/22/16		2/11/16		15
13190	Internal Review GMP	10	2/12/16		2/25/16		10
13200	Prep for County Council Submission	5	2/26/16		3/3/16		5
13210	Remaining BP Price to County Council	5	3/4/16		3/10/16		5
13150	Remaining BP Contractors Issued NTP	0	3/11/16				0
Submittals							



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
A Pernix Group Company

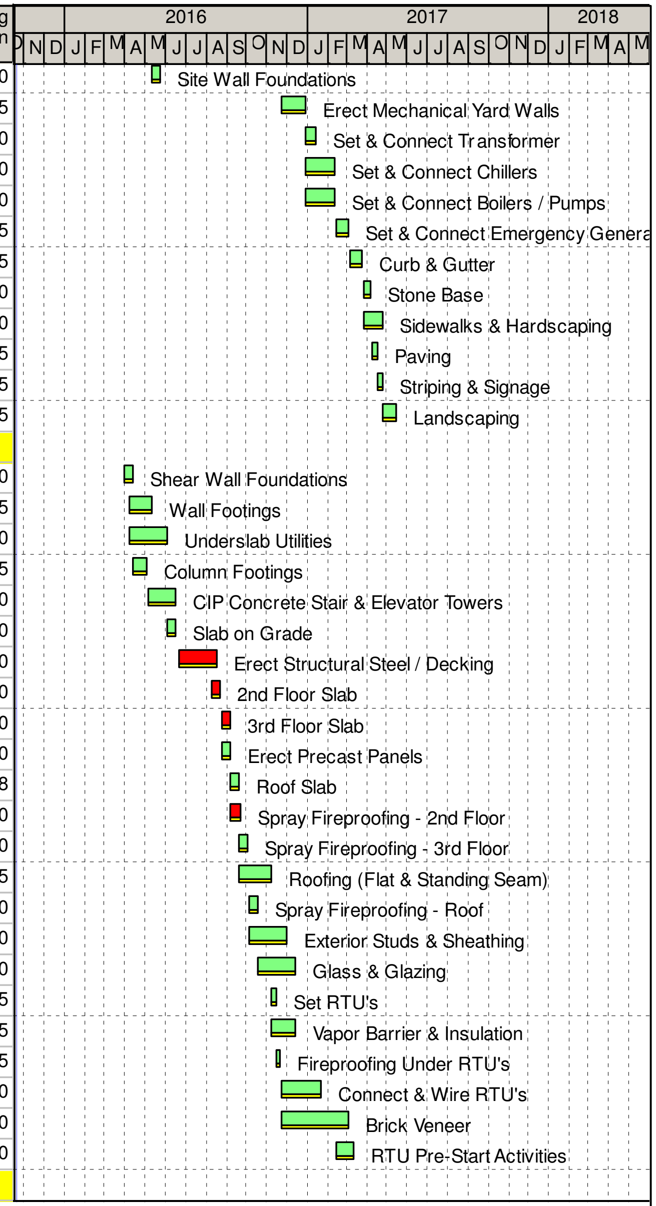
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
40000	Demolition Submittals & Approvals	15	11/9/15		12/1/15		15
40080	Stone Column Submittals & Approvals	20	1/20/16		2/16/16		20
40010	Foundation Shop Dwgs & Approvals	25	1/20/16		2/23/16		25
40090	Precast Shop Dwgs & Approvals	35	3/11/16		4/29/16		35
40020	Structural Steel Shop Dwgs & Approvals	35	3/11/16		4/29/16		35
40050	FP Shop Dwgs & Approvals	35	3/11/16		4/29/16		35
40100	Glass & Glazing Shop Dwgs & Approval	40	3/11/16		5/6/16		40
40040	MEP Submittals & Approvals	40	3/11/16		5/6/16		40
40030	Chiller & Boiler Shop Dwgs & Approvals	45	3/11/16		5/13/16		45
40060	Finishes Submittals & Approvals	60	3/11/16		6/6/16		60
40070	BIM Coordination	40	3/28/16		5/20/16		40
Fab/Deliver							
50010	Fab/Deliver Rebar	15	2/24/16		3/15/16		15
50020	Fab/Deliver Structural Steel	35	5/2/16		6/20/16		35
50050	Fab/Deliver Fire Pump	35	5/2/16		6/20/16		35
50090	Fab/Deliver Precast	55	5/2/16		7/19/16		55
50100	Fab/Deliver Glass & Glazing	20	5/9/16		6/6/16		20
50080	Fab/Deliver RTU's	50	5/9/16		7/19/16		50
50040	Fab/Deliver Electrical Switchgear	60	5/9/16		8/2/16		60
50070	Fab/Deliver Emergency Generator	100	5/9/16		9/28/16		100
50030	Fab/Deliver Chillers & Boilers	80	5/16/16		9/7/16		80
Construction							
Asbestos Abatement & Demolition							
71000	Salvage County Owned Material from Existing Buil	5	11/30/15		12/4/15		5
71040	Asbestos Abatement	17	12/7/15		12/31/15		17
71010	Disconnect Utilities from Existing Buildings	2	1/4/16		1/5/16		2
71030	Demo Existing Buildings	15	1/6/16		1/26/16		15
Sitework							
71020	Erosion Control Devices	5	1/20/16		1/26/16		5
72000	Grading	15	1/27/16		2/16/16		15
72060	Buiding Pad Ready	0	2/17/16				0
72010	Storm Drainage	20	2/17/16		3/15/16		20
73000	Stone Columns	20	2/17/16		3/15/16		20
72030	Water Main	10	3/16/16		3/30/16		10
72020	Sanitary Sewer	20	3/16/16		4/13/16		20
72040	Electrical Ductbank	10	3/31/16		4/13/16		10
72050	UG HVAC Piping	20	3/31/16		4/27/16		20



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
A Pernix Group Company

Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
73050	Site Wall Foundations	10	5/12/16		5/25/16		10
72110	Erect Mechanical Yard Walls	25	11/22/16		12/29/16		25
72130	Set & Connect Transformer	10	12/30/16		1/13/17		10
72140	Set & Connect Chillers	30	12/30/16		2/10/17		30
72150	Set & Connect Boilers / Pumps	30	12/30/16		2/10/17		30
72160	Set & Connect Emergency Generator	15	2/13/17		3/3/17		15
72070	Curb & Gutter	15	3/6/17		3/24/17		15
72090	Stone Base	10	3/27/17		4/7/17		10
72080	Sidewalks & Hardscaping	20	3/27/17		4/24/17		20
72170	Paving	5	4/10/17		4/17/17		5
72100	Striping & Signage	5	4/18/17		4/24/17		5
72120	Landscaping	15	4/25/17		5/15/17		15
Foundation & Shell							
73010	Shear Wall Foundations	10	3/31/16		4/13/16		10
73020	Wall Footings	25	4/7/16		5/11/16		25
73060	Underslab Utilities	40	4/7/16		6/2/16		40
73040	Column Footings	15	4/14/16		5/4/16		15
73030	CIP Concrete Stair & Elevator Towers	30	5/5/16		6/16/16		30
73070	Slab on Grade	10	6/3/16		6/16/16		10
73110	Erect Structural Steel / Decking	40	6/21/16		8/16/16		40
73080	2nd Floor Slab	10	8/10/16		8/23/16		10
73090	3rd Floor Slab	10	8/24/16		9/7/16		10
73120	Erect Precast Panels	10	8/24/16		9/7/16		10
73100	Roof Slab	8	9/8/16		9/19/16		8
73220	Spray Fireproofing - 2nd Floor	10	9/8/16		9/21/16		10
73170	Spray Fireproofing - 3rd Floor	10	9/20/16		10/3/16		10
73140	Roofing (Flat & Standing Seam)	35	9/20/16		11/7/16		35
73180	Spray Fireproofing - Roof	10	10/4/16		10/17/16		10
73190	Exterior Studs & Sheathing	40	10/4/16		11/30/16		40
73130	Glass & Glazing	40	10/18/16		12/14/16		40
73160	Set RTU's	5	11/8/16		11/14/16		5
73200	Vapor Barrier & Insulation	25	11/8/16		12/14/16		25
73230	Fireproofing Under RTU's	5	11/15/16		11/21/16		5
73150	Connect & Wire RTU's	40	11/22/16		1/20/17		40
73210	Brick Veneer	70	11/22/16		3/3/17		70
73240	RTU Pre-Start Activities	20	2/13/17		3/10/17		20
First Floor							



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
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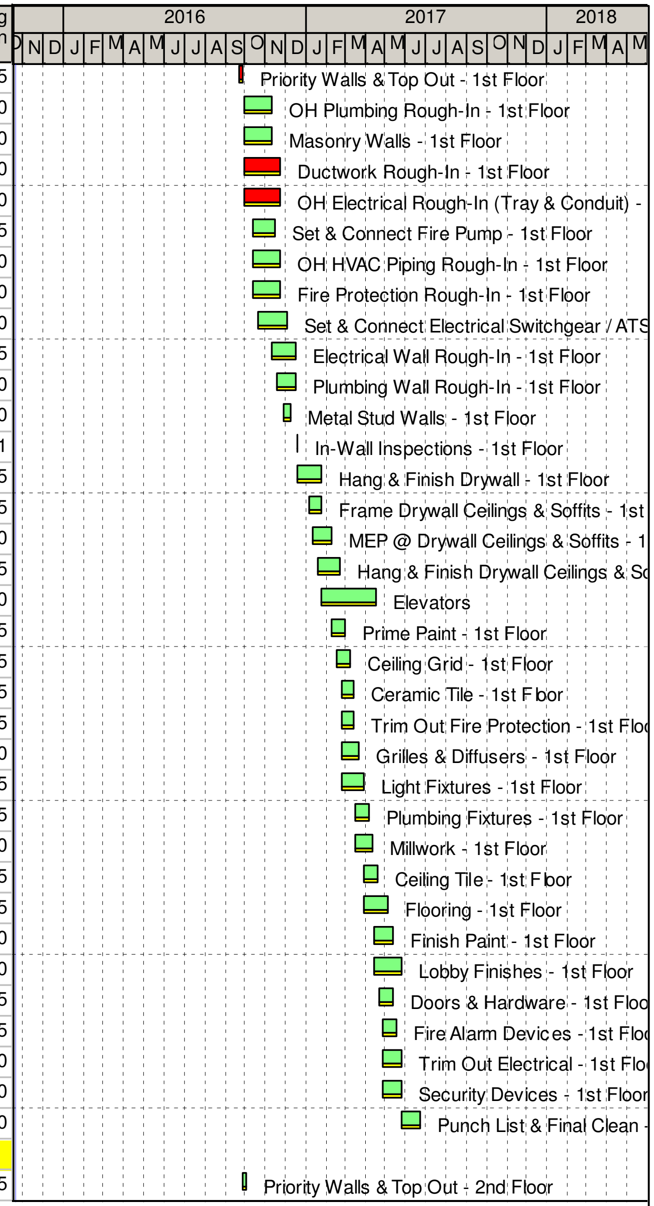
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
81000	Priority Walls & Top Out - 1st Floor	5	9/22/16		9/28/16		5
81030	OH Plumbing Rough-In - 1st Floor	30	9/29/16		11/9/16		30
81040	Masonry Walls - 1st Floor	30	9/29/16		11/9/16		30
81010	Ductwork Rough-In - 1st Floor	40	9/29/16		11/23/16		40
81020	OH Electrical Rough-In (Tray & Conduit) - 1st Flo	40	9/29/16		11/23/16		40
81320	Set & Connect Fire Pump - 1st Floor	25	10/13/16		11/16/16		25
81110	OH HVAC Piping Rough-In - 1st Floor	30	10/13/16		11/23/16		30
81120	Fire Protection Rough-In - 1st Floor	30	10/13/16		11/23/16		30
81310	Set & Connect Electrical Switchgear / ATS - 1st Fl	30	10/20/16		12/2/16		30
81090	Electrical Wall Rough-In - 1st Floor	25	11/10/16		12/16/16		25
81100	Plumbing Wall Rough-In - 1st Floor	20	11/17/16		12/16/16		20
81050	Metal Stud Walls - 1st Floor	10	11/28/16		12/9/16		10
81130	In-Wall Inspections - 1st Floor	1	12/19/16		12/19/16		1
81140	Hang & Finish Drywall - 1st Floor	25	12/20/16		1/25/17		25
81150	Frame Drywall Ceilings & Soffits - 1st Floor	15	1/5/17		1/25/17		15
81160	MEP @ Drywall Ceilings & Soffits - 1st Floor	20	1/12/17		2/8/17		20
81170	Hang & Finish Drywall Ceilings & Soffits - 1st Floor	25	1/19/17		2/22/17		25
81330	Elevators	60	1/23/17		4/17/17		60
81190	Prime Paint - 1st Floor	15	2/9/17		3/1/17		15
81210	Ceiling Grid - 1st Floor	15	2/16/17		3/8/17		15
81180	Ceramic Tile - 1st Floor	15	2/23/17		3/15/17		15
81250	Trim Out Fire Protection - 1st Floor	15	2/23/17		3/15/17		15
81240	Grilles & Diffusers - 1st Floor	20	2/23/17		3/22/17		20
81230	Light Fixtures - 1st Floor	25	2/23/17		3/29/17		25
81270	Plumbing Fixtures - 1st Floor	15	3/16/17		4/5/17		15
81260	Millwork - 1st Floor	20	3/16/17		4/12/17		20
81220	Ceiling Tile - 1st Floor	15	3/30/17		4/20/17		15
81280	Flooring - 1st Floor	25	3/30/17		5/4/17		25
81200	Finish Paint - 1st Floor	20	4/13/17		5/11/17		20
81360	Lobby Finishes - 1st Floor	30	4/13/17		5/25/17		30
81300	Doors & Hardware - 1st Floor	15	4/21/17		5/11/17		15
81350	Fire Alarm Devices - 1st Floor	15	4/28/17		5/18/17		15
81290	Trim Out Electrical - 1st Floor	20	4/28/17		5/25/17		20
81340	Security Devices - 1st Floor	20	4/28/17		5/25/17		20
81500	Punch List & Final Clean - 1st Floor	20	5/26/17		6/23/17		20
Second Floor							
82000	Priority Walls & Top Out - 2nd Floor	5	9/27/16		10/3/16		5



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
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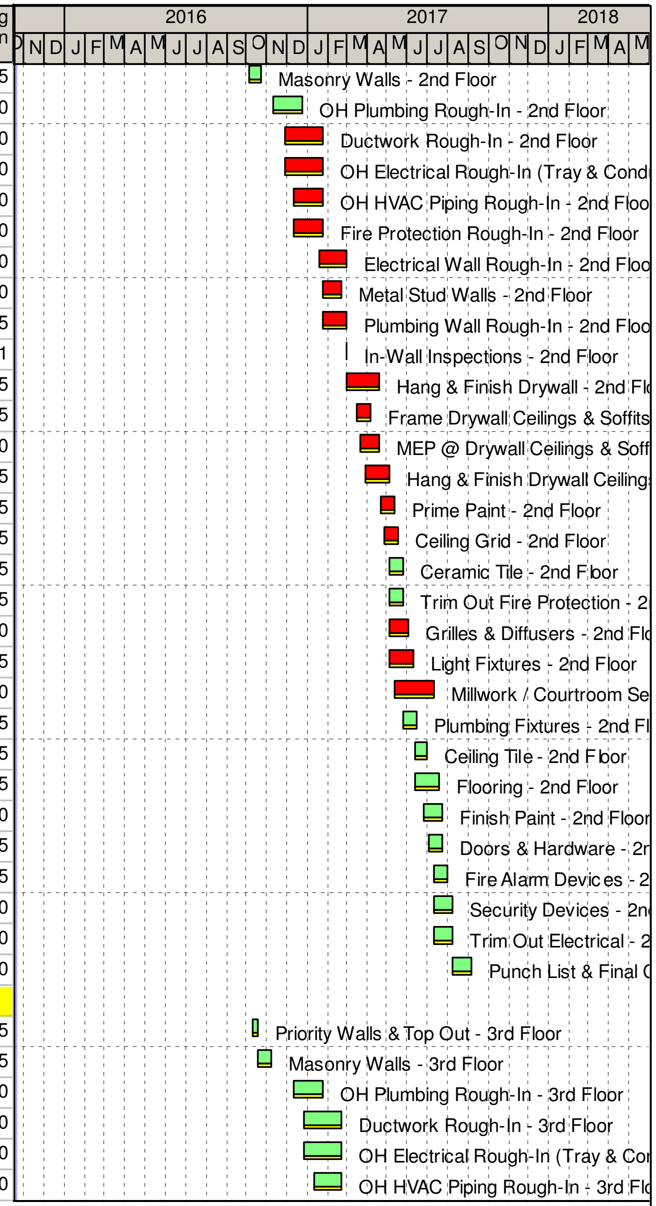
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
82040	Masonry Walls - 2nd Floor	15	10/4/16		10/24/16		15
82030	OH Plumbing Rough-In - 2nd Floor	30	11/10/16		12/23/16		30
82010	Ductwork Rough-In - 2nd Floor	40	11/28/16		1/24/17		40
82020	OH Electrical Rough-In (Tray & Conduit) - 2nd Flc	40	11/28/16		1/24/17		40
82110	OH HVAC Piping Rough-In - 2nd Floor	30	12/12/16		1/24/17		30
82120	Fire Protection Rough-In - 2nd Floor	30	12/12/16		1/24/17		30
82090	Electrical Wall Rough-In - 2nd Floor	30	1/18/17		2/28/17		30
82050	Metal Stud Walls - 2nd Floor	20	1/25/17		2/21/17		20
82100	Plumbing Wall Rough-In - 2nd Floor	25	1/25/17		2/28/17		25
82130	In-Wall Inspections - 2nd Floor	1	3/1/17		3/1/17		1
82140	Hang & Finish Drywall - 2nd Floor	35	3/2/17		4/20/17		35
82150	Frame Drywall Ceilings & Soffits - 2nd Floor	15	3/16/17		4/5/17		15
82160	MEP @ Drywall Ceilings & Soffits - 2nd Floor	20	3/23/17		4/20/17		20
82170	Hang & Finish Drywall Ceilings & Soffits - 2nd Floc	25	3/30/17		5/4/17		25
82190	Prime Paint - 2nd Floor	15	4/21/17		5/11/17		15
82210	Ceiling Grid - 2nd Floor	15	4/28/17		5/18/17		15
82180	Ceramic Tile - 2nd Floor	15	5/5/17		5/25/17		15
82250	Trim Out Fire Protection - 2nd Floor	15	5/5/17		5/25/17		15
82240	Grilles & Diffusers - 2nd Floor	20	5/5/17		6/2/17		20
82230	Light Fixtures - 2nd Floor	25	5/5/17		6/9/17		25
82260	Millwork / Courtroom Seating - 2nd Floor	40	5/12/17		7/10/17		40
82270	Plumbing Fixtures - 2nd Floor	15	5/26/17		6/16/17		15
82220	Ceiling Tile - 2nd Floor	15	6/12/17		6/30/17		15
82280	Flooring - 2nd Floor	25	6/12/17		7/17/17		25
82200	Finish Paint - 2nd Floor	20	6/26/17		7/24/17		20
82300	Doors & Hardware - 2nd Floor	15	7/3/17		7/24/17		15
82350	Fire Alarm Devices - 2nd Floor	15	7/11/17		7/31/17		15
82340	Security Devices - 2nd Floor	20	7/11/17		8/7/17		20
82290	Trim Out Electrical - 2nd Floor	20	7/11/17		8/7/17		20
82500	Punch List & Final Clean - 2nd Floor	20	8/8/17		9/5/17		20
Third Floor							
83000	Priority Walls & Top Out - 3rd Floor	5	10/11/16		10/17/16		5
83040	Masonry Walls - 3rd Floor	15	10/18/16		11/7/16		15
83030	OH Plumbing Rough-In - 3rd Floor	30	12/12/16		1/24/17		30
83010	Ductwork Rough-In - 3rd Floor	40	12/27/16		2/21/17		40
83020	OH Electrical Rough-In (Tray & Conduit) - 3rd Flo	40	12/27/16		2/21/17		40
83110	OH HVAC Piping Rough-In - 3rd Floor	30	1/11/17		2/21/17		30



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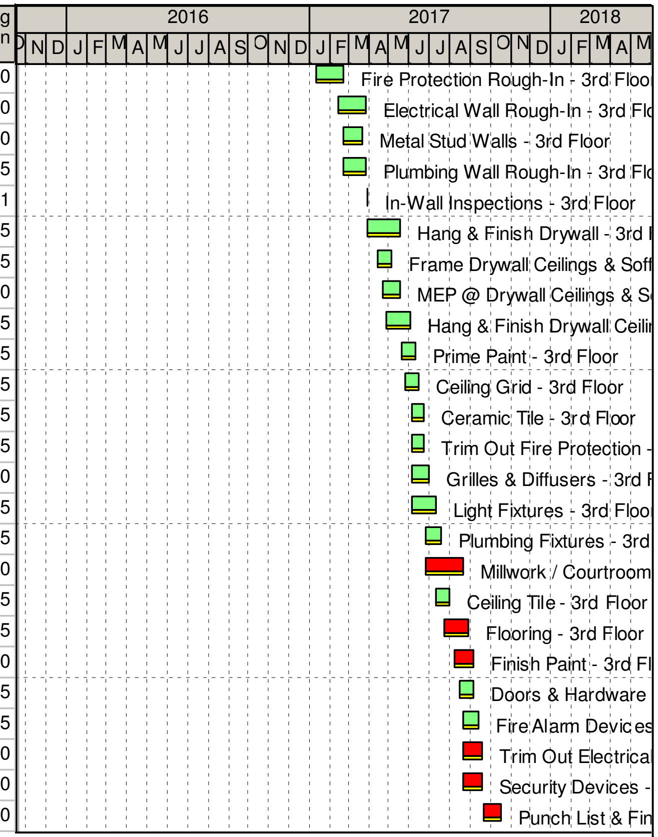
Activity ID	Activity Name	Original Duration	BL Project Start	Actual Start	BL Project Finish	Actual Finish	Remaining Duration
83120	Fire Protection Rough-In - 3rd Floor	30	1/11/17		2/21/17		30
83090	Electrical Wall Rough-In - 3rd Floor	30	2/15/17		3/28/17		30
83050	Metal Stud Walls - 3rd Floor	20	2/22/17		3/21/17		20
83100	Plumbing Wall Rough-In - 3rd Floor	25	2/22/17		3/28/17		25
83130	In-Wall Inspections - 3rd Floor	1	3/29/17		3/29/17		1
83140	Hang & Finish Drywall - 3rd Floor	35	3/30/17		5/18/17		35
83150	Frame Drywall Ceilings & Soffits - 3rd Floor	15	4/13/17		5/4/17		15
83160	MEP @ Drywall Ceilings & Soffits - 3rd Floor	20	4/21/17		5/18/17		20
83170	Hang & Finish Drywall Ceilings & Soffits - 3rd Floo	25	4/28/17		6/2/17		25
83190	Prime Paint - 3rd Floor	15	5/19/17		6/9/17		15
83210	Ceiling Grid - 3rd Floor	15	5/26/17		6/16/17		15
83180	Ceramic Tile - 3rd Floor	15	6/5/17		6/23/17		15
83250	Trim Out Fire Protection - 3rd Floor	15	6/5/17		6/23/17		15
83240	Grilles & Diffusers - 3rd Floor	20	6/5/17		6/30/17		20
83230	Light Fixtures - 3rd Floor	25	6/5/17		7/10/17		25
83270	Plumbing Fixtures - 3rd Floor	15	6/26/17		7/17/17		15
83260	Millwork / Courtroom Seating - 3rd Floor	40	6/26/17		8/21/17		40
83220	Ceiling Tile - 3rd Floor	15	7/11/17		7/31/17		15
83280	Flooring - 3rd Floor	25	7/25/17		8/28/17		25
83200	Finish Paint - 3rd Floor	20	8/8/17		9/5/17		20
83300	Doors & Hardware - 3rd Floor	15	8/15/17		9/5/17		15
83350	Fire Alarm Devices - 3rd Floor	15	8/22/17		9/12/17		15
83290	Trim Out Electrical - 3rd Floor	20	8/22/17		9/19/17		20
83340	Security Devices - 3rd Floor	20	8/22/17		9/19/17		20
83500	Punch List & Final Clean - 3rd Floor	20	9/20/17		10/17/17		20



Florence County Judicial Center
Preliminary Schedule

BE&K Building Group
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GENERAL NOTES

0. BUILDING CODE: 2012 INTERNATIONAL BUILDING CODE.
1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS PRIOR TO STARTING CONSTRUCTION AND THE OWNER'S REPRESENTATIVE SHALL BE NOTIFIED OF ANY DISCREPANCIES OR INCONSISTENCIES.
2. ALL DIMENSIONS TO TAKE PRECEDENCE OVER SCALE SHOWN ON PLANS, SECTIONS AND DETAILS.
3. NOTES AND DETAILS ON DRAWINGS SHALL TAKE PRECEDENCE OVER GENERAL NOTES AND TYPICAL DETAILS.
4. ALL ASTM SPECIFICATIONS NOTED ON THESE DRAWINGS SHALL BE OF THE LATEST REVISION UNLESS NOTED OTHERWISE.
5. THE CONTRACT STRUCTURAL DRAWINGS AND SPECIFICATIONS REPRESENT THE FINISHED STRUCTURE, UNLESS NOTED OTHERWISE. THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE, WORKMEN, OR OTHER PERSONS DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT NOT BE LIMITED TO, BRACING, SHORING FOR CONSTRUCTION EQUIPMENT, SHORING FOR THE BUILDING, SHORING FOR EARTH BANKS, FORM SCAFFOLDING, PLANKING, SAFETY NETS, SUPPORT AND BRACING FOR CRANES AND GINE POLES, ETC. THE CONTRACTOR SHALL SUPERVISE AND DIRECT THE WORK AND SHALL BE SOLELY RESPONSIBLE FOR ALL CONSTRUCTION MEANS, METHODS, TECHNIQUES, SEQUENCES, AND PROCEDURES, AS A PART OF THEIR RESPONSIBILITY.
6. OPENINGS, POCKETS, ETC. SHALL NOT BE PLACED IN SLABS, DECKS, BEAMS, JOISTS, COLUMNS, WALLS, ETC. UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL DRAWINGS. NOTIFY OWNER'S REPRESENTATIVE WHEN OTHER DRAWINGS SHOW OPENINGS, POCKETS, ETC. THAT ARE NOT LIKEWISE SHOWN ON THE STRUCTURAL DRAWINGS. HOLES 3" ROUND OR SQUARE (MAXIMUM) SPACED AT 2'-0" (MINIMUM) IN A FLOOR SLAB, ROOF SLAB (DECK), OR WALL SHALL BE EXEMPT FROM THIS REQUIREMENT. SLEEVES, INSERTS AND OTHER ITEMS TO BE CAST IN CONCRETE SHALL BE SET BY THE CONTRACTOR AT LOCATIONS DESIGNATED BY, AND UNDER THE SUPERVISION OF, A REPRESENTATIVE OF EACH TRADE.
7. SEE ARCHITECTURAL DRAWINGS FOR THE FOLLOWING: (A) SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS. (B) SIZE AND LOCATION OF ALL INTERIOR AND EXTERIOR NON-BEARING PARTITIONS. (C) SIZE AND LOCATION OF ALL CONCRETE CURBS, FLOOR DRAINS, SLOPES, DEPRESSED AREAS, ETC. (D) SIZE AND LOCATION OF ALL FLOOR AND ROOF OPENINGS. (E) FLOOR AND ROOF FINISHES.
8. SEE MECHANICAL, PLUMBING, AND ELECTRICAL DRAWINGS FOR THE FOLLOWING INFORMATION: (A) PIPE RUNS, SLEEVES, HANGERS, TRENCHES, WALL AND SLAB OPENINGS, ETC. (B) ELECTRICAL CONDUIT RUNS, BOXES, OUTLETS IN WALLS AND SLABS. (C) CONCRETE INSERTS FOR ELECTRICAL, MECHANICAL, OR PLUMBING FIXTURES. (D) MACHINE OR EQUIPMENT BASES, ANCHOR BOLTS FOR MOTOR MOUNTS. (E) UNDERGROUND CONCRETE DUCTS, TRENCHES, PITS, OR MANHOLES.
9. ALL HEAVY EQUIPMENT PIECES, SUCH AS COMPUTERS OR SERVERS, SAFES, FILE CABINETS, ETC. WITH A UNIT LOAD HIGHER THAN THE DESIGN LOAD SHALL NOT BE PLACED ON ANY FLOOR WITHOUT THE APPROVAL OF THE STRUCTURAL ENGINEER.
10. CONSTRUCTION MATERIALS SHALL BE SPREAD OUT IF PLACED ON FRAMED FLOORS OR ROOF. THE CONSTRUCTION MATERIAL LOAD SHALL NOT EXCEED THE DESIGN LIVE LOAD FOR EACH PARTICULAR LEVEL. ALLOWABLE CONSTRUCTION LIVE LOAD ON COMPOSITE FLOORS PRIOR TO SLAB PLACEMENT IS 20PSF.
11. EQUIVALENT MATERIALS SUBSTITUTED AS PER "APPROVED EQUAL" NOTE SHALL BE APPROVED BY ENGINEER OF RECORD BEFORE USE. ANY MATERIAL DESIGNATED WITH A BRAND NAME MAY BE SUBSTITUTED WITH ITS EQUAL IF THE SO CALLED EQUIVALENT IS FIRST APPROVED BY THE ENGINEER. THE CONTRACTOR SHALL SUPPLY INFORMATION AS REQUESTED TO VERIFY MATERIAL IS EQUAL TO SPECIFIED MATERIAL.
12. WHERE DETAILS SHOWN ON STRUCTURAL DRAWINGS FOR ONE CONDITION, IT SHALL APPLY TO ALL SIMILAR OR LIKE CONDITIONS, UNLESS NOTED OR SHOWN OTHERWISE ON PLANS.
13. MECHANICAL EQUIPMENT SHOWN ON STRUCTURAL DRAWINGS IS FOR REFERENCE ONLY. EXACT LOCATIONS AND UNIT SIZES ARE TO BE COORDINATED BY CONTRACTOR PRIOR TO CONSTRUCTION AND SUBMITTAL OF SHOP DRAWINGS. DESIGNATED ROOF FRAMING MAY BE ADJUSTED AS REQUIRED (WITH ENGINEERS APPROVAL) TO MATCH EQUIPMENT SELECTED.
14. DIMENSIONS AND LOCATIONS OF STRUCTURAL SUPPORT FOR HOLES IN ROOFS AND FLOOR SLABS SHALL BE COORDINATED BY THE GENERAL CONTRACTOR WITH THEIR RESPECTIVE TRADES PRIOR TO CONSTRUCTION/FABRICATION AND SUBMITTAL OF SHOP DRAWINGS.

DESIGN LOADS

DESIGN LOADS SHALL CONFORM TO 2012-IBC, UNO.

LIVE LOAD (WITH REDUCTIONS PER CODE, TYP):
TYPICAL LIVE LOAD FOR 1ST FLOOR= 100 PSF - 2000 LBS CONCENTRATED
CORRIDORS ABOVE 1ST FLOOR= 85 PSF - 1000 LBS CONCENTRATED
CORRIDORS ON 1ST FLOOR = 100 PSF - 1000 LBS CONCENTRATED
ASSEMBLY AREA FIXED SEATS AND ROOMS ON THE 2ND AND 3RD FLOORS = 85 PSF
STAIRS AND EXITS = 100 PSF

ROOF LIVE LOAD = 20 PSF

WIND LOADS:
DESIGN LOADS SHALL CONFORM TO ASCE 7-10.
BASIC WIND SPEED (V3s)= 140 MPH
BUILDING CATEGORY: III
WIND EXPOSURE: C
INTERNAL PRESSURE COEFF= +/- 0.18
COMPONENT AND CLADDING WIND PRESSURE:
PER ASCE 7-10

SEISMIC LOADS:
A. DESIGN ACCELERATION (SHORT PERIOD) , SDS = 0.51g
B. DESIGN ACCELERATION (1-SEC PERIOD), SD1 = 0.26g
C. SITE CLASS: D (PER GEOTECHNICAL REPORT)
D. SEISMIC USE GROUP/IMPORTANCE FACTOR: 1.25
E. SEISMIC DESIGN CATEGORY: D
F. BASIC STRUCTURAL SYSTEM AND SEISMIC RESISTING SYSTEM: LOAD BEARING STEEL FRAME WITH SPECIAL CONCRETE SHEARWALL SYSTEM
G. ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE
H. RESPONSE MODIFICATION FACTOR: MAIN BUILDING: R=6
I. DESIGN BASE SHEAR: 1220K

SNOW LOADS:
GROUND SNOW LOAD= 10PSF
FLAT ROOF SNOW LOAD= 11PSF
SNOW EXPOSURE, Ce=1.2
SNOW LOAD IMPORTANCE FACTOR, Is: 1.1
THERMAL FACTOR, Ct=1.2
RAIN ON SNOW UNIFORM LOAD = 16PSF

FOUNDATION NOTES

0. FOUNDATION DESIGN IS BASED ON THE RECOMMENDATIONS MADE IN THE GEOTECHNICAL REPORT PREPARED FOR THE SITE BY: S&ME DATED SEPTEMBER 30,2014.
1. FOUNDATION DESIGNS ARE BASED ON A 5000PSF ALLOWABLE BEARING PRESSURE AFTER SUBSURFACE MODIFICATION USING COMPACTED AGGREGATE PIERS. REFER TO RAMMED AGGREGATE PIER NOTES FOR ADDITIONAL INFORMATION.
2. SEE GEOTECHNICAL REPORT FOR SITE PREPARATION AND FOR DIRECTIONS ON PLACING FOUNDATIONS NEAR ROCK.
3. ALL EXCAVATION AND BACKFILLING PROCEDURES AND MATERIALS SHALL CONFORM TO RECOMMENDATIONS OUTLINED BY GEOTECHNICAL INVESTIGATION. THE TESTING AGENCY SHALL INSPECT AND APPROVE ALL PROCEDURES AND MATERIALS.
4. TEMPORARY DEWATERING SHALL BE PROVIDED AS REQUIRED FOR CONSTRUCTION OF THE FOUNDATIONS, AND GROUND FLOOR SLABS.
5. WHERE FINISHED GRADES DIFFER ON OPPOSITE SIDES OF WALL, BALANCE ADJACENT FILL AND COMPACTION ON EACH SIDE OF THE WALL TO THE TOP OF THE LOWEST GRADE BEFORE COMPLETING THE BALANCE OF THE FILL.
6. ALL SLABS AND FOUNDATIONS BEARING ON FILL MATERIAL SHALL BEAR ON APPROVED STRUCTURAL FILL AS SPECIFIED AND AS APPROVED BY THE TESTING AGENCY.
7. PROVIDE FOUNDATION DRAINS AS SHOWN ON STRUCTURAL, CIVIL AND/OR ARCHITECTURAL DRAWINGS.
8. PROVIDE WATERPROOFING, MAT DRAINAGE AND FILTER FABRIC ON ALL RETAINING WALLS AS SHOWN ON STRUCTURAL, CIVIL AND/OR ARCHITECTURAL DRAWINGS.
9. WHERE UTILITIES CROSS NEW FOUNDATIONS, STEP FOOTINGS BELOW THE UTILITY AND SLEEVE IN FOUNDATION WALL. DO NOT UNDERMINE.
10. PROVIDE EQUIPMENT PADS BELOW ALL EXTERIOR EQUIPMENT. MIN THICKNESS = 12" MIN OR GREATER AS INDICATED BY EQUIPMENT MANUFACTURER. PROVIDE TOE FOOTING AROUND PERIMETER PER TYPICAL DETAILS. REINFORCE AS INDICATED IN REINFORCING NOTES.
11. EXTEND ALL WALL FOOTING REINFORCING CLASS 'A' SPLICE (24" MIN) INTO ADJACENT COLUMN FOOTINGS.
12. TOP OF FOOTINGS BELOW FINISHED FLOOR INDICATED (-0'-0").
13. SEE ARCHITECTURAL DRAWINGS FOR EXTREMITIES OF CONCRETE SLABS AND LOCATIONS OF DEPRESSIONS REQUIRED FOR FLOOR FINISHES, WALK-OFF MATS, ETC.
14. PROVIDE VAPOR RETARDER AND GRANULAR BASE UNDER ALL CONCRETE SLABS ON GRADE.
15. FOOTINGS ARE LOCATED AT COLUMN LINES OR CENTER OF WALLS UNLESS SHOWN OTHERWISE ON PLANS.
16. FENCE, GATE, FLAG POLE, AND LIGHT POLE BASES/SUPPORTS AND ATTACHMENTS ARE TO BE DESIGNED BY THE RESPECTIVE DESIGNER/SUPPLIER UNDER THE DIRECT SUPERVISION OF AN ENGINEER REGISTERED IN THE STATE OF SOUTH CAROLINA UNLESS OTHERWISE DETAILED IN THE DOCUMENTS.
17. FREESTANDING CANOPY FOUNDATIONS NOT INDICATED ON THE DRAWINGS ARE TO BE DESIGNED BY THE RESPECTIVE DESIGNER/SUPPLIER UNDER THE DIRECT SUPERVISION OF AN ENGINEER REGISTERED IN THE STATE OF SOUTH CAROLINA.

COMPACTED AGGREGATE PIERS

0. COMPACTED AGGREGATE PIERS ARE TO BE DESIGNED TO ACHIEVE A MINIMUM BEARING PRESSURE OF 5000 PSF AND HOLD DIFFERENTIAL SETTLEMENTS TO LESS THAN 3/4".
1. DESIGN OF THE COMPACTED AGGREGATE PIERS SHALL BE PERFORMED BY A QUALIFIED FOUNDATION DESIGN ENGINEER REGISTERED IN THE PROJECT STATE.
2. SIGNED & SEALED CALCULATIONS AND SHOP SHOWING SIZES, LOCATIONS, MINIMUM DEPTH BELOW EXISTING GRADE ELEVATIONS, INSTALLATION PROCEDURES, ETC. SHALL BE SUBMITTED TO THE ENGINEER OF RECORD FOR REVIEW AND APPROVAL. A LETTER CERTIFYING THE DESIGN WILL ACHIEVE A 5000 PSF BEARING CAPACITY SHOULD BE SUBMITTED WITH THE SHOP DRAWINGS.
3. REFER TO GEOTECHNICAL REPORT PREPARED BY S&ME (S&ME PROJECT #1439-14-021) DATED 9.30.2014 FOR RECOMMENDED SIZES, SPACING AND DEPTHS, ALONG WITH ADDITIONAL GEOTECHNICAL INFORMATION.
4. STONE UTILIZED IN COMPACTED AGGREGATE PIERS SHALL BE #57 GRADATION, VARIATIONS WOULD BE ACCEPTABLE WITH WRITTEN APPROVAL.
5. MAINTAIN AT LEAST A SOIL CLASSIFICATION OF "D".
6. AGGREGATE PIERS ARE NOT REQUIRED AT THE SCREEN WALLS, EXTERIOR RETAINING WALLS AND INTERIOR STRIP FOOTINGS PROVIDED THE SOIL IN THESE AREAS CAN SUSTAIN 3000PSF.
7. ANY ADDITIONAL GEOTECHNICAL FIELD INVESTIGATION, SAMPLING, TESTING, ETC. BEYOND THE INFORMATION PROVIDED IN THE GEOTECHNICAL REPORT WILL BE THE COMPACTED AGGREGATE PIER SUBCONTRACTOR'S RESPONSIBILITY AND SHOULD BE COORDINATED WITH THE OWNER AND GENERAL CONTRACTOR.

FUTURE EXPANSION

0. FUTURE STRUCTURAL EXPANSION OF THE BUILDING HAS NOT BEEN CONSIDERED IN THE GRAVITY OR LATERAL DESIGN OF THIS STRUCTURE.

033000 CONCRETE NOTES

0. CAST-IN-PLACE CONCRETE MIX DESIGNS SHALL BE AS NOTED:
   A) 3000 PSI - FOUNDATIONS
   B) 4000 PSI - PIERS, FOUNDATION WALLS, COLUMNS, BEAMS, ELEVATED SLABS & SLABS ON GRADE
   C) 5000 PSI - SHEARWALLS/MAT FOUNDATIONS
   D) 4000 PSI - AIR ENTRAINED - EXTERIOR SLABS ON GRADE
   E) 3500 PSI - SEMI LT-WT (120 PCF) - COMPOSITE SLABS AND SLABS ON DECK
1. NO CALCIUM CHLORIDE SHALL BE USED IN ANY CONCRETE.
2. THE CONTRACTOR SHALL SUBMIT FOR APPROVAL DETAILED DRAWINGS SHOWING THE LOCATION OF ALL CONSTRUCTION JOINTS, CURBS, SLAB DEPRESSIONS, SLEEVES, OPENINGS, ETC. IN ALL CONCRETE WORK.
3. CONSTRUCTION JOINTS IN ELEVATED FRAMING SHALL BE MADE AT THE THIRD POINT OF SPAN WITH VERTICAL BULKHEADS PARALLEL TO THE PRIMARY DIRECTION OF FRAMING.
4. PROVIDE KEYS IN SLABS OR BEAMS AT CONSTRUCTION JOINTS.
5. ALL REINFORCING SHALL BE CONTINUOUS THROUGH CONSTRUCTION JOINTS.
6. ALL CONSTRUCTION JOINTS SHALL BE WIRE BRUSHED, CLEANED, AND COATED WITH CONCRETE BONDING AGENT IMMEDIATELY PRIOR TO PLACING NEW CONCRETE.
7. THERE SHALL BE NO HORIZONTAL CONSTRUCTION JOINTS IN SLABS AND BEAMS.
8. FORMS SHALL BE CAMBERED AS INDICATED. CAMBER SHALL NOT BE ACHIEVED BY ADDING THICKNESS TO SLAB OR BEAMS.
9. REFER TO ARCHITECTURAL DRAWINGS FOR CLIPS, GROOVES, GROUNDS, ETC. TO BE CAST IN CONCRETE AND FOR CONCRETE FINISHES.
10. HANGER INSERTS IN CONCRETE SLAB SHALL BE PLACED SO THAT 1" CONCRETE COVER OCCURS BETWEEN INSERT AND TOP OF SLAB CONDUIT SHALL BE PLACED UNDER THE SLAB. NO CONDUITS SHALL BE PERMITTED TO RUN HORIZONTALLY IN COMPOSITE OR FORMED SLABS OR SLABS ON GRADE.
11. SLEEVE PLUMBING OPENINGS IN CONCRETE WALLS AND SLABS BEFORE PLACING CONCRETE WITH SCHEDULE 40 GALVANIZED STEEL MIN. ADJUST REINFORCING AT SLEEVES TO PROVIDE REQUIRED COVER TO REINFORCING. CORING IS NOT PERMITTED IN FLOOR SLABS, ROOF SLABS, COLUMNS, BEAMS, AND WALLS UNLESS PERMITTED BY OWNER'S REPRESENTATIVE. SPACINGS SHALL COMPLY WITH GENERAL NOTE #6
12. ALL REINFORCING BARS, ANCHOR BOLTS, AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION PRIOR TO PLACING CONCRETE. CONTRACTOR SHALL PROVIDE TESTING AGENCY PROPER STORAGE FACILITIES FOR CONCRETE TEST CYLINDERS TO MAINTAIN CYLINDERS BETWEEN 60° AND 80°F AND IN A MOIST CONDITION.
13. CONDUIT SHALL NOT BE PLACED IN CAST-IN-PLACE CONCRETE. ANY EXCEPTIONS ARE TO BE AS INDICATED BY THE ENGINEER OF RECORD AND COORDINATION DRAWINGS SHOWING THE EXACT NUMBER, SIZE AND LOCATION OF CONDUIT ARE TO BE APPROVED BY THE ENGINEER OF RECORD.
14. ALL EXPOSED EDGES OF CONCRETE SHALL HAVE A 3/4" CONTINUOUS CHAMFER. (SLABS, BEAMS, COLUMNS, AND WALLS, ETC.)
15. BASE GROUT SHALL BE NON-SHRINK GROUT CONFORMING TO ASTM C1107. USE NON-METALLIC GROUT AT EXPOSED LOCATIONS. 8000PSI MIN.
16. COLUMN BASE GROUT SHALL BE PLACED AND ALLOWED TO CURE AFTER COLUMNS HAVE BEEN PLUMBED AND BEFORE ELEVATED SLABS AND COLUMN ISOLATION JOINTS HAVE BEEN CAST.
17. ALL CONCRETE WALLS, BEAMS, RAILS, ETC. SHALL HAVE CORNER BARS SAME SIZE AND SPACING AS HORIZONTAL REINFORCEMENT, UNLESS NOTED OTHERWISE.
18. PROVIDE AND INSTALL ALL PLATES, ANGLES, REINFORCING, ETC., EMBEDDED IN CAST-IN PLACE CONCRETE.
19. ALL CONCRETE SLABS TO SLOPE TO FLOOR DRAINS, IN ROOMS OR AREAS THAT HAVE FLOOR DRAINS. SEE ARCHITECTURAL PLANS AND PLUMBING PLANS FOR LOCATIONS.
20. AT TILE FINISHED AREAS, LOCATE ALL GRADE SLAB CONSTRUCTION JOINTS AT TILE JOINTS. IN ADDITION, PROVIDE A CONTROL JOINT AT ALL OTHER TILE JOINTS AND VERIFY LOCATION OF TILE FINISHED AREAS AND JOINT PATTERN WITH ARCHITECTURAL DRAWINGS.

033000 REINFORCING NOTES

0. REINFORCING BARS SHALL CONFORM TO THE FOLLOWING:
   A. DEFORMED BARS CONFORMING TO ASTM A706, GRADE 60.
   B. COLD-ROLLED, DEFORMED BAR ANCHORS CONFORMING TO ASTM A496 WITH TENSILE STRENGTH OF 80,000.
1. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A185 PLAIN WIRE OR A497 DEFORMED WIRE.
2. CONCRETE REINFORCEMENT SHALL BE DETAILED, FABRICATED, LABELED, SUPPORTED, AND SPACED IN FORMS, AND SECURED IN PLACE IN ACCORDANCE WITH THE PROCEDURES AND REQUIREMENTS OUTLINED IN THE LATEST EDITION OF THE "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE", ACI 318 AND THE "ACI DETAILING MANUAL - LATEST EDITION", ACI SP-66. CHECKED SHOP DRAWINGS SHOWING REINFORCING DETAILS, INCLUDING CONCRETE REINFORCING SIZES, SPACING, AND LOCATION SHALL BE SUBMITTED FOR APPROVAL. ALL REINFORCING SHOWN ON THE PLACEMENT DRAWINGS (PLANS, DETAILS, AND ELEVATIONS) SHALL HAVE A UNIQUE MARK AND SHALL BE LISTED SEPARATELY SHOWING LENGTHS, QUANTITIES, AND BAR BENDING DETAILS.
3. CONTRACTOR SHALL NOT PLACE ANY REINFORCING UNTIL SHOP DRAWINGS, APPROVED, BY THE STRUCTURAL ENGINEER ARE RECEIVED AT THE JOB SITE.
4. ALL WELDED WIRE FABRIC (WWF) SHALL BE LAPPED TWO (2) FULL MESH PANELS AND TIED SECURELY. WELDED WIRE FABRIC SHALL BE PROVIDED IN FLAT SHEETS, NOT ROLLS.
5. WHERE VERTICAL WALL REINFORCING IS SPLICED AT TOP OF FOOTING, PROVIDE SPLICE BARS IN FOOTING SAME SIZE, GRADE, AND SPACING AS VERTICAL WALL REINFORCING, UNO. PROVIDE STANDARD HOOK IN FOOTING AND LAP WITH VERTICAL WALL REINFORCING AS NOTED ABOVE.
6. AT COLUMNS AND PIERS WHERE VERTICAL REINFORCING IS SPLICED AT TOP OF FOOTING, PROVIDE SPLICE BARS IN FOOTING SAME SIZE, GRADE, AND QUANTITY AS VERTICAL COLUMN OR PIER REINFORCING. PROVIDE STANDARD HOOK IN FOOTING AND LAP WITH COLUMN OR PIER REINFORCING PER SPLICE LENGTHS. MECHANICAL SPLICES ARE REQUIRED FOR #14 AND #18 BARS AND WHERE INDICATED.
7. FOR BEAMS AND SLABS, THE MINIMUM CLEAR DISTANCE BETWEEN PARALLEL BARS SHALL BE THE DIAMETER OF THE BAR, 1.333 TIMES THE MAXIMUM AGGREGATE SIZE, BUT IN NO CASE LESS THAN 1". WHERE TWO OR MORE LAYERS OF REINFORCING ARE USED, PROVIDE #8 SPACERS AT 4'-0" ALONG BEAM. FOR COLUMNS AND WALLS, THE MINIMUM CLEAR DISTANCE BETWEEN BARS SHALL BE 1-1/2 BAR DIAMETERS, BUT IN NO CASE LESS THAN 1-1/2".
8. PLACEMENT OF REINFORCEMENT SHALL BE SUCH THAT ADEQUATE SPACE IS PROVIDED BETWEEN BARS TO ALLOW ASSAGE OF CONCRETE, VIBRATORS, ETC.
9. BARS SHALL BE IN CONTACT WHEN FORMING LAP SPLICES, UNO.
10. ALL REINFORCEMENT SHALL BE BENT COLD, UNO.
11. CONCRETE SLABS ON GRADE SHALL BE REINFORCED WITH #3@18" EW AT 1 1/2" CLEAR FROM TOP, UNO ON THE PLANS.
12. ADDITIONAL BARS SHALL BE PROVIDED AROUND ALL FLOOR AND WALL OPENINGS AS SHOWN ON DETAILS, ETC.
13. WHERE REQUIRED, PROVIDE MECHANICAL SPLICE IN CONFORMANCE WITH ACI 318. SUBMIT ENGINEERING DATA INCLUDING INSTALLATION PROCEDURES, TESTING PROCEDURES, AND SAMPLES FOR APPROVAL PRIOR TO INSTALLATION OF ANY MECHANICAL SPLICES. MECHANICAL SPLICES MAY SUBSTITUTEFOR LAP SPLICES FOR BARS OTHER THAN #14 AND #18 FOLLOWING THE REQUIREMENTS LISTED ABOVE.
14. ALL CONCRETE WALLS, BEAMS, RAILS, ETC. SHALL HAVE CORNER BARS SAME SIZE AND SPACING AS HORIZONTAL REINFORCEMENT, UNLESS NOTED OTHERWISE.
15. PROVIDE AND INSTALL ALL PLATES, ANGLES, REINFORCING, ETC., EMBEDDED IN CAST-IN PLACE CONCRETE.
16. CLEAR COVERAGE OF CONCRETE OVER OUTER MAIN REINFORCING BARS SHALL BE AS FOLLOWS, UNO:
   A. CONCRETE POURED DIRECTLY AGAINST EARTH: 3"
   B. STRUCTURAL SLABS: 3/4" TOP AND BOTTOM
   C. FORMED CONCRETE AGAINST EARTH: 2"
   D. WALLS WITH 2 LAYERS OF REINF: INTERIOR FACE: 3/4" EXTERIOR FACE: 2"
   E. WALLS WITH ONE LAYER OF REINF: PLACE VERTICAL BAR AT CENTER OF WALL AND HORIZ BAR ON ONE SIDE.
   F. COLUMNS (VERTICAL REINFORCING): 2" OR EQUAL TO THE BAR DIAMETER, WHICHEVER IS GREATER
   G. BEAMS (TOP AND BOTT REINF): 2" OR EQUAL TO THE BAR DIAMETER, WHICHEVER IS GREATER
17. EQUIPMENT PADS SHALL BE REINFORCED AS FOLLOWS, UNO:
   A. 6" PADS: #4 AT 12" EACH WAY CENTERED IN SLAB THICKNESS.
   B. 8" PADS: #4 AT 12" EACH WAY CENTERED IN SLAB THICKNESS.
   C. 12" PADS: #4 AT 12" EACH WAY TOP & BOTT.
18. THE FOLLOWING REINFORCING SHALL BE USED IN CONCRETE WALLS, UNO:
   A. 8" WALLS: ONE LAYER OF REINF: #6 AT 12" HORIZ REINF; #6 AT 8" VERT REINF
   B. 12" WALLS: TWO LAYERS OF REINF: #6 AT 12" HORIZ REINF EACH FACE; #6 AT 8" VERT REINF EACH FACE
   C. 16" WALLS: TWO LAYERS OF REINF: #6 AT 12" HORIZ REINF EACH FACE; #6 AT 8" VERT REINF EACH FACE
19. IN SLABS AND BEAMS, SPLICES FOR REINFORCING SHALL NOT BE AT POINTS OF MAXIMUM STRESS WITHOUT THE APPROVAL OF THE OWNER'S REPRESENTATIVE. FOR ALL CONCRETE WORK, LAP SPLICES, WHERE PERMITTED, SHALL CONFORM TO THE FOLLOWING, UNO:
   #3 BAR- 20" #8 BAR- 63"
   #4 BAR- 26" #9 BAR- 72"
   #5 BAR- 32" #10 BAR- 80"
   #6 BAR- 38" #11 BAR- 89"
   #7 BAR- 55"

042000 MASONRY NOTES

0. ALL MASONRY CONSTRUCTION SHALL CONFORM TO THE 2012 INTERNATIONAL BUILDING CODE, CHAPTER 21, SECTION 2104, ACI 530-10 "BUILDING CODE REQUIREMENTS FOR MASONRY STRUCTURES", AND ACI 530.1, "SPECIFICATION FOR MASONRY STRUCTURES".
1. CONCRETE MASONRY UNITS SHALL CONFORM TO THE FOLLOWING:
   A. LOAD BEARING AND NON-LOAD BEARING MASONRY WALLS: ASTM C90 HOLLOW UNITS.
   B. CONCRETE BUILDING BRICK: ASTM C55
2. MORTAR FOR CMU SHALL CONFORM TO ASTM C270:
   A. TYPE "S" MORTAR, UNO. MASONRY CEMENT MORTAR IS NOT PERMITTED FOR CMU. USE PORTLAND-LIME MORTAR OR MORTAR CEMENT MORTAR
3. THE MINIMUM COMPRESSIVE STRENGTH FOR ALL MASONRY SHALL BE AS FOLLOWS, UNO: 8"&12" CMU- f'm = 1900 PSI
4. GROUT FOR ALL MASONRY SHALL CONFORM TO ASTM C476:
   A. PROVIDE FINE GROUT IN GROUT SPACES LESS THAN 2" IN ANY HORIZONTAL DIMENSION OR WHERE CLEARANCE BETWEEN REINFORCING AND MASONRY IS LESS THAN 3/4"
   B. PROVIDE COURSE GROUT IN SPACES 2" OR GREATER IN ALL HORIZONTAL DIMENSIONS PROVIDED THE CLEARANCE BETWEEN REINFORCING AND MASONRY IS NOT LESS THAN 3/4"
   C. THE MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS FOR ALL GROUT SHALL BE 3000 PSI, UNO. SUBMIT GROUT MIX DESIGNS INCLUDING MANUFACTURER'S CERTIFICATION FOR MATERIALS USED PRIOR TO THE START OF ANY MASONRY WORK.
5. ALL REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60, DEFORMED BARS.
6. ALL TESTING AND INSPECTION OF MASONRY WORK SHALL BE AS SPECIFIED IN ACI 530.105.
   A. WALLS- INSPECT EACH SECTION OF WALL AND VERIFY REINFORCEMENT PLACEMENT PRIOR TO GROUTING OPERATIONS. VERIFY THAT VERTICAL CELLS AND BOND BEAMS TO RECEIVE GROUT ARE CLEANED OUT TO RECEIVE GROUT.
   B. GROUT-OBSERVE GROUT OPERATIONS TO INSURE CONFORMANCE WITH PLANS AND SPECIFICATIONS.
7. PROVIDE DOWELS TO FOOTINGS AT REINFORCED BLOCK CELLS, SAME SIZE AND SPACING AS INDICATED ON DRAWINGS.
8. FOR BRICK EXPANSION AND CONTROL JOINTS SEE ARCHITECTURAL DRAWINGS. FOR CMU CONTROL JOINTS, SPACE AT 25'-0" OC MAX UNLESS NOTED OTHERWISE. DO NOT PLACE CONTROL JOINTS IN SHEAR WALLS. CONTROL JOINTS ARE NOT REQUIRED BELOW GRADE. CONTROL JOINTS ARE TO BE LOCATED A MINIMUM OF 8" AWAY FROM THE EDGE OF THE BEARING PLATE.
9. ALL HOLES THROUGH MASONRY-BEARING WALLS SHALL BE PRE-PLANNED BY ALL CONTRACTORS USING AND LOCATION APPROVED BY THE ARCHITECT-ENGINEER PRIOR TO CONSTRUCTING WALLS. ALL HOLES MUST HAVE ADEQUATE ENGINEER APPROVED LINTELS. LINTELS TO BE DETERMINED BY SIZE AND LOCATION OF OPENING.
10. SOLID GROUT ALL POCKETS FOR STRUCTURAL ELEMENTS AFTER PLACEMENT.
11. SEE SPECIFICATIONS FOR ADDITIONAL REQUIREMENTS
12. SEE SECTIONS & DETAILS FOR REINFORCING ARRANGEMENTS AND/OR SPECIAL CONDITIONS.
13. SEE ARCH FOR BRICK VENEER CONSTRUCTION
14. GROUT KEYS SHALL BE FORMED BETWEEN GROUT LIFTS WHEN THE FIRST LIFT IS PERMITTED TO SET PRIOR TO PLACEMENT OF THE SUBSEQUENT LIFT. GROUT KEYS ARE NOT TO BE USED WITHIN BEAMS/LINTELS OR BENEATH CLOSED BOTTOM UNITS WHERE SPECIFIED.
15. GROUT KEYS SHALL BE FORMED BY TERMINATING THE GROUT A MINIMUM OF 1 1/2" BELOW THE MORTAR JOINT.
16. PLACE BAR(S) IN END CELL OF ALL JAMB OPENINGS, CORNERS, AND END CELLS AT CONTROL JOINTS, AND SPACE BARS AS INDICATED ON PLANS BETWEEN JAMBS, CORNERS, AND/OR CONTROL JOINTS.
17. PROVIDE A 2 1/2" CLEAR COVER FROM EXTERIOR FACE OF MASONRY TO REINFORCING FOR ALL DOUBLY REINFORCED MASONRY WALLS.
18. MASONRY PIERS ARE TO BE CONTINUOUS BETWEEN FLOORS. DEVELOP REINFORCEMENT INTO FOOTINGS OR LOWER FLOOR WALLS AND FLOOR/PARAPET ABOVE. PENETRATIONS OF MASONRY PIERS ARE NOT PERMITTED
19. SLEEVES THRU MASONRY WALLS ARE TO BE SCHEDULE 40 GALVANIZED STEEL MIN. SOLID GROUT ALL VOIDS AROUND SLEEVE. POSITION SLEEVES TO AVOID INTERFERENCE WITH VERTICAL AND HORIZONTAL REINFORCING.
20. MASONRY OFF OF ELEVATED SLABS IS TO BE CONSTRUCTED OF LIGHTWEIGHT UNITS.
21. SPLICE LENGTHS FOR MASONRY REINFORCEMENT SHALL BE AS FOLLOWS, UNO:

CMU SPLICE LENGTH SCHEDULE		
BAR SIZE	SPLICE/DEVELOPMENT LENGTH (Ld)	
	1 BAR/CELL	2 BAR/CELL
#3	18"	18"
#4	24"	24"
#5	26"	35"
#6	43"	66"
#7	60"	-

NOTES:
1. SPLICE LENGTHS ARE BASED ON THE REINFORCEMENT CENTERED IN THE CELL FOR 1 BAR/ CELL & 2 1/2" CLEAR COVER FOR 2 BAR/CELL
2. SPLICE LENGTH ARE BASED ON A CMU COMPRESSIVE STRENGTH OF 1900 PSI.
3. MECHANICAL COUPLERS CAN BE USED @ CONTRACTORS OPTION.

054000 LIGHT GAUGE ROOF TRUSS NOTES

1. THE STRUCTURAL DESIGN OF THE TRUSS SYSTEM SHALL BE PERFORMED BY OR UNDER THE DIRECT SUPERVISION OF A PROFESSIONAL ENGINEER REGISTERED IN THE PROJECT STATE.
2. SEALED AND SIGNED SHOP DRAWINGS AND CALCULATIONS SHALL BE SUBMITTED FOR APPROVAL BY THE ARCHITECT AND ENGINEER OF RECORD.
3. SHOP DRAWINGS ARE TO CLEARLY INDICATED ALL TRUSS CONNECTION DETAILS AND CONNECTIONS TO OTHER STRUCTURAL ELEMENTS.
4. ALL METAL COMPONENTS 16 GAUGE OR LIGHTER ARE TO HAVE A MINIMUM YIELD STRENGTH OF 33 KSI.
5. ALL METAL COMPONENTS 16 GAUGE OR THICKER ARE TO HAVE A MINIMUM YIELD STRENGTH OF 50 KSI.
6. SCREWS ARE TO BE "HILTI" SELF DRILLING SCREWS TYPE 12-24 x 7/8" HWH WITH #4 POINT OR APPROVED EQUIVALENT, MINIMUM.
7. FASTEN METAL DECKING AND OTHER METAL MEMBERS TO TRUSSES WITH "HILTI" SELF DRILLING SCREWS TYPE 12-24x7/8"HWH WITH #4 POINT OR EQUAL.
8. ALL CLIP ANGLES AND METAL CHANNELS AT TRUSS BEARINGS TO BE 14 GAUGE MINIMUM, FASTENED WITH A MINIMUM OF SIX "HILTI" SELF DRILLING SCREWS TYPE 12-24x7/8" HWH WITH #4 POINT OR EQUAL AT EACH LEG OF THE ANGLE OR CHANNEL UNLESS NOTED OTHERWISE ON THE PLANS.
9. TRUSS BRACING CONFIGURATION TO BE COORDINATED WITH UTILITIES.
10. BRACING AND CONNECTIONS AT TRUSS SUPPORTS ARE TO BE DESIGNED FOR A MINIMUM 150 PLF HORIZONTAL OUT OF PLANE FORCE IN THE DIRECTION OF THE SUPPORTING ELEMENT UNLESS NOTED OTHERWISE.
11. ALL TOP CHORDS ARE TO ALIGN FOR A SMOOTH PLANE. CAMBER TRUSS ELEMENTS/COMPONENTS AS REQUIRED.
12. TOP AND BOTTOM CHORDS ARE TO CONFORM TO THE CONFIGURATION SHOWN IN WALL SECTIONS AND BUILDING SECTIONS, REFER TO ARCHITECTURAL AND STRUCTURAL DRAWINGS.
13. COORDINATE WITH MECHANICAL CONTRACTOR FOR ANY DUCTWORK WHICH WILL REQUIRE SPECIFIC WEB MEMBER CONFIGURATION.

POST INSTALLED ANCHOR NOTES

1. POST-INSTALLED ANCHORS/EPOXY SHALL ONLY BE USED WHERE SPECIFIED ON THE DRAWINGS.
2. CONTRACTOR SHALL OBTAIN APPROVAL FROM ENGINEER PRIOR TO USING POST-INSTALLED ANCHORS/EPOXY FOR MISSING OR MISPLACED CAST-IN-PLACE ANCHORS.
3. CARE SHALL BE GIVEN TO AVOID CONFLICTS WITH EXISTING REBAR. HOLES SHALL BE DRILLED AND CLEANED PER THE MANUFACTURER'S INSTRUCTIONS.
4. ANCHORS SHALL BE INSTALLED PER THE MANUFACTURER'S INSTALLATION INSTRUCTIONS AT NOT LESS THAN MINIMUM EDGE DISTANCES AND/OR SPACINGS INDICATED IN THE MANUFACTURER'S LITERATURE.
5. UNLESS SPECIFIED OTHERWISE, ANCHORS SHALL BE EMBEDDED IN THE APPROPRIATE SUBSTRATE WITH A MINIMUM EMBEDMENT OF 8 TIMES THE NOMINAL ANCHOR DIAMETER OR THE EMBEDMENT REQUIRED TO SUPPORT THE INTENDED LOAD.
6. SUBSTITUTION REQUESTS, FOR PRODUCTS OTHER THAN THOSE LISTED BELOW, SHALL BE SUBMITTED TO THE ENGINEER WITH CALCULATIONS THAT ARE PREPARED & SEALED BY A REGISTERED PROFESSIONAL ENGINEER SHOWING THAT THE SUBSTITUTED PRODUCT WILL ACHIEVE AN EQUIVALENT CAPACITY USING THE APPROPRIATE DESIGN PROCEDURE REQUIRED BY THE BUILDING CODE.
7. ALL POST INSTALLED ANCHORS ARE SUBJECT TO CONTINUOUS SPECIAL INSPECTION OF THE INSTALLATION. INSPECTION COSTS INCURRED BY THE OWNER FOR POST INSTALLED ANCHORS REQUIRED DUE TO CONTRACTOR ERROR SHALL BE REIMBURSED TO THE OWNER BY THE CONTRACTOR
8. ACCEPTABLE PRODUCTS:
   ADHESIVE ANCHORS:
   SIMPSON STRONG-TIE: SET-XP ADHESIVE
   POWER'S: PE1000 EPOXY
   HILTI: HIT-R 500-SD
   MECHANICAL ANCHORS:
   SIMPSON STRONG-TIE: STRONG-BOLT
   POWERS: POWER STUD SD2
   HILTI: KWIK BOLT TZ

051200 STRUCTURAL STEEL NOTES

0. ALL STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING (UNO):
   A. ASTM A36 FOR ANGLES, CHANNEL & PLATES UNO.
   B. ASTM A992 OR A572, GRADE 50 (Fy = 50 KSI) FOR ALL WIDE FLANGE SECTIONS.
   C. ASTM A500, GRADE B FOR STRUCTURAL STEEL TUBING (NOTED HSS...)
   D. ASTM A53, TYPE S, GRADE B FOR STRUCTURAL STEEL PIPE (NOTED PIPE)
   E. ASTM A36 OR A572 GR50 FOR HP SHAPES
   F. ASTM A572 FOR BRACING CONNECTIONS
   G. ASTM F1554 FOR ANCHOR BOLTS (GR. 50 U.N.O.)
   G. ASTM A572 GR 50 FOR PLATES AND GUSSETS USED IN MOMENT FRAMES OR BRACING WHERE STEEL MATERIAL IS NOT INDICATED, OR NOTED ABOVE STEEL SHALL BE A36.
1. STEEL FABRICATOR TO BE AISC CERTIFIED. REFER TO SPECIFICATIONS.
2. ALL BOLTS, NUTS, AND WASHERS SHALL CONFORM TO THE REQUIREMENTS OF ASTM A325 OR ASTM A490. ALL BOLTS SHALL BE 3/4" DIAMETER UNO.
3. ALL WELDING ELECTRODES SHALL BE E70XX EXCEPT ELECTRODES FOR WELDING METAL DECK SHALL BE E60XX.
4. ALL DETAILING, FABRICATION, AND ERECTION SHALL CONFORM TO AISC SPECIFICATIONS AND CODES, CURRENT EDITION.
5. FABRICATORS SHALL BE CERTIFIED BY AISC.
6. ALL CONNECTIONS SHALL BE DESIGNED AND DETAILED BY THE FABRICATOR'S ENGINEER. DETAILING SHALL BE PERFORMED USING RATIONAL ENGINEERING DESIGN AND STANDARD PRACTICE IN ACCORDANCE WITH THE CONTRACT DOCUMENTS. THE GENERAL DETAILS SHOWN ON THE DRAWINGS ARE CONCEPTUAL ONLY AND DO NOT INDICATE THE REQUIRED NUMBER OF BOLTS OR WELD SIZES, UNO.
7. THE MINIMUM NUMBER OF BOLTS PER CONNECTION SHALL BE TWO (2).
8. MINIMUM FILLET WELDS SHALL COMPLY WITH AISC BUT SHALL NOT BE LESS THAN 3/16", UNO.
9. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE CONTROL OF ALL ERECTION PROCEDURES AND SEQUENCES WITH RELATION TO TEMPERATURE DIFFERENTIALS, ESPECIALLY WITH RESPECT TO STRUCTURAL STEEL FRAMING INTO MASONRY OR CONCRETE WALLS, BEAMS, OR COLUMNS.
10. ALL WELDING SHALL BE DONE BY QUALIFIED WELDERS AND SHALL CONFORM TO THE AWS D1.1, "STRUCTURAL WELDING CODE - STEEL", CURRENT EDITION.
11. ALL BEAMS AND JOISTS SHALL BE FABRICATED WITH A NATURAL CAMBER UP. PROVIDE CAMBERS AS INDICATED ON THE DRAWINGS.
12. THERE SHALL BE NO FIELD CUTTING OF STRUCTURAL STEEL MEMBERS FOR THE WORK OF OTHER TRADES WITHOUT THE PRIOR APPROVAL OF THE OWNER'S REPRESENTATIVE
13. ALL ADDITIONAL STEEL REQUIRED BY THE CONTRACTOR FOR ERECTION PURPOSES AND SITE ACCESS OF STOCKPILED MATERIALS SHALL BE PROVIDED AT NO COST TO THE OWNER. ALL SUCH ADDITIONAL STEEL SHALL BE REMOVED BY THE CONTRACTOR UNLESS APPROVED BY THE OWNER IN WRITING.
14. DO NOT SHOP PRIME STEEL RECEIVING FIREPROOFING MATERIAL. REFER TO ARCHITECTURAL DRAWINGS FOR THICKNESS AND LOCATIONS OF FIREPROOFING.
15. STRUCTURAL STEEL EXPOSED IN OCCUPIED SPACES OR "FEATURED" SHALL BE ARCHITECTURALLY EXPOSED STRUCTURAL STEEL (AESS) AS INDICATED AND AS SHOWN ON ARCHITECTURAL DRAWINGS.
16. ALL STRUCTURAL STEEL (MEMBERS AND CONNECTIONS) PERMANENTLY EXPOSED TO WEATHER SHALL BE HOT-DIPPED GALVANIZED AFTER FABRICATION.
17. SLOTTED HOLES EQUAL TO OR GREATER THAN "SHORT SLOTTED HOLES" PER AISC TABLE J3.3 ARE NOT PERMITTED WITHOUT THE CONSENT OF THE ENGINEER OF RECORD.
18. SPLICING IN LOCATIONS NOT INDICATED ON THE CONTRACT DOCUMENTS IS PROHIBITED WITHOUT THE WRITTEN CONSENT OF THE ENGINEER OF RECORD PRIOR TO SUBMITTAL OF FABRICATION DRAWINGS.
19. ALL HSS TO HSS CONNECTIONS ARE TO BE COMPLETE JOINT PENETRATION WELDS USING APPROVED DETAIL PER AWS FIGURE 3.8 & TABLE 3.6
20. PROVIDE C8x8.2 FRAME (SPANNING BETWEEN JOISTS OR OTHER STRUCTURE) FOR ROOF DRAINS, EQUIPMENT, OR OPENINGS IN ROOF CAUSED BY EQUIPMENT NOT SHOWN ON STRUCTURAL DRAWINGS.
21. PROVIDE C12x20.7 SPANNING BETWEEN FRAMING (WELD TO TOP FLANGES OF FRAMING) AT ROOFTOP MECHANICAL EQUIPMENT TO SUPPORT CURB.
22. AFTER STEEL BASE PLATES HAVE BEEN PROPERLY LOCATED AND ALIGNED, GROUT PLATES SOLIDLY WITH GROUT WORKED UNDER STEEL TO COMPLETELY FILL SPACE.
23. IF NOT SHOWN ON CONTRACT DOCUMENTS PROVIDE A CONTINUOUS 1/4" FILLET WELD FOR ALL FIELD WELDS ON MISCELLANEOUS CONNECTIONS.
24. ALL BRACED FRAMES ARE PART OF THE SEISMIC LATERAL FORCE RESISTING SYSTEM (SLRS)
25. SEE PLANS FOR MOMENT FRAMES THAT ARE PART OF THE SLRS.
26. ALL SLRS CONNECTIONS ARE TO BE DESIGNED FOR THE LOADS REQUIRED IN THE AISC 341 PART 1. DESIGN BRACE FRAME CONNECTIONS USING THE UNIFORM FORCE METHOD AND RESOLVE REACTIONS TO MINIMIZE/ELIMINATE MOMENTS APPLIED TO COLUMNS AND BEAMS DUE TO CONNECTION ECCENTRICITIES.
27. DETAIL ALL SLRS CONNECTIONS AS REQUIRED BY THE AISC 341 PART 1.
28. SLRS GUSSET PLATES ARE TO BE A572 GRADE 50 PLATE.
29. ALL BOLTS USED IN THE SLRS ARE TO BE A490 (PRETENSIONED) WITH THE THREADS EXCLUDED FROM THE SHEAR PLANE.
30. FAYING SURFACE FOR BOLTED SLRS CONNECTIONS SHALL SATISFY THE REQUIREMENTS FOR SLIP-CRITICAL CONNECTIONS WITH A FAYING SURFACE WITH A CLASS 'A' COEFFICIENT OR HIGHER.
31. TYPICAL CONNECTIONS SHALL BE SIMPLE SHEAR CONNECTIONS UTILIZING HIGH-STRENGTH BOLTS IN BEARING-TYPE CONNECTIONS WITH THREADS INCLUDED IN THE SHEAR PLANE IN SINGLE SHEAR. THE CAPACITIES SHALL BE AS FOLLOWS:
   A. CONNECTIONS FOR NON-FRAME MEMBERS ARE AS INDICATED IN THE REACTION TABLE ON THIS SHEET OR AS INDICATED BELOW
   B. CONNECTIONS FOR NON-COMPOSITE BEAMS WITH REACTIONS NOT OTHERWISE INDICATED SHALL BE ADEQUATE TO PROVIDE FOR 0.6 TIMES THE MAXIMUM UNIFORMLY DISTRIBUTED LOAD FOR THAT SPAN BASED ON THE ALLOWABLE MOMENT CAPACITY OF THE BEAM, UNO.
   C. CONNECTIONS FOR COMPOSITE BEAMS WITH REACTIONS NOT OTHERWISE INDICATED SHALL BE DESIGNED FOR 1.35 TIMES THE CAPACITY FOR NON-COMPOSITE BEAMS, UNO.
   D. BEAMS SHORTER THAN 8'-0" MAY BE DESIGNED 20 KIPS UNO.
   E. REACTIONS INDICATED IN THE PLANS ARE UN-FACTORED REACTIONS (ASD), UNO
   F. CONNECTION DESIGN SHALL BE DONE PER THE AISC STEEL CONSTRUCTION MANUAL THIRTEENTH EDITION OR LATER.

051200 COMPOSITE BEAM AND SHEAR STUD NOTES

1. STUD LAYOUT SHALL BE PER AISC CODE, CHAPTER I.
2. HEADED SHEAR STUD CONNECTORS, WELDING, AND TESTING SHALL CONFORM TO THE REQUIREMENTS OF STRUCTURAL WELDING CODE - STEEL, AWS D1.1.
3. TOP FLANGE OF STRUCTURAL STEEL BEAMS AND SUPPORTS TO RECEIVE STUDS SHALL BE FREE OF PAINT, SCALE, RUST, AND OTHER SUBSTANCES WHICH WOULD BE DETRIMENTAL TO THE WELDING OF STUDS THROUGH DECK.
4. WHERE A CLOSURE PLATE OR MISCELLANEOUS STEEL ANGLE IS WELDED TO THE TOP FLANGE OF BEAM TO RECEIVE SHEAR STUDS, ATTACH STUDS DIRECTLY TO BEAM FLANGE AND NOT TO (OR THROUGH) ANGLES.
5. PROVIDE D2L/DBA ANCHORS WHERE INDICATED IN SECTIONS AND DETAILS. "STICK WELDING" OF REINFORCING IS NOT PERMITTED AS AN ALTERNATE.
6. FOR BEAMS UNDER COMPOSITE SLABS WITHOUT STUDS INDICATED, PROVIDE A MINIMUM OF 1 STUD PER 12".

055000 METAL FABRICATIONS

1. SEE SPECIFICATION 055000 FOR THE PERFORMANCE DESIGN, DETAILING AND PROVISION OF MISCELLANEOUS METALS NOT DETAILED IN THE STRUCTURAL CONTRACT DOCUMENTS. THIS INCLUDES, BUT SHALL NOT BE LIMITED TO, LADDERS, FRAMING/EMBEDS/HOIST BEAMS FOR THE SELECTED ELEVATOR SYSTEMS, ANGLE AND CHANNEL FRAMING FOR THE SUPPORT OF OVERHEAD DOORS, AND FRAMING FOR THE SUPPORT OF REVOLVING DOORS. DESIGNS SHALL COMPLY WITH IBC AND ASCE-7 DESIGN LOADS AND REQUIREMENTS.

053100 COMPOSITE METAL ROOF AND FLOOR DECK NOTES

0. WHERE SHEAR STUD SPACING EXCEEDS 16" O.C. OR NO SHEAR STUDS ARE PROVIDE AT DECK SUPPORT, PROVIDE 5/8" DIAMETER PUDDLE WELDS AT 12" O.C.
1. ALL COMPOSITE DECK SHALL BE 3", AS MANUFACTURED BY CONSOLIDATED SYSTEMS INC. OR APPROVED EQUAL, UNLESS NOTED OTHERWISE
2. All COMPOSITE DECK SHALL BE 20 GAUGE MINIMUM
3. METAL DECK IS DESIGNED FOR UNSHORED CONSTRUCTION OF TWO CONTINUOUS SPANS OR MORE. THE DECK SUPPLIER SHALL INCREASE THE GAGE THICKNESS IF NECESSARY FOR SINGLE SPAN CONDITIONS
4. CONSTRUCTION MATERIAL MAY NOT BE PLACED ON BARE METAL DECK.
5. FINAL TOTAL SLAB THICKNESS IS INDICATED ON PLANS. CONTRACTOR IS TO PROVIDE ADDITIONAL CONCRETE REQUIRED DUE TO THE DEFLECTION OF UNSHORED DECK.
6. REMOVE ALL FERRELS AND DEBRIS FROM DECK PRIOR TO SLAB PLACEMENT
7. CONDUIT SHALL NOT BE PLACED IN COMPOSITE SLABS. ANY EXCEPTIONS ARE TO BE AS INDICATED BY THE ENGINEER OF RECORD AND COORDINATION DRAWINGS SHOWING THE EXACT NUMBER, SIZE AND LOCATION OF CONDUIT ARE TO BE APPROVED BY THE ENGINEER OF RECORD PRIOR TO PLACEMENT.
8. SEE SPECIFICATIONS FOR ADDITIONAL REQUIREMENTS.
9. ATTACHMENT:
   8.1. TYPE 3" DECK:
      (A) COMPOSITE DECK UNITS SHALL BE ATTACHED TO EACH STRUCTURAL SUPPORT MEMBER WITH 5/8" DIAMETER PUDDLE WELDS ON A 24/4 PATTERN.
      (B) SIDE LAPS OF ADJACENT UNITS SHALL BE FASTENED BY SCREWING WITH "HILTI" SELF DRILLING SCREWS TYPE #10-16 x 7/8" HWH WITH #4 POINT SHEETMETAL SCREWS SPACED AT 12" O.C.
   8.2. TYPE 1.5" DECK
      (A) TYPE B ROOF DECK, 18GAGE MIN
      (B) FASTEN PER TYPICAL DETAILS PROVIDED AND AS REQUIRED BY THE ROOF TRUSS/LIGHT GAGE DESIGN.

STRUCTURAL ABBREVIATIONS

&	AND
@	AT
ACI	AMERICAN CONCRETE INSTITUTE
ADDL	ADDITIONAL
AISC	AMERICAN INSTITUTE OF STEEL CONSTRUCTION
ALT	ALTERNATE
ANCH	ANCHOR, ANCHORS
ARCH	ARCHITECTURAL
ASTM	AMERICAN SOCIETY OF TESTING AND MATERIALS
AWS	AMERICAN WELDING SOCIETY
BL	BUILDING LINE
BLDG	BUILDING
BM	BEAM
BOTT	BOTTOM
BRG	BEARING
CBF	CONCENTRICALLY BRACED FRAME
CC	CENTER-TO-CENTER
CIV	CIVIL
CJ	CONTROL AND/OR CONSTRUCTION JOINT
CL	CENTERLINE
CLR	CLEAR
CLSM	CONTROLLED LOW STRENGTH MATERIAL (100PSI UNCONFINED MAX)
CMU	CONCRETE MASONRY UNIT
COL	COLUMN
CONC	CONCRETE
CONN	CONNECTION
CONST	CONSTRUCTION
CONT	CONTINUE, CONTINUOUS
CONTR	CONTRACTOR
CTR	CENTER
DBA/D2L	DEFORMED BAR ANCHOR
DIA	DIAMETER
DP	DEEP
DWG	DRAWING
DWGS	DRAWINGS
EA	EACH
EBF	ECCENTRICALLY BRACED FRAME
EF	EACH FACE
EL	ELEVATION
ELEC	ELECTRICAL
ELEV	ELEVATOR
EMBED	EMBEDDED, EMBEDMENT
ENG	ENGINEER
EOR	ENGINEER OF RECORD
EOS	EDGE OF SLAB
EQ	EQUAL
EQUIP	EQUIPMENT
ES	EACH SIDE
ETC	ET CETERA
ETF	ELEVATION, TOP OF FOOTING
EW	EACH WAY
EXP	EXPANSION
EXT	EXTERIOR
FAB	FABRICATOR
FND	FOUNDATION
FIN	FINISH, FINISHED
FLR	FLOOR
FS	FAR SIDE
FTG	FOOTING
FV	FIELD VERIFY
GC	GENERAL CONTRACTOR
HK	HOOK
HORIZ	HORIZONTAL
ID	INSIDE DIAMETER
IF	INSIDE FACE
IN.2	SQUARE INCHES
IN.3	INCHES CUBED
IN.4	INCHES TO THE FOURTH POWER
INFO	INFORMATION
INT	INTERIOR
JST	JOIST
JT	JOINT
KIPS	X 1000 POUNDS
KSF	KIPS PER SQUARE FOOT
KSI	KIPS PER SQUARE INCH
LG	LONG
LONG	LONGITUDINAL
MATL	MATERIAL
MAX	MAXIMUM
MECH	MECHANICAL
MIN	MINIMUM
MSL	MEAN SEA LEVEL
MTL	METAL
NF	NEAR FACE
NO	NUMBER
NS	NEAR SIDE
OC	ON CENTER
OD	OUTSIDE DIAMETER
PCF	POUNDS PER CUBIC FOOT
PERP	PERPENDICULAR
PL	PLATE
PSF	POUNDS PER SQUARE FOOT
PSI	POUNDS PER SQUARE INCH
REF	REFERENCE, REFERENCED
REINF	REINFORCED, REINFORCING
REQ	REQUIRE, REQUIRED
REQ'D	REQUIRE, REQUIRED
SCHED	SCHEDULE
SHT	SHEET
SIM	SIMILAR
SL	SLOPE
SMF	SPECIAL MOMENT FRAME
SP	SPACE, SPACES
SQ	SQUARE
SS	STAINLESS STEEL
STAG	STAGGER
STD	STANDARD
STL	STEEL
T	TOP
T/	TOP OF
THK	THICK
TRANS	TRANSVERSE
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
VERT	VERTICAL
W/	WITH
WD	WOOD
WP	WORKING POINT
WWF	WELDED WIRE FABRIC

S&W
ARCHITECTURE ENGINEERING INTERIORS

STEVENS & WILKINSON
1501 MAIN STREET, SUITE 730
COLUMBIA, SC 29201
P 803.765.0320 F 803.254.6209
WWW.STEVENS-WILKINSON.COM

Goforth Brown & Associates
architects and planners
1413 West Evans Street
Florence, South Carolina

Justice Planning Associates, Inc.

SUBMITTALS (BY LETTER)/REVISIONS (BY NUMBER)

NO.	DATE	DESCRIPTION
A	1 .2 .1	O NER RE IE
	11. .1	DD PRICIN SET
C	11.1 .15	A RE ATE PIER PRICIN ON
D	12.1 .15	CM RE IE

☐ APPROVED FOR CONSTRUCTION
■ NOT APPROVED FOR CONSTRUCTION

S&W PROJECT NUMBER: 13022.00

DATE: 12.18.15

FLORENCE COUNTY

FLORENCE COUNTY
JUDICIAL CENTER

GENERAL NOTES

S0.01