REPORT OF PRELIMINARY GEOTECHNICAL EXPLORATION

Niagara Florence Industrial Park

Florence, South Carolina
S&ME Project No. 4263-15-021-01

Prepared By:

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Reference: Report of Preliminary Geotechnical Exploration
Niagara Florence Industrial Park
Florence, South Carolina
S&ME Project No. 4263-15-046-01

S&ME, Inc. has completed the preliminary geotechnical exploration for the referenced project after receiving authorization to proceed on April 8, 2015. Our exploration was conducted in general accordance with our Proposal No. 42-1400518rl, dated January 9, 2015.

The purpose of this exploration was to evaluate general subsurface conditions within the specified parcels for site certification purposes. This report characterizes the general surface and subsurface conditions of the site, offers preliminary recommendations regarding site preparation, suitability of on-site soils for use in construction and potential foundation types. The recommendations contained herein should be considered of a preliminary nature and are not valid for design without the confirmation of an additional design level subsurface exploration.

S&ME, Inc. appreciates this opportunity to work as your geotechnical engineering consultant. If you should have any questions concerning this report, please do not hesitate to contact us.

Very truly yours,
S&ME, Inc.

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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.

This is a preliminary exploration. The number of borings performed is insufficient to allow reliance upon the preliminary conclusions provided in this report for final design purposes. Additional exploration is required to confirm the preliminary conclusions once the site layout plan has been finalized and precise building locations are known.

1. **Soil Conditions:** Beneath the surface materials of topsoil and gravel, the soil profile generally consisted of an upper stratum of soft to very stiff fine grained soils (Stratum I) to depths of about 10 to 13 feet. Below that, a stratum of medium dense sands with hard clay lenses (Stratum II) was encountered to depths of 11 to 22 feet. Underlying Stratum II, soundings encountered an interbedded stratum of silts, clays, and sands to depths of 20 to 27 feet. Below Stratum III we encountered very stiff silts and clays with sand seams (Stratum IV) to the maximum exploration depth of about 30 feet.

2. **Subsurface Water:** At the time of drilling, the subsurface water level was interpreted from pore pressure readings to range from about 4 to 12 feet below the ground surface. Shallow perched water is likely to develop at this site, especially in low-lying portions of the site, due to the poor drainage capacity and low infiltration rate of the upper soil layers. Water levels may vary across the site, due to various influences including perched water, topography, and seasonal fluctuations.

3. **Site Preparation & Surface Stabilization:** Establish positive drainage at the site as soon as possible. After stripping of surface soils including construction debris, logging debris, and topsoil, the exposed surface soils within building pad and pavement subgrade areas should be thoroughly densified with a heavy sheepsfoot roller prior to new fill placement. Some overexcavation of soft upper clays or loose clayey sands should be anticipated to be necessary. Proofrolling of the subgrade by the contractor under the observation of the geotechnical engineer should be used to identify the unstable areas that require more rigorous stabilization efforts prior to new fill placement.

4. **Seismic Site Class:** Cone sounding data and shear wave velocity field test data indicates that this site is best described as IBC 2012 seismic Site Class D. Based on the apparent age and soil structure of the subsurface soils, widespread

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1 The use of a smooth-drum vibratory roller is not recommended in the upper clay soils. The vibrations may tend to draw shallow perched water up to the surface, and the smooth drum may be less effective than the sheepsfoot style compaction equipment that is designed for use in clayey soils.
liquefaction was determined to be unlikely at this site, considering the anticipated ground accelerations associated with the design magnitude earthquake.

5. **Seismic Design Parameters:** Based on the soil profile, the following Site Class D seismic design parameters are applicable: $F_A = 1.34$, $F_V = 2.00$, $S_{DS} = 0.51g$, $S_{D1} = 0.27g$, and Mapped MCE Geometric Mean Peak Ground Acceleration ($PGA_M$) = 0.37g. For structures in Seismic Risk Category I, II, or III, this indicates Seismic Design Category D.

6. **Foundation Types:** We anticipate the use of shallow foundations will likely be acceptable for lightly to moderately loaded structures, utilizing working load bearing pressures ranging from 2,000 psf to 3,000 psf, depending upon the structures’ loads and locations within the site. Bearing conditions were significantly variable between the test locations. Generally speaking, surface soils were softest in the northern portion of the site near U.S. Highway 76, and stiffer in the southern portion of the site near Paper Mill Road. Static settlement magnitudes are estimated to be within the typically acceptable range for shallow foundations under light to moderate loadings, but may exceed acceptable tolerances for heavily-loaded structures in large industrial applications unless ground improvement techniques such as undercutting and replacement or preconsolidation by surcharging with a temporary soil embankment is performed. Deep (pile or pier) foundations may be required for support of heavily loaded buildings or equipment.
1. **PURPOSE AND SCOPE**

The purpose of this report is to provide specific due diligence services necessary to satisfy portions of the South Carolina Department of Commerce Industrial Site Certification Program. This preliminary geotechnical exploration was performed to evaluate the general subsurface conditions at the site, and to generally discuss the findings with regard to future site development. It is important to realize that our preliminary geotechnical report is not of sufficient detail for use in final design and that additional geotechnical exploration and evaluation will be required for each building once the proposed site layout and design plans are finalized.

2. **PROJECT AND SITE DESCRIPTION**

Project information was provided to Mr. Charles Oates with S&ME during a meeting with Mr. John Richards with Thomas & Hutton on May 14, 2014. Project information included a drawing prepared by Thomas & Hutton dated May 13, 2014, titled “Conceptual Layout of Industrial Park”. According to the provided information, the subject property consists of approximately 529 acres that is located south of and adjacent to U.S. Highway 76, in Florence, Florence County, South Carolina. The subject property is comprised of developed land, and both undeveloped and cleared land surrounding the former DuPont Teijin Films facility. A Site Vicinity Map is included in Appendix A as Figure 1.

3. **EXPLORATION PROCEDURES**

During April, 2015, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

- We performed a site walkover, observing features of topography, existing structures, ground cover, and surface soils at the project site.

- We established eleven cone penetration test (CPT) sounding locations across the parcel, and established two MASW (Multichannel Analysis of Surface Waves) and MAM (Microtremor Array Measurement) test locations. The approximate sounding and seismic test locations are shown on the test location sketch included as Figure 2 in the Appendix A.

- At each test sounding location, a direct push sample was collected to observe the near surface soils.

A description of the field tests performed during the exploration as well as the CPT sounding logs and hand auger boring logs are attached in Appendix B. No laboratory testing was proposed to be performed as part of this exploration.
4. **SURFACE CONDITIONS**

The site currently consist of wooded areas, recently harvested timber areas, agricultural fields, and a non-operating industrial facility. The wooded areas generally consist of medium dense pines and mixed hardwoods ranging from 6 to 12 inches in diameter, while most recently harvested areas consist of logging debris and chest high grasses. Several agricultural fields were also noted which appear to be active. There are several cleared lanes through the site area. Organic topsoil was encountered at test locations C-1, C-2, C-3, C-4, C-8, C-9, C-10, C-11, ranging from about 4 to 8 inches in thickness, gravel was encountered at test location C-5 and was approximately 4 inches in thickness. Fill soil was encountered at the ground surface at test locations C-6, and C-7.

5. **SUBSURFACE CONDITIONS**

The generalized subsurface conditions at the site are described below. For more detailed descriptions and stratifications at specific test locations, the respective Test Records should be reviewed in Appendix B.

5.1 **Local Geology**

A review of local geologic mapping indicates that the site area lies within an outcrop area of the Bear Bluff Formation (Tb), typically interlayered terrestrial clays, silts, and sands laid down near the end of the Pliocene age approximately 3 million years ago. At this site, the soils appear to be part of the fluvial facies of the Bear Bluff, representing archaic river terrace materials deposited by fluvial action in the Pee Dee River valley. These materials form a mantle approximately 40 feet thick which overlie much older (60 million years), calcareous soils below. The surface has been reworked to some degree by erosional processes over geologic time, and the surface soils can be said to be fully developed in that they exhibit distinct pedological horizons.

The Bear Bluff Formation has been eroded in the Pee Dee floodplain and sediments of the Bear Bluff are exposed along the bluffs on either side of the valley. Materials underlying the Bear Bluff are mapped as Cretaceous age sediments of the Donoho Creek Formation (Kdc), approximately 60 million years old. The site is also located very near the landward margin of the slightly younger Pee Dee Formation (Kpd), which occurs just above the Donoho Creek and outcrops just to the south of the site along the west bank of the Pee Dee River. While not penetrated by the borings, a thin zone of Pee Dee Formation material may be present in the site area on top of the Donoho Creek.

The Donoho Creek Formation is part of the Black Creek Group. This formation consists of interlayered cross-bedded sands, clays and sandy clays deposited in a mostly terrestrial environment. Soils are generally well consolidated; calcareous deposits indicating marine deposition are rare. Samples recovered in borings are typically silts and clays in the upper contact zone, transitioning at greater depths to sands with considerable amounts of moderately to highly plastic fines. Ledges of very hard materials are often encountered in the Donoho Creek Formation, exhibiting SPT N-values of greater than 100 blows per foot. The Donoho Creek Formation generally forms the bearing layer for
deep foundations supporting heavy structures in the area, and is rarely penetrated fully by geotechnical borings.

Though not encountered in any of the borings, the Middendorf Formation forms the lowermost Coastal Plain strata in the site vicinity, extending to metamorphic bedrock at great depth.

5.2 Interpreted Subsurface Soil Profile

A subsurface cross-sectional profile of the soils on the site is attached in Appendix A as Figure 3. The cross-section orientation in plan view is shown on Figure 2. The profile is given to provide a representation of the conditions over widely spaced locations. Please note that the profile is not to scale and is provided for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions will vary between test locations. It was beyond the scope of our exploration to survey the ground surface elevation at our test locations; therefore, for the purpose of illustrating the cross-sectional soil profile the ground surface is shown as level, which does not reflect the actual surface topography.

Soils encountered at the test locations were grouped into four general strata based on estimated physical properties derived from the CPT sounding results. These strata are discussed in the following sections.

5.2.1 Stratum I: Upper Soft to Very Stiff Fine Grained Soils

Beneath the surface topsoil our soundings encountered cohesive fine grained soils that extend to depths of about 10 to 13 feet beneath the ground surface. The soil types observed within the direct push samples generally consist of clayey sands (SC), silty sands (SM), and sandy lean clays (CL) to the maximum direct push depth of 4 feet. Coloration of these soils was tan, orange, and gray, and moisture condition of these soils ranged from moist to wet.

These soils exhibited CPT tip stresses generally ranged from about 20 to 200 tons per square foot (tsf) but typically averaged 60 to 80 tsf, and sleeve stresses ranged from about 1.0 to 6.0 tsf. These values indicate a very stiff consistency. In sounding C-4 a very hard seam was noted between 5 and 7 feet in depth, this seam reached a maximum tip stress of 200 tsf, and a maximum sleeve stress of 10 tsf.

5.2.2 Stratum II: Medium Dense Sands with Hard Clay Lenses

Underlying Stratum I and beginning at depths of 8 to 16 feet, the CPT soundings encountered layers of stiff clays and medium dense to dense sands to depths of 11 to 22 feet and was not fully penetrated by soundings C-1, C-3, and C-5. Tip stress measurements ranged from about 100 to 350 tsf, typically ranging from about 100 to 150 tsf, indicating a generally hard consistency in the clays and a medium dense to dense condition in the sands. Sleeve stresses were measured to range from about 2 to 5 tsf.
5.2.3 Stratum III: Interbedded Silt, Clay, and Sand Mixtures

Below Stratum II and beginning at depths of 11 to 22 feet, the CPT soundings encountered a layer of interbedded sands, silts, and clays. The stratum extended to depths of approximately 20 to 27 feet. Tip stress measurements ranged from about 10 to 60 tsf, with sleeve stresses ranging from near 0.1 to 2 tsf, indicating a loose to medium dense relative density in the sandy soils, and a firm to very stiff consistency in the cohesive soils.

5.2.4 Stratum IV: Lower Very Stiff Silts and Clays

Below the soils of Stratum III, beginning at a depth of approximately 20 to 27 feet and extending to the sounding termination depth at C-2, C-4, and C-6 through C-11, the CPT soundings encountered a layer of very stiff silts and clays. Maximum reaction force to advancement of the drilling tools was encountered in all soundings which penetrated into this stratum, except C-10, at depths ranging from 16 to 28 feet. Tip stress measurements ranged from about 80 to 300 tsf, and sleeve stresses were generally 2 to 5 tsf, indicating a generally very stiff to very hard consistency. Tip and sleeve stresses generally increased with depth.

5.3 Subsurface Water

The subsurface water level was interpreted to range between 4 to 12 feet below the ground surface at the time of the exploration, based upon the pore pressure readings measured in the CPT soundings. This likely represents a perched water table. Water levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence water elevations.

Due to the clayey nature of near-surface soils in some portions of the site, there is potential for perched water conditions to develop or worsen following periods of wet weather. These conditions can often be managed by the installation of drainage ditches prior to site grading and maintenance of ditches during construction. Permanent pavement underdrains are advisable around the perimeter of roads and parking lots.

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

6. BUILDING CODE SEISMIC PROVISIONS

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

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

### 6.1.1 Liquefaction Potential of Bearing Soils

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.

Liquefaction of saturated, loose, cohesionless soils occurs when they are subjected to earthquake loading that causes the pore pressures to increase and the effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results. Earthquake-induced ground surface acceleration at the site was assumed from the building code design peak ground acceleration adjusted for site effects (PGA$_M$) of 0.37g for this site.

To evaluate liquefaction potential, we performed analyses using the data obtained in the borings, considering the characteristics of the soil and water levels observed in the boring. The liquefaction analysis was performed based on the design earthquake prescribed by the 2012 edition of the International Building Code, the “simplified procedure” as presented in Youd et al. (2001), and recent research concerning the liquefaction resistance of aged sands (Hayati & Andrus, 2008; Andrus et al. 2009; Hayati & Andrus, 2009).

To help evaluate the consequences of liquefaction, we 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 soil profile is expected to liquefy. The level of risk is generally defined below.

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

The LPI for this site is expected to be less than 1, which indicates that the risk of liquefaction induced damage is low. For this reason, and because the earthquake-related settlements associated with this LPI value are expected to be insignificant, our analysis
indicates that liquefaction during the design seismic event is unlikely to be a significant design concern at this site, and therefore Site Class F conditions do not apply.

### 6.1.2 Site Class Determination

Shear wave velocity data obtained within the limits of the proposed construction extends to a depth of about 170 feet. In accordance with Chapter 20 of ASCE 7-10, an average velocity of 955 feet per second was estimated between our two test array locations in the upper 100 feet of the soil profile ($V_{s100}$). See also Figures 4 and 5 in Appendix A. Based on this data and our knowledge of the geologic conditions in this area, the use of Seismic Site Class “D” parameters appears to be appropriate.

### 6.1.3 Design Spectral Acceleration Values

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 $F_V$ in IBC Section 1613.3.3, tables 1613.3.3(1) and 1613.3.3(2). 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 1: Spectral Design Values

<table>
<thead>
<tr>
<th></th>
<th>2012 IBC (2008 Seismic Hazard Maps)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{MS}$</td>
<td>0.77 g</td>
</tr>
<tr>
<td>$S_{M1}$</td>
<td>0.40 g</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>0.51 g</td>
</tr>
<tr>
<td>$S_{D1}$</td>
<td>0.27 g</td>
</tr>
<tr>
<td>$F_A$</td>
<td>1.34</td>
</tr>
<tr>
<td>$F_V$</td>
<td>2.00</td>
</tr>
<tr>
<td>PGA_m</td>
<td>0.37 g</td>
</tr>
</tbody>
</table>

Under the 2012 IBC, for a structure having a Seismic Risk Category classification of I, II, III, or IV spectral response acceleration factors given above correspond to Seismic Design Category D.

7. **PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS**

The preliminary conclusions and recommendations included this section are based upon the project information outlined in the introduction, and the data obtained during our exploration at the widely spaced sounding locations.

7.1 **Foundation Types**

The soil profile encountered appears generally suitable for development of typical lightly to moderately loaded industrial buildings using shallow foundations. The use of shallow foundations for support of isolated column loads up to 250 kips with a uniform area load of up to 250 psf, and an allowable shallow foundation bearing pressure of at least 2,000 pounds per square foot (psf) appears feasible throughout the site, with total static settlement magnitudes ranging from $\frac{1}{2}$ to $\frac{3}{4}$ inch for typical structural column configurations, provided that footings are properly constructed, the subgrade is properly prepared, and the fill is properly placed and compacted. In portions of the site, it may be feasible to use an allowable shallow foundation design bearing pressure of up to 3,000 psf where shallow bearing conditions are favorable, with total settlement magnitudes of 1 inch or less under the loading parameters described above.

Please note that these settlement estimates are for illustrative purposes only, and do not take into account site grade changes (cut and fill) or specific structural load distribution patterns for any particular structure. Static settlements will need to be estimated for each specific structure during future explorations.

Construction of commercial or industrial buildings with loadings greater than those listed above could result in settlement magnitudes of greater than one inch. Structures such as these may require deep foundations for support of structural loads, or preconsolidation techniques to improve the ground and dissipate some of the settlement potential and/or removal and replacement of the soft near-surface soils.
7.2 Site Preparation and Fill Considerations

Soils containing significant amounts of fine-grained material similar to those encountered in Stratum I and Stratum II are likely to become unstable, and be difficult to work when wet. As such, we recommend that grading be performed during the typically drier months of summer and fall, if possible. If grading is performed during periods of wet weather, grading costs may be greater. It may also be necessary to use rigorous stabilization methods, such as placement of a reinforcement geo-textile, in order to stabilize wet clayey soils. The potential need for near-surface stabilization may be greater in the lower-lying areas of the site where water levels are closer to the ground surface.

7.2.1 Stripping

Where encountered, organic-laden topsoil and rootmat materials should be stripped to a sufficient depth where exposed soils contain less than about 5 percent organics by weight. Clearing and stripping with relatively light, wide-tracked equipment would help prevent mixing of topsoil and rootmat with the underlying soils, which may otherwise be suitable for use. Excessive use of heavy rubber-tired equipment during stripping and grading, particularly during wet periods, may rut the surface and increase the depth of cut required to reach stable soils. The grading contractor should spoil organics and organic-laden soils outside the building and pavement areas.

7.2.2 Proofrolling

Once stripped, the subgrade soils should be evaluated by proofrolling under the observation of the Geotechnical Engineer prior to the placement of fill material. Proofrolling should be conducted by having the contractor make multiple passes over the soil surface with a fully-loaded tandem axle dump truck, off-road dump truck, or earth-moving pan. Areas of unstable material as indicated by the proofroll may require selective undercutting or further stabilization prior to fill placement, as determined by the Geotechnical Engineer.

7.2.3 Surface Densification

After removal of topsoil and rootmat material, the exposed surface soils within building pad and pavement subgrade areas should be thoroughly densified with a heavy sheepfoot\(^2\) roller prior to new fill placement. Some overexcavation of soft upper clays should be anticipated to be necessary.

7.2.4 On-Site Soil Suitability for Use as Fill

The clays and clayey sands below the topsoil may be used as fill if properly conditioned; however, these soils may require significant drying in order to achieve moisture contents

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\(^2\) The use of a smooth-drum vibratory roller is not recommended in the upper clay soils. The vibrations may tend to draw shallow perched water up to the surface, and the smooth drum may be less effective than the sheepfoot style compaction equipment that is designed for use in clayey soils.
suitable for compaction. On other local projects, this has required mechanical or chemical stabilization to accomplish, as these soils are unlikely to dry without assistance.

- **Mechanical stabilization:** spread the material out in thin layers and continuously turn it over with a disc harrow during dry weather until moisture content testing indicates that the material is 1 to 2 percent drier than the optimum moisture content.

- **Chemical stabilization:** spread the material out in thin layers and intermix it with 2 to 4 percent by weight of quicklime (calcium-oxide), blend, and compact.

In order to evaluate borrow material suitability at specific areas of interest; test pits should be performed under the direction of S&ME to obtain samples and perform laboratory testing. Where located below the water table, it should be anticipated that the moisture content of borrowed soils may be above the optimum moisture content, and that some drying may be required prior to compaction. If borrow excavation occurs during a wet season, soils above the optimum moisture content may be encountered nearer to the surface.

### 7.3 Recommendations for Additional Exploration Work

The sounding data provided gives only a general indication of the soil conditions at eleven widely-spaced locations on the site, and is intended only to give a general overview of the soil conditions across the site. Additional exploration will be necessary to allow us to provide specific design recommendations for any proposed structure. We recommend that additional work include a comprehensive geotechnical exploration based upon the proposed building layout.

The additional geotechnical exploration should include borings or soundings of sufficient number and depth as well as a comprehensive laboratory testing program, to allow us to finalize foundation recommendations once site development and design has advanced beyond a preliminary stage, and building locations and structural loads are known.

It may be possible to upgrade this site to seismic design category C by performing a site-specific seismic response (SSRA) analysis using the MASW/MAM data collected during this preliminary exploration. While there are no guarantees that this would be the result of an SSRA, experience shows that the general procedure of the Code may result in conservative spectral acceleration values that can sometimes be improved upon by performing the site specific procedures as described in ASCE 7-10. It was beyond the scope of this exploration to perform an SSRA.

### 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 preliminary subsurface exploration. The nature and extent of variations between the soundings will not become evident until construction. If variations appear evident, then we should be given a reasonable opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the development is planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

Assessment of site environmental conditions; sampling of soils, groundwater or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts was beyond the scope of the geotechnical portion of this project.
APPENDIX A

SITE VICINITY MAP

TEST LOCATION SKETCH

SUBSURFACE CROSS-SECTIONAL SOIL PROFILE A-A’

SHEAR WAVE VELOCITY PROFILE SW-1

SHEAR WAVE VELOCITY PROFILE SW-2
Scale: Not To Scale
Source: Google Earth
Date: January, 2015
Drawn By: WAG

SITE VICINITY MAP
Niagara Florence Industrial Park
Florence, South Carolina

Approximate Site Location
The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.
Figure 4
Shear Wave Velocity Profile SW-1
Niagara Florence Industrial Park
Florence, South Carolina
4263-15-046
Shear Wave Velocity, Vs (ft/sec)

\[ \bar{V}_{s100} = 978 \text{ ft/sec} \]
$\bar{v}_{s100} = 933 \text{ ft/sec}$
APPENDIX B

SUMMARY OF EXPLORATION PROCEDURES

CPT CLASSIFICATION LEGEND

CPT SOUNDING LOGS

SPT CLASSIFICATION LEGEND

DIRECT PUSH SAMPLE LOGS
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

Where practical, we reviewed available topographic maps, county soil surveys, reports of nearby investigations and aerial photographs when preparing the boring and sampling plan. Then we walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures.

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

BORING AND SAMPLING

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

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
SUMMARY OF EXPLORATION PROCEDURES

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, and as measured in the hand auger borings.

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

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

A combination of active and passive sources was used to develop the wave frequencies required to obtain velocities to a depth of 100 feet. The results of the active and passive sources were combined to produce a single shear wave velocity profile. Based on section 1615.1.5 and Equation 16-44 of 2009 International Building Code, the calculated weighted average shear wave velocities, \( v_s \), using the developed Shear Wave Velocity Profiles were determined.
CPT Soil Classification Legend

<table>
<thead>
<tr>
<th>Zone</th>
<th>Q/N</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>Sensitive, Fine Grained</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Organic Soils - Peats</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>Clays - Clay to Silty Clay</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>Silt Mixtures - Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>Sand Mixtures - Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>6</td>
<td>4.5</td>
<td>Sands - Clean Sand to Silty Sand</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Gravelly Sand to Sand</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>Very Stiff Clay to Clayey Sand*</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>Very Stiff, Fine Grained*</td>
</tr>
</tbody>
</table>

(*) Heavily Overconsolidated or Cemented

Robertson’s Soil Behavior Type (SBT), 1990

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

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

Relative Density and Consistency Table

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

Date: Apr. 14, 2015
Estimated Water Depth: 6.5 ft
Rig/Operator: Track/A. Feix

Total Depth: 25.2 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.75

Equivalent

C-11
Cone Penetration Test

Date: Apr. 15, 2015
Estimated Water Depth: 5.5 ft
Rig/Operator: Track/A. Feix

Total Depth: 28.1 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.75

S&ME Project No: 4263-15-046-01
Niagara Florence Industrial Park
Florence, SC

Depth (ft)
0 5 10 15 20 25

Tip Resistance
\( q_t \) (tsf)
0 100 200 300 400

Sleeve Friction
\( f_s \) (tsf)
0 5 10 15 20

Pore Pressure
\( u_z \) (tsf)
0 2 4 6 8

Friction Ratio
\( R_f \) (%)
2 4 6 8

Equivalent
\( N_{eq} \)
1 10 100

\( SBT_{fr} \)
MAI = 1

Depth (ft)
0 5 10 15 20 25

Soils:
- Very Stiff Fine Grained
- Very Stiff Clay to Clayey Sand
- Sand-Clean Sand to Silty Sand
- Silt Mixtures-Clay Silt to Silty Clay
- Estimated Water Depth: 5.5 ft
- Rig/Operator: Track/A. Feix
- Total Depth: 28.1 ft
- Termination Criteria: Maximum Reaction Force
- Cone Size: 1.75

CPT REPORT - DYNAMIC NIAGARA SITE CERTIFICATION.GPJ  S&ME 2008_06_24.GDT  5/21/15
Cone Penetration Test

Date: Apr. 15, 2015
Estimated Water Depth: 7 ft
Rig/Operator: Track/A. Feix

Total Depth: 16.3 ft
Termination Criteria: Target Depth
Cone Size: 1.75

Depth (ft)

Tip Resistance $q_t$ (tsf)

Sleeve Friction $f_s$ (tsf)

Pore Pressure $u_2$ (tsf)

Friction Ratio $R_f$ (%)

Equivalent $N_{eq}$

$SBT_fv$

MAI = 1

Depth (ft)

Very Stiff Fine Grained Soils

Clays-Clay to Silty Clay

Very Stiff Fine Grained Soils
Cone Penetration Test

Date: Apr. 14, 2015
Estimated Water Depth: 4 ft
Rig/Operator: Track/A. Feix

Total Depth: 27.8 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.75

Depth (ft) Tip Resistance $q_t$ (tsf) Sleeve Friction $f_s$ (tsf) Pore Pressure $u_s$ (tsf) Friction Ratio $R_f$ (%) Equivalent $N_e$

Very Stiff Fine Grained Soils
Very Stiff Fine Grained Soils
Very Stiff Clay to Clayey Sand
Sand Mixtures-Silty Sand to Sandy Silt

Electronic Filename: H14A1502C.ECP
Cone Penetration Test

Date: Apr. 14, 2015
Estimated Water Depth: 4.5 ft
Rig/Operator: Track/A. Feix

Total Depth: 27.0 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.75
### Soil Classification Chart

#### Major Divisions

<table>
<thead>
<tr>
<th>Clean Gravels</th>
<th>Clean Sands</th>
<th>Sands with Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRAVELS WITH FINES</td>
<td>(LITTLE OR NO FINES)</td>
<td>(APPRECIABLE AMOUNT OF FINES)</td>
</tr>
<tr>
<td>GRAVELS WITH FINES</td>
<td>(LITTLE OR NO FINES)</td>
<td>(APPRECIABLE AMOUNT OF FINES)</td>
</tr>
<tr>
<td>SANDS WITH FINES</td>
<td>(APPRECIABLE AMOUNT OF FINES)</td>
<td></td>
</tr>
</tbody>
</table>

#### Symbols

<table>
<thead>
<tr>
<th>Graph</th>
<th>Letter</th>
<th>Typical Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>GW</td>
<td></td>
<td>WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td>GP</td>
<td></td>
<td>POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES</td>
</tr>
<tr>
<td>GM</td>
<td></td>
<td>SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES</td>
</tr>
<tr>
<td>GC</td>
<td></td>
<td>CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td>SW</td>
<td></td>
<td>WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES</td>
</tr>
<tr>
<td>SP</td>
<td></td>
<td>POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES</td>
</tr>
<tr>
<td>SM</td>
<td></td>
<td>SILTY SANDS, SAND - SILT MIXTURES</td>
</tr>
<tr>
<td>SC</td>
<td></td>
<td>CLAYEY SANDS, SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td>ML</td>
<td></td>
<td>INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY</td>
</tr>
<tr>
<td>CL</td>
<td></td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS</td>
</tr>
<tr>
<td>OL</td>
<td></td>
<td>ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY</td>
</tr>
<tr>
<td>MH</td>
<td></td>
<td>INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS</td>
</tr>
<tr>
<td>CH</td>
<td></td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY</td>
</tr>
<tr>
<td>OH</td>
<td></td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS</td>
</tr>
<tr>
<td>PT</td>
<td></td>
<td>PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS</td>
</tr>
</tbody>
</table>

#### Notes

- Dual symbols are used to indicate borderline soil classifications.
- More than 50% of material is larger than No. 200 sieve size.
- More than 50% of coarse fraction retained on No. 4 sieve.
- More than 50% of coarse fraction passing on No. 4 sieve.
- Liquid limit less than 50.
- Liquid limit greater than 50.
- More than 50% of material is smaller than No. 200 sieve size.

---

### Soil Classification:

**SANDY SOILS**
- **SAND** and **SANDY SOILS**
- **CLAY** and **CLAYY SOILS**
- **SILT** and **SILTY CLAYS**
- **ORGANIC SILTS AND CLAYS**
- **INORGANIC SILTS AND CLAYS**
- **HIGHLY ORGANIC SOILS**

---

**Gravels**
- **CLEAN GRAVELS**
  - GRAVELS (LITTLE OR NO FINES)
  - GRAVELS (APPRECIABLE AMOUNT OF FINES)
- **GRAVELS WITH FINES**
  - GRAVELS WITH FINES (LITTLE OR NO FINES)
  - GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)

---

**Inorganic Silts and Very Fine Sands**
- **INORGANIC SILTS AND VERY FINE SANDS**
- **INORGANIC SILTS**
- **INORGANIC CLAYS**
- **INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY**
- **INORGANIC CLAYS OF HIGH PLASTICITY**

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**Organic Silts**
- **ORGANIC SILTS**
- **ORGANIC SILTY CLAYS**
- **ORGANIC SILTS AND ORGANIC SILTY CLAYS**

---

**Highly Organic Soils**
- **PEAT, HUMUS, SWAMP SOILS**
- **HIGHLY ORGANIC SOILS**

---

**Symbols**
- **GW**
- **GP**
- **GM**
- **GC**
- **SW**
- **SP**
- **SM**
- **SC**
- **ML**
- **CL**
- **OL**
- **MH**
- **CH**
- **OH**
- **PT**

---

**Notes**
- More than 50% of material is larger than No. 200 sieve size.
- More than 50% of coarse fraction retained on No. 4 sieve.
- More than 50% of coarse fraction passing on No. 4 sieve.
- Liquid limit less than 50.
- Liquid limit greater than 50.
- More than 50% of material is smaller than No. 200 sieve size.
# HAND AUGER BORING LOG: DP-1

**PROJECT:** Niagara Florence Industrial Park  
**Florence, SC**  
**4263-15-046-01**

<table>
<thead>
<tr>
<th>DATE STARTED:</th>
<th>DATE FINISHED:</th>
<th>NOTES:</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/14/15</td>
<td>4/14/15</td>
<td></td>
</tr>
</tbody>
</table>

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**WATER LEVEL:**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TOPSOIL - Approximately 6 inches in thickness.</td>
</tr>
<tr>
<td></td>
<td>SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, brown, moist.</td>
</tr>
<tr>
<td>2</td>
<td>CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, yellow to brown, moist.</td>
</tr>
<tr>
<td></td>
<td>No sample recovered.</td>
</tr>
</tbody>
</table>

---

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
**TOPSOIL** - Approximately 4 inches in thickness.

**CLAYEY SAND (SC)** - Mostly fine to medium sands, some low plasticity fines, brown, moist.

- - - - Orange to Gray at -1'.

**SANDY LEAN CLAY (CL)** - Mostly low plasticity fines, some fine sands, orange to red, moist.

---

**GRAPHIC LOG**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TOPSOIL - Approximately 4 inches in thickness.</td>
</tr>
<tr>
<td>2</td>
<td>CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, brown, moist.</td>
</tr>
<tr>
<td></td>
<td>- - - - Orange to Gray at -1'.</td>
</tr>
<tr>
<td>3</td>
<td>SANDY LEAN CLAY (CL) - Mostly low plasticity fines, some fine sands, orange to red, moist.</td>
</tr>
</tbody>
</table>

---

**HAND AUGER BORING LOG: DP-10**

**PROJECT:** Niagara Florence Industrial Park
Florence, SC 4263-15-046-01

**DATE STARTED:** 4/14/15  
**DATE FINISHED:** 4/14/15

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**WATER LEVEL:**

---

**NOTES:**

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
**MATERIAL DESCRIPTION**

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

- **TOPSOIL** - Approximately 3 inches in thickness.
- **SILTY SAND (SM)** - Mostly fine to medium sands, some low plasticity fines, tan to gray, wet.
- **CLAYEY SAND (SC)** - Mostly fine to medium sands, some low plasticity fines, orange to gray, wet to moist.
- **SANDY LEAN CLAY (CL)** - Mostly low plasticity fines, some fine sands, orange to gray, moist.

1. **PENETRATION RESISTANCE** is the number of blows of a 15 lb hammer falling 20 in., driving a 1.75 in. O.D. 45 DEGREE CONE 1.75 IN.
**TOPSOIL** - Approximately 6 inches in thickness.

**SILTY SAND (SM)** - Mostly fine to medium sands, some low plasticity fines, brown, moist.

**CLAYEY SAND (SC)** - Mostly fine to medium sands, some low plasticity fines, yellow to brown, moist.

**SANDY LEAN CLAY (CL)** - Mostly low plasticity fines, some fine sands, tan to gray, moist.

---

**GRAPHIC LOG**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TOPSOIL - Approximately 6 inches in thickness.</td>
</tr>
<tr>
<td>2</td>
<td>SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, brown, moist.</td>
</tr>
<tr>
<td>3</td>
<td>CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, yellow to brown, moist.</td>
</tr>
<tr>
<td>4</td>
<td>SANDY LEAN CLAY (CL) - Mostly low plasticity fines, some fine sands, tan to gray, moist.</td>
</tr>
</tbody>
</table>

---

**HAND AUGER BORING LOG: DP-2**

**PROJECT:** Niagara Florence Industrial Park
Florence, SC 4263-15-046-01

**DATE STARTED:** 4/14/15  
**DATE FINISHED:** 4/14/15

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**WATER LEVEL:**

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
**TOPSOIL** - Approximately 8 inches in thickness.

**SILTY SAND (SM)** - Mostly fine to medium sands, some low plasticity fines, brown, moist.

**SANDY LEAN CLAY (CL)** - Mostly low plasticity fines, some fine sands, tan to gray, moist.

---

**PROJECT:** Niagara Florence Industrial Park  
Florence, SC  
4263-15-046-01

**HAND AUGER BORING LOG: DP-3**

**DATE STARTED:** 4/14/15  
**DATE FINISHED:** 4/14/15

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>TOPSOIL - Approximately 8 inches in thickness.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, brown, moist.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>SANDY LEAN CLAY (CL) - Mostly low plasticity fines, some fine sands, tan to gray, moist.</td>
</tr>
</tbody>
</table>

---

1. **PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.**
TOPSOIL - Approximately 4 inches in thickness.

POORLY GRADED SAND (SP) - Mostly fine to medium sands, trace fines, tan, moist.

SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, orange, moist.

CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, orange, wet.

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
**HAND AUGER BORING LOG: DP-5**

**PROJECT:** Niagara Florence Industrial Park  
Florence, SC  
4263-15-046-01

<table>
<thead>
<tr>
<th>DATE STARTED:</th>
<th>DATE FINISHED:</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/14/15</td>
<td>4/14/15</td>
</tr>
</tbody>
</table>

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**WATER LEVEL:**

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

**MATERIAL DESCRIPTION**

- GRAVEL - Approximately 4 inches.
- FILL SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, yellow to brown, moist.
- CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, tan to orange, moist.
- No Recovery

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
### HAND AUGER BORING LOG: DP-6

<table>
<thead>
<tr>
<th>MATERIAL DESCRIPTION</th>
<th>ELEVATION (feet)</th>
<th>WATER LEVEL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FILL SILTY SAND (SM)</strong> - Mostly fine to medium sands, some low plasticity fines, trace gravel, brown, moist.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>CLAYEY SAND (SC)</strong> - Mostly fine to medium sands, some low plasticity fines, gray, moist.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PROJECT:** Niagara Florence Industrial Park  
**Florence, SC**  
**4263-15-046-01**

**DATE STARTED:** 4/14/15  
**DATE FINISHED:** 4/14/15

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**NOTES:**

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1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
### Hand Auger Boring Log: DP-7

**Project:** Niagara Florence Industrial Park  
**Florence, SC**  
**4263-15-046-01**

- **Date Started:** 4/14/15  
- **Date Finished:** 4/14/15

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Graphic Log</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Silty Sand (SM) Plowzone - Mostly fine to medium sands, some low plasticity fines, small roots, brown to gray, moist.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Clayey Sand (SC) - Mostly fine to medium sands, some low plasticity fines, tan to gray, moist.</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>Sandy Lean Clay (CL) - Mostly low plasticity fines, some fine sands, gray to orange, moist.</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>No Recovery</td>
</tr>
</tbody>
</table>

**Sampling Method:** Vertek  
**Performed By:** A. Feix

**Notes:**

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1. Penetration Resistance is the number of blows of a 15 lb hammer falling 20 in., driving a 1.75 in. O.D. 45 degree cone 1.75 in.
**HAND AUGER BORING LOG: DP-8**

**DATE STARTED:** 4/14/15  
**DATE FINISHED:** 4/14/15

**PROJECT:** Niagara Florence Industrial Park  
**Florence, SC**  
**4263-15-046-01**

**SAMPLING METHOD:** Vertek  
**PERFORMED BY:** A. Feix

**WATER LEVEL:**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>TOPSOIL - Approximately 6 inches in thickness.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SILTY SAND (SM) - Mostly fine to medium sands, some low plasticity fines, tan to gray, moist.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>CLAYEY SAND (SC) - Mostly fine to medium sands, some low plasticity fines, gray to orange, moist.</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>No Recovery</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

1. PENETRATION RESISTANCE IS THE NUMBER OF BLOWS OF A 15 LB HAMMER FALLING 20 IN., DRIVING A 1.75 IN. O.D. 45 DEGREE CONE 1.75 IN.
TOPSOIL - Approximately 3 inches in thickness.

POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sands, few low plasticity fines, tan, moist.

CLAYEY SAND (SC) - Mostly fine to medium sands, few low plasticity fines, orange, moist.

SANDY LEAN CLAY (CL) - Mostly low plasticity fines, some fine sands, orange to red, moist.