# ECS Southeast, LLP

## Geotechnical Engineering Report

## UNCW Parking Deck

Wilmington, New Hanover County, North Carolina

ECS Project Number # 22:27313R2

February 5, 2019

ECS SOUTHEAST, LLP
"Setting the Standard for Service"
Geotechnical • Construction Materials • Environmental • Facilities
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SC Registered Engineering Firm 3252

February 5, 2019

Mr. Richard Collier
McKim & Creed
243 North Front Street
Wilmington, North Carolina 28401

ECS Project No. 22:27313R2

Reference: Geotechnical Engineering Report
**UNCW Parking Deck**
Wilmington, New Hanover County, North Carolina

Dear Mr. Collier:

ECS Southeast, LLP (ECS) has completed the subsurface exploration, and geotechnical engineering analyses for the above-referenced project. Our services were performed in general accordance with our Proposal No. 22:22867, dated November 2, 2018. This report presents our understanding of the geotechnical aspects of the project along, the results of the field exploration conducted, and our design and construction.

It has been our pleasure to be of service to McKim & Creed during the design phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and we would like to provide our services during construction phase operations as well to verify the assumptions of subsurface conditions made for this report. Should you have any questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

**ECS Southeast, LLP**

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## EXECUTIVE SUMMARY

The following summarizes the main findings of the exploration, particularly those that may have a cost impact on the planned development. Further, our principal foundation recommendations are summarized. Information gleaned from the executive summary should not be utilized in lieu of reading the entire geotechnical report.

- The geotechnical exploration performed for the planned development included eleven (11) electronic cone penetration test (CPT) soundings drilled to termination and refusal depths ranging from 10 to 70 feet. Two (2) Kessler dynamic cone penetrometer (DCP) tests were performed in the proposed pavement areas.

- On January 31, 2019, three (3) Kessler dynamic cone penetrometer (DCP) tests were performed in the proposed pavement areas in the west lot.

- The soundings generally encountered coastal plain soils consisting of Very Loose to Very Dense, Silty, Clean, and Gravelly SAND (SM, SP, SW) with layers of very soft to very stiff, Sandy and Clayey SILT (ML), and Silty, Lean, and Fat CLAY (CL-ML, CL, CH). Refusal was encountered in sounding S-5 at approximately 70 feet beneath the existing ground surface.

- In summary, the proposed structure can be supported with a shallow foundation with ground improvement consisting of rigid inclusions having an allowable bearing pressure of 6,000 psf or alternatively a deep foundation system consisting of 10” pipe piles, or 16” auger cast in place piles. Specific embedment depths and allowable loads are provided in Section 5.1.1 of the report.

# 1.0 INTRODUCTION

## 1.1 GENERAL

The purpose of this study was to provide geotechnical information for the design of a proposed new multi-level parking deck and revisions to the existing parking lot located off of Riegel Road in Wilmington, New Hanover County, North Carolina.

This report contains the results of our subsurface explorations, site characterization, engineering analyses, and recommendations for the design and construction of the proposed structure, drives, and parking lot.

## 1.2 SCOPE OF SERVICES

To obtain the necessary geotechnical information required for design of the structure nine (9) (CPT) soundings were performed. In the proposed pavement areas, two (2) (CPT) soundings and two (2) Kessler DCP tests were performed during the initial field investigation on November 20, 2018. Three (3) additional Kessler DCP tests were performed on January 31, 2019. The soundings were advanced to termination and refusal depths of approximately 10 to 70 feet beneath the ground surface.

This report discusses our exploratory and testing procedures, presents our findings and evaluations and includes the following.

- A brief review and description of our field test procedures and the results of testing conducted;
- A review of surface topographical features and site conditions;
- A review of area and site geologic conditions;
- A review of subsurface soil stratigraphy with pertinent available physical properties;
- Preliminary foundation recommendations;
    - Allowable bearing pressure;
    - Settlement estimates (total and differential);
- Deep foundation recommendations;
- Site development recommendations;
- Suitability of soils for use as fill material;
- Discussion of groundwater impact;
- Compaction recommendations;
- Pavement design recommendations;
- Special conditions encountered;
- Seismic site classification and liquefaction potential;
- Site vicinity map;
- Exploration location plan; and
- CPT sounding logs.

## 1.3 AUTHORIZATION

Our services were provided in accordance with our Proposal No. 22.22867, dated November 2, 2018, and Proposal No. 22.23032, dated January 23, 2019, and includes the Terms and Conditions of Service outlined with our Proposal.

# 2.0 PROJECT INFORMATION

## 2.1 PROJECT LOCATION

The proposed site is located off of Riegel Road on UNCW's campus in Wilmington, New Hanover County, North Carolina. The site is bounded on the southwest by Riegel Road, on the south by an existing building, and on the northwest by existing parking lot, and on the north and east by wooded areas. Figure 2.1.1 below shows an image of where the site is located.


**Figure 2.1.1 Site Location**

## 2.2 CURRENT SITE CONDITIONS

The site currently consists of an existing paved parking lot "L" with moderately wooded areas around the parking lot. Based on our site visit and approximate elevations taken from Google Earth, the site appears to slightly slope up towards the northwest side of the site with typical elevations on site ranging from around 38 to 46 feet.

## 2.3 PROPOSED CONSTRUCTION

ECS understands that the project consists of construction of a new four-level or five-level parking deck with approximately 780 to 1,000 parking spaces. The project also includes revisions to the existing parking lot "L" consisting of 98 surface parking spaces.

### 2.3.1 Site Civil Features

- Grading for drives, parking area and the building pad
- Cuts and fills less than 5 feet

### 2.3.2 Structural Information/Loads

At the time of this report, additional project information including structural loads and grading information was not available. The following information explains our assumed structural loads for the purpose of the recommendations made in this report:

**Table 2.3.2.1 Design Values**

| SUBJECT | DESIGN INFORMATION / EXPECTATIONS |
|---|---|
| Usage | Parking Deck |
| Column Loads | Up to 1,400 kips for interior columns and up to 350 kips for exterior columns |
| Wall Loads | Up to 40 kips/ft. |
| Finish Floor Elevation | +/- 5 feet (assumed) |

# 3.0 FIELD EXPLORATION

## 3.1 FIELD EXPLORATION PROGRAM

The field exploration was planned with the objective of characterizing the project site in general geotechnical and geological terms and to evaluate subsequent field and laboratory data to assist in the determination of geotechnical recommendations.

### 3.1.1 Cone Penetrometer Soundings

The subsurface conditions were explored by drilling eleven (11) electronic cone penetration test (CPT) soundings within the proposed parking deck and surface lot. The soundings were advanced to termination and refusal depths of 10 to 70 feet.

Sounding locations were located in the field by an ECS representative using a hand held GPS unit and referencing existing site features. The approximate as-drilled sounding locations are shown on the Exploration Location Diagram in Appendix A.

The CPT soundings were conducted in general accordance with ASTM D 5778. The cone used in the soundings has a tip area of 10 $cm^2$ and a sleeve area of 150 $cm^2$. The CPT soundings recorded tip resistance and sleeve friction measurements to assist in determining pertinent index and engineering properties of the site soils. The ratio of the sleeve friction to tip resistance is then used to aid in assessing the soil types through which the tip is advanced. The results of the CPT soundings are presented in Appendix B.

Within sounding S-5, seismic tests were performed at approximately three foot intervals to termination depth to measure the shear wave velocity ($v_s$) of the subsurface materials to aid in assessing the dynamic response properties of the site subsurface materials. The seismic shear waves are generated by making impact with a 20-pound sledgehammer onto a steel beam. The impacts are initiated on the right and left sides of the CPT rig and the corresponding wave traces recorded on an oscilloscope are analyzed to determine the shear wave velocity of the tested material. The waves are measured with three geophones that are installed in the cone. The results of the CPT soundings are presented in Appendix B.

### 3.1.2 Kessler Dynamic Cone Penetrometer Tests

Two (2) Kessler Dynamic Cone Penetrometer (DCP) tests were performed in the location of the drive and parking lot areas during the initial field investigation on November 20, 2018. Three (3) additional Kessler DCP tests were performed on January 31, 2019 in the west lot. The Kessler DCP is used to estimate the strength characteristics of soils. The Kessler DCP was continuously driven approximately 2 feet below the existing ground surface. The Kessler DCP is driven into the soil by dropping a Dual-Mass 17.6 lb Hammer from a height of 22.6 inches. The depth of cone penetration is measured at selected penetration or hammer drop intervals and the soil shear strength is reported in terms of the Kessler DCP index. The Kessler DCP index is based on the average penetration depth resulting from one blow of the 17.6 lb hammer. The Kessler DCP index can be correlated to CBR and modulus of rigidity. The individual results of the Kessler DCP tests are presented in Appendix B.

### 3.2 REGIONAL/SITE GEOLOGY

The site is located in the Coastal Plain Physiographic Province of North Carolina. The Coastal Plain is composed of seven terraces, each representing a former level of the Atlantic Ocean. Soils in this area generally consist of sedimentary materials transported from other areas by the ocean or rivers. These deposits vary in thickness from a thin veneer along the western edge of the region to more than 10,000 feet near the coast. The sedimentary deposits of the Coastal Plain rest upon consolidated rocks similar to those underlying the Piedmont and Mountain Physiographic Provinces. In general, shallow unconfined groundwater movement within the overlying soils is largely controlled by topographic gradients. Recharge occurs primarily by infiltration along higher elevations and typically discharges into streams or other surface water bodies. The elevation of the shallow water table is transient and can vary greatly with seasonal fluctuations in precipitation.

Based on the U.S. Geological Survey[1,2] the site of the proposed construction is underlain by the Castle Hayne Formation (Tec). The formation generally consists of bluish gray to tan, loosely consolidate fossiliferous sand with silt and clay underlain by limestone. The coastal plain soils generally consist of silty, clean, and gravelly sands, silts, and silty, lean, and fat clays. An overview of the general site geology is illustrated in Figure 3.2.1 below.


**Figure 3.2.1**

Geologic map for Figure 3.2.1 obtained from The North Carolina Dept. of Environment, Health, and Natural Resources, Division of Land Resources, NC Geological Survey, in cooperation with the NC Center for Geographic Information and Analysis, 1998, Geology - North Carolina (1:250,000), coverage data file geol250 and Google Earth.

[1] The North Carolina Dept. of Environment, Health, and Natural Resources, Division of Land Resources, NC Geological Survey, in cooperation with the NC Center for Geographic Information and Analysis, 1998, Geology - North Carolina (1:250,000), coverage data file geol250. The data represents the digital equivalent of the official State Geology map (1:500,000 scale), but was digitized from (1:250,000 scale) base maps.

[2] Rhodes, Thomas S., and Conrad, Stephen G., 1985, Geologic Map of North Carolina: Department of Natural Resources and Community Development, Division of Land Resources, and the NC Geological Survey, 1:500,000-scale, compiled by Brown, Philip M., et al, and Parker, John M. III, and in association with the State Geologic Map Advisory Committee.

## 3.3 SUBSURFACE CHARACTERIZATION

The subsurface conditions encountered were generally consistent with published geological mapping. The following sections provide generalized characterizations of the soil encountered during our subsurface exploration. For subsurface information at a specific location, refer to the CPT Sounding Logs in Appendix B.

**Table 3.3.1 Subsurface Stratigraphy**

| Approximate Depth Range | Stratum | Description | Ranges of N*-Values(1) blows per foot (bpf) |
|---|---|---|---|
| 0 to 0.25 (Surface cover) | N/A | Soundings/Borings contained an observed thickness of minimal amounts to 3 inches of topsoil. Deeper topsoil or organic laden soils are most likely present in wet, poorly drained areas and potentially unexplored areas of the site. In soundings, S-5 and S-8, approximately 4 inches of asphalt underlain by 5 inches of aggregate base was encountered. | N/A |
| (0-0.25) to 8 | I | Very Loose to Dense, Silty, Clean, and Gravelly SAND (SM, SP, SW) with occasional interbedded layers of very soft to firm, Sandy SILT (ML). Possible Fill SAND (SP) was encountered in some of the soundings in the upper 2 to 4 feet. | 2 to 34 |
| 8 to 15 | II | Very Loose to Dense, Silty, and Clean SAND (SM, SP) with interbedded layers of very soft to very stiff, Sandy and Clayey SILT (ML) and Silty and Lean CLAY (CL-ML, CL) | 2 to 44 |
| 15 to 26 | III | Loose to Very Dense, Silty, Clean, and Cemented SAND (SM, SP) with interbedded layers of soft to very stiff, Sandy and Clayey SILT (ML) and Silty and Lean CLAY (CL-ML, CL) | 3 to 52 |
| 26 to 30 | IV | Medium Dense to Very Dense, Silty, Clean, and Cemented SAND (SM, SP) | 16 to 52 |
| 30 to 60 | | Medium Dense, Silty and Clean SAND (SM, SP) with occasional interbedded layers of stiff to very stiff, Sandy SILT (ML) | 13 to 27 |
| 60 to 70 | IV | Soft to Stiff, Sandy and Clayey SILT (ML) and Silty, Lean, and Fat CLAY (CL-ML, CL, CH) with interbedded layers of loose to medium dense, Silty and Clean SAND (SM, SP). Sounding, S-5 encountered refusal at 70 feet. | 4 to 18 |

Notes: (1) Cone Penetration Test

## 3.4 GROUNDWATER OBSERVATIONS

Porewater pressure measurements were made at the sounding and boring locations during exploration as noted on the CPT sounding logs and the hand auger boring logs in Appendix B. The apparent groundwater depths were observed at the time of exploration to have approximately ranged from 4.2 to 14.0 feet below ground surface.

The highest groundwater observations are normally encountered in the late winter and early spring. Variations in the long-term water table may occur as a result of changes in precipitation, evaporation, surface water runoff, construction activities, and other factors not immediately apparent at the time of this exploration. If long term water levels are crucial to the development of this site, it would be prudent to verify water levels with the use of perforated pipes or piezometers.

## 3.5 SEASONAL HIGH WATER TABLE AND INFILTRATION TESTING

The results of the seasonal high water table and infiltration testing are shown in the report included in Appendix D of this report.

## 4.0 LABORATORY TESTING

The laboratory testing performed by ECS for this project consisted of selected tests performed on samples obtained during our field exploration operations. The following paragraphs briefly discuss the results of the completed laboratory testing program. Classification and index property tests were performed on representative soil samples obtained from the test borings in order to aid in classifying soils according to the Unified Soil Classification System and to quantify and correlate engineering properties.

An experienced geotechnical engineer/engineering geologist visually classified each soil sample from the test borings on the basis of texture and plasticity in accordance with the Unified Soil Classification System (USCS) and ASTM D-2488 (Description and Identification of Soils-Visual/Manual Procedures). After classification, the geotechnical engineer/engineering geologist grouped the various soil types into the major zones noted on the boring logs in Appendix B. The group symbols for each soil type are indicated in parentheses following the soil descriptions on the boring logs. The stratification lines designating the interfaces between earth materials on the boring logs are approximate; in situ, the transitions may be gradual.

# 5.0 DESIGN RECOMMENDATIONS

## 5.1 BUILDING DESIGN

The following sections provide recommendations for foundation design, soil supported slabs, and pavement design.

### 5.1 Foundation Options

#### 5.1.1 Intermediate Foundations (Ground Improvement for Shallow Foundations)

A ground improvement system consist of rigid inclusions can be used to transfer the stress of a shallow foundation system to stiffer soils at the site. Rigid inclusion systems general consist of cement treated aggregate, grouted aggregate, or concrete column elements and a load transfer platform below the foundation. The elements installed with a displacement tool mounted on a tracked carrier system. As the tool is advanced to the designed bear stratum elevation, granular soil layers in the upper stratums are densified by the displacement tool. Once the design bearing stratum elevation is encountered by the displacement tool, grout is pumped through the tool out a port at the tip of the tool. The tool is extracted at a designed rate and grout pressure to maintain a positive grout head during extraction. The element is terminated at a design elevation below the foundation elevation. Typically, gravel or stone is placed in the remaining space between the top of the element and the site subgrade to mark the element location to prevent grading contractors or utility contractors from excavating and damaging the elements after installation.

Depending on the soil conditions at the foundation subgrade elevation, a load transfer platform consisting of the existing granular soil or granular fill consisting of sand or gravel will need to be installed after foundation excavation is performed. This may result in additional excavation of the foundations, and the foundation contractor should be aware of this and account for it in the bidding process.

Based on the provided loads for the parking deck, ECS performed a preliminary rigid inclusion analysis for the design of shallow foundation system. The design of the foundation shall utilize the following parameters:

**Table 5.1.1.1 Foundation Design**

| Design Parameter | Column Footing | Wall Footing |
|---|---|---|
| Net Allowable Bearing Pressure[1] | 6,000 psf | 6,000 psf |
| Acceptable Bearing Soil Material | Medium Dense SAND (SP) - Stratum I | Medium Dense SAND (SP) - Stratum I |
| Minimum Width | 24 inches | 16 inches |
| Estimated Total Settlement | 1.5 inches | 1.5 inches |
| Estimated Differential Settlement | Less than 1 inch between columns | Less than 0.5 inches over 50 feet |

1. Net allowable bearing pressure is the applied pressure in excess of the surrounding overburden soils above the base of the foundation. The frost depth for this region is 6 inches.

2 For short-term loading conditions, the allowable bearing pressure can be increase by 1/3.

3. The coefficient of friction for the foundation is 0.40

Detailed design of the rigid inclusion ground improvement program is performed by a design/build specialty geotechnical contractor because of the proprietary nature of the various methods used to construct them. The various methods result in different diameters, depths, and capacity for the elements.

Specifications for rigid inclusion foundation systems should be prepared by the design-build foundation contractor. The specifications should include a provision for at least one load test of a completed element to confirm that the installation can produce elements with a capacity equal to or greater than that used for design.

Rigid inclusion elements should be installed after the site is filled to final grade. Care should be taken to minimize vibrations when installing rigid inclusions near existing structures.

We recommend that ECS be retained to monitor the rigid inclusions installer's operations as a Quality Assurance service. ECS' services will supplement the installer's internal Quality Control program. Together, the QA and QC programs will monitor installation depths, element lengths, installation procedures, concrete/grout properties and cylinder breaks. These items will be documented for each element installed, to provide a complete installation report.

### 5.1.2 Deep Foundations

**Driven Steel Pipe Piles:** Steel Pipe Piles can be used to support the proposed parking deck structure. The allowable pile capacities (based on a factor of safety of 2.0 for axial capacity, 3.0 for uplift capacity and ½ inch lateral deflection for lateral capacity) are presented in the following table:

**Table 5.1.2.1 10-Inch Driven Steel Pipe Pile with Square Plate at bottom**

| Embedment Depth (Feet) | Axial Capacity (Tons) | Uplift (Tons) | Lateral (Tons) |
|---|---|---|---|
| 70 feet | 90 | 11 | 6 |

If shallow refusal is encountered due to dense sand layers, the allowable axial and uplift capacities listed above should be reduced. ECS should be contacted to provide reduced capacities based on the encountered conditions and the PDA analysis performed during the test pile phase.

The loading assumes a free head pile condition with axial and shear forces applied to the pile head. The design capacity of the soils includes the potential negative friction forces for the clay layers in the soil profile and settlement associated with liquefaction.

It is highly recommended that several over length piles be driven prior to the start of production pile driving, to establish the driving criteria, pile lengths to be ordered and to determine if auger "pilot" holes are justified. Depending on the final design load of the pile system selected, a pile load test should be performed for piles with axial load capacities greater than 40 tons in order to verify the pile capacity. The over length piles could be driven in production pile locations. Production piles should not be ordered until the pile lengths can be determined. A minimum of four over length piles, are recommended for this site due to variable soil conditions encountered at the site.

Pile installation operations and PDA testing should be monitored by a senior soil technician working under the supervision of a Licensed Engineer. ECS has the PDA equipment and would be pleased to provide PDA testing once the method of installation and the contractor has been selected. The results of the PDA test are typically submitting three days after the completion of test pile operations.

ECS recommends that the successful bidder submit proposed equipment information including the proposed hammer types and details regarding helmet, cushion, etc. ECS can perform wave equation pile analyses (WEAP) to check the ability of the hammer to drive the pile to the required ultimate resistance, if requested. The WEAP analysis is also a valuable means to check the efficiency of the hammer as well as a means of establishing the driving criteria for production pile installation. The WEAP analysis is also used to check the driving stresses that develop within the piles during driving operations.

**Auger Cast-In-Place Piles:** Auger Cast-In-Place Piles can support the proposed parking deck structure. The allowable pile capacities (based on a factor of safety of 2.0 for axial capacity, 3.0 for uplift capacity and ½ inch lateral deflection for later capacity) are presented in the following table:

**Table 5.1.2.2 16-Inch Auger Cast In Place (ACIP) Pile**

| Embedment Depth (Feet) | Axial Capacity (Tons) | Uplift (Tons) | Lateral (Tons) |
|---|---|---|---|
| 40 feet | 36 | 9 | 20 |
| 70 feet | 110 | 20 | 20 |

If shallow refusal is encountered due to dense sand layers, the allowable axial and uplift capacities listed above should be reduced. ECS should be contacted to provide reduced capacities based on the encountered conditions and the pile load test results.

The loading assumes a fixed head pile condition with axial and shear forces applied to the pile head. The design capacity of the soils includes the potential negative friction forces for the clay layers in the soil profile and settlement associated with liquefaction.

The auger withdrawal should be performed in accordance with the North Carolina Building Code (International Building Code 2015 with State Amendments) Section 1810.4.8 to maintain the appropriate grout head for the project. A thorough monitoring of the auger cast pile installation procedures, the grout head, and the grout factor should be implemented. The grout factor is the actual grout volume of the pile divided by the theoretical pile volume. Due to the presence of soft clay and silt layers in some of the soundings, the grout factor could exceed 2 to 3 due to grout expansion into these layers.

We recommend that at least four auger cast test piles be installed in the parking structure to confirm the pile length and to confirm the contractor's installation procedures and techniques.

Depending on the final design load of the pile system selected, a pile load test should be performed for piles with axial load capacities greater than 40 tons in order to verify the pile capacity. Axial compression load tests should be performed in accordance with ASTM D1143. Tension pile load tests should be performed for each structure in accordance with ASTM 3689. The criteria for the pile load test acceptance is the Davisson Offset Limit. The structural engineer of record should determine the number of pile load tests to be performed.

During installation, it is recommended that an approximate 60 inch distance be maintained between adjacent piles. A minimum period of 12 hours is required for installing adjacent piles at less than this minimum distance.

The minimum grout strength for the auger cast in place piles should conform to the NC Building Code Section 1810.3.2.6.

**5.1.3 Floor Slabs**

Assuming the lowest finished floor elevation is around current elevations, it appears that the slabs for the structures will bear on the Stratum I soils – SAND or Approved Structural Fill. Provided the subgrade recommendations of this report are followed, this material is likely suitable for the support of a slab-on-grade. The following graphic depicts our soil-supported slab recommendations:

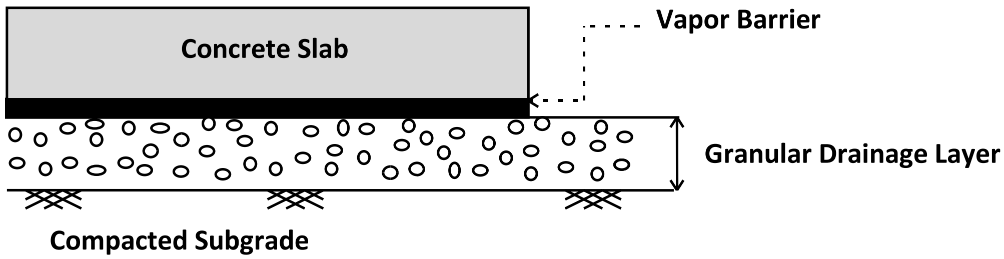



**Figure 5.1.2.1**

1. Grain Drainage Layer Thickness: 4 inches
2. Grain Drainage Layer Material: GRAVEL (GP, GW, GP-SM, GW-SM), SAND (SP, SW, SP-SM, SW-SM). Material should have less than 20 percent fines, and can consist of No. 57 stone, No. 67 stone, ABC, or screenings (ACI 302.1R-15). Gravel or stone should be wrapped with non-woven geotextile (Mirafi 140N or equivalent).
3. Subgrade compacted to 98% maximum dry density per ASTM D698.

**Subgrade Modulus:** Provided the placement of structural fill and granular drainage layer per the recommendations discussed herein, the slab may be designed assuming a modulus of subgrade reaction, k of 150 pci (lbs/cu. inch). The modulus of subgrade reaction value is based on a 1 ft by 1 ft plate load test basis.

**Slab Isolation:** Ground-supported slabs should be isolated from the foundations and foundation-supported elements of the structures so that differential movement between the foundations and slab will not induce excessive shear and bending stresses in the floor slab. Where the structural configuration prevents the use of a free-floating slab, the slab should be designed with suitable reinforcement and load transfer devices to preclude overstressing of the slab. Maximum differential settlement of soils supporting interior slabs is anticipated to be less than 0.5 inches in 50 feet.

**5.1.4 Site Retaining Walls**

Site retaining walls are often constructed from the “bottom-up” and therefore the type of soil used to backfill the wall is chosen or specified by contract. The lateral earth pressures developed behind site retaining walls is a function of the backfill soil type within an approximate 45-degree angle from the base of the wall upward.

**Lateral Earth Pressures:** Retaining walls should be designed to withstand the lateral earth pressures exerted by the backfill. The pressure diagram is triangular. It is anticipated that retaining walls associated with the building structure, such as for the unloading/loading dock situation, will be rigid walls restrained from rotation by the floor slab. For rigid walls, the "At Rest" ($k_o$) soil condition should be used in the wall design and evaluation. For walls that are free to deflect at their tops, the "Active" ($k_a$) soil condition should be used in the wall design and evaluation. In the design of these retaining wall structures, the following soil parameters can be utilized. These parameters assume that Granular Soils meeting the requirements recommended herein for Retaining Wall Backfill will comprise the backfill in the Critical Zone. The Critical Zone is defined as the area between the back of the retaining wall structure and an imaginary line projected upward and rearward from the bottom back edge of the wall footing at a 45-degree angle.

**Table 5.1.4.1 Retaining Wall Backfill in the Critical Zone**

| Soil Parameter | Estimated value |
|---|---|
| Coefficient of Earth Pressure at Rest ($K_o$) | 0.47 |
| Coefficient of Active Earth Pressure ($K_a$) | 0.31 |
| Retained Soil Moist Unit Weight (γ) | 125 pcf |
| Cohesion (C) | 0 psf |
| Angle of Internal Friction (ϕ) | 32° |
| Friction Coefficient [Concrete on Soil] (μ) | 0.40 |
| At-rest Equivalent Fluid Pressure | 59H (psf) |
| Active Equivalent Fluid Pressure | 39H (psf) |

**Table 5.1.4.2 Foundation Soils (Natural Subgrades or On-Site Borrow)**

| Soil Parameter | Estimated value |
|---|---|
| Allowable Net Soil Bearing Pressure | 3,000 psf(natural subgrade)<br>6,000 psf (ground improvement) |
| Minimum Wall Embedment Below Grade | 18 inches |
| Coefficient of Passive Earth Pressure ($K_p$) | 3.25 |
| Soil Moist Unit Weight (γ) | 120 pcf |
| Cohesion (C) | 0 |
| Interface Friction Angle [Concrete on Soil] ($\phi_f$) | 20° |
| Sliding Friction Coefficient [Concrete on Soil] (μ) | 0.40 |
| Passive equivalent fluid pressure | 390H (psf) |

**Retaining Wall Backfill:** All soils used as backfill within the Critical Zone behind retaining walls should have USCS classifications of Silty SAND (SM) or more granular with a maximum of 20% fines (i.e., % passing No. 200 Sieve size) and minimum angle of internal friction of 32 degrees when compacted to a minimum of 98% of its maximum dry density per ASTM D 698. Any existing soils not meeting these criteria should be removed from the Critical Zone of the walls, as determined by ECS personnel at the time of construction.

**Foundation Drains:** Retaining walls should be provided with a foundation drainage system to relieve hydrostatic pressures which may develop in the wall backfill. This system should consist of weepholes through the wall and/or a 4-inch perforated, closed joint drain line located along the backside of the walls above the top of the footing. The drain line should be surrounded by a minimum of 6 inches of AASHTO Size No. 57 Stone wrapped with an approved non-woven filter fabric, such as Mirafi 140-N or equivalent.

**Wall Drains:** All site retaining walls should be drained so that hydrostatic pressures do not build up behind the walls. Wall drains can consist of a 12-inch wide zone of free draining Gravel, such as AASHTO No. 57 Stone, employed directly behind the wall and separated from the soils beyond with a non-woven filter fabric. Alternatively, the wall drain can consist of a suitable geocomposite drainage board material. The wall drain should be hydraulically connected to the foundation drain.

### 5.1.5 Underslab Subdrainage

ECS doesn't anticipate the need for foundation drainage or underslab subdrainage unless site elevations are lowered to point that the FFE of the slab is within two feet of the groundwater elevation (Four feet below the current elevation of soundings S-3 and S-6). If required, the system may consist of a perimeter foundation drain located one foot below footing subgrade and one foot outside the footing perimeter. The foundation drains should have a minimum diameter of 8 inches, and they should be slotted or appropriately perforated. For the filter fabric we recommend a non-woven product such as Mirafi 140N with an AOS of 70 (U.S. Sieve). An equivalent geotextile fabric can also be used if approved by the Geotechnical Engineer of Record.

A network of a few interior drain lines is needed in addition to the interior perimeter lines due to the size of the footprint. Lateral drain lines under the floor slab should be placed at no more than 60 feet on center or as designed by the GER. Underslab drain lines should have a minimum diameter of 4 inches, and they should be slotted or appropriately perforated. For the filter fabric we recommend a non-woven product such as Mirafi 140N with an AOS of 70 (U.S. Sieve). An equivalent geotextile fabric can also be used if approved by the Geotechnical Engineer of Record. Clean out access should be installed at all sharp bends and at approximately every 100 feet for straight runs.

The exterior and interior drains should be designed to flow a collection point that terminates in a storm water box and daylights out into a storm water collection feature such as a ditch or pond.

### 5.1.6 Seismic Design Considerations

**Seismic Site Classification:** The International Building Code (IBC) 2015 requires site classification for seismic design based on the upper 100 feet of a soil profile. Three methods are utilized in classifying sites, namely the shear wave velocity ($v_s$) method; the unconfined compressive strength ($s_u$) method; and the Standard Penetration Resistance (N-value) method. The first method (shear wave velocity) was used in classifying this site.

The results of the shear wave velocity profiles are contained in Appendix B. The seismic site class definitions for the weighted average of shear wave velocity or SPT N-value in the upper 100 feet of the soil profile are shown in the following table:

**Table 5.1.6.1: Seismic Site Classification**

| Site Class | Soil Profile Name | Shear Wave Velocity, Vs, (ft./s) | N value (bpf) |
|---|---|---|---|
| A | Hard Rock | Vs > 5,000 fps | N/A |
| B | Rock | 2,500 < Vs ≤ 5,000 fps | N/A |
| C | Very dense soil and soft rock | 1,200 < Vs ≤ 2,500 fps | >50 |
| D | Stiff Soil Profile | 600 ≤ Vs ≤ 1,200 fps | 15 to 60 |
| E | Soft Soil Profile | Vs < 600 fps | <15 |

The seismic Site Class for the site was determined by calculating a weighted average of the shear velocities of the overburden to the depth of rock/refusal. The CPT test data indicates that the existing natural, overburden soils at the site have shear velocities ranging from approximately 210 ft/sec to 1549 ft/sec. The method for determining the weighted average value is presented in Section 1613.5.5 of the IBC 2009. The weighted average value for the site is 701 ft/sec. Based on the results of the CPT soundings and our evaluation of the site, the site shall be assigned a seismic class "D".

**Liquefaction:** The potential for liquefaction at the site is considered low based upon the CPT results and the liquefaction index procedure developed by Iwasaki (1982). Based on our CPT results and our evaluation using a site peak ground acceleration of 0.17, an earthquake event with a magnitude of 7.3 and procedures developed by Robertson (2009) and Boulanger & Idriss (2014), the liquefaction induced settlement at the subject site is estimated to be approximately 2 inches or less.

**Ground Motion Parameters:** In addition to the seismic site classification noted above, ECS has determined the design spectral response acceleration parameters following the IBC 2009 methodology The Mapped Reponses were estimated from the free Java Ground Motion Parameter Calculator available from the USGS website. The design responses for the short (0.2 sec, $S_{DS}$) and 1-second period ($S_{D1}$) are noted in bold at the far right end of the following table.

**Table 5.1.6.2: Ground Motion Parameters (IBC 2015 Method)**

| Period (sec) | Mapped Spectral Response Accelerations (g) | | Values of Site Coefficient for Site Class | | Maximum Spectral Response Acceleration Adjusted for Site Class (g) | | Design Spectral Response Acceleration (g) | |
|---|---|---|---|---|---|---|---|---|
| Reference | Figures 1613.3.1 (1) & (2) | | Tables 1613.3.3 (1) & (2) | | Eqs. 16-37 & 16-38 | | Eqs. 16-39 & 16-40 | |
| 0.2 | $S_S$ | 0.216 | $F_a$ | 1.600 | $S_{MS}=F_aS_s$ | 0.345 | $S_{DS}=2/3\ S_{MS}$ | 0.230 |
| 1.0 | $S_1$ | 0.091 | $F_v$ | 2.400 | $S_{M1}=F_vS_1$ | 0.218 | $S_{D1}=2/3\ S_{M1}$ | 0.145 |

The Site Class definition should not be confused with the Seismic Design Category designation, which the Structural Engineer typically assesses.

## 5.2 SITE DESIGN CONSIDERATIONS

### 5.2.1 Pavement Sections

**Subgrade Characteristics:** Based on the results of our hand auger borings, it appears that the soils that will be exposed as pavement subgrades consisting of SAND (SM, SP) or Approved Structural Fill. Based on the soil type and the Kessler DCP results, a CBR value of 10 has been selected to model the in place subgrade soils. The pavement design assumes subgrades consist of suitable materials evaluated by ECS and placed and compacted to at least 98 percent of the maximum dry density as determined by the standard Proctor test (ASTM D 698) in accordance with the project specifications.

**Design Considerations:** For the design and construction of exterior pavements, the subgrades should be prepared in strict accordance with the recommendations in the “Subgrade Preparation” and “Engineered Fill Placement” sections of this report. An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course layer, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should minimize the possibility of the subgrade materials becoming saturated during the normal service period of the pavement. The soundings performed in the existing parking lot encountered approximately 4 inches of asphalt overlying approximately 5 inches of aggregate base.

Anticipated traffic conditions were not provided to ECS. However, based on our experience for light duty traffic for similar projects, a flexible pavement section may consist of at least 2 inches of surface mix asphalt overlying at least 6 inches of grade aggregate base in the parking and roadway areas for personal vehicles. Similarly, a heavy duty, flexible pavement section may consist of at least 2 inches of surface mix asphalt overlying at least 8 inches of graded aggregate base in the roadway areas for delivery trucks and garbage trucks. Alternatively, a section consisting of 1.5 inches of surface mix and 2.5 inches of intermediate mix overlying at least 8 inches of graded aggregate base in the roadway areas can be used where the intermediate is put down for construction traffic and topped with the 1.5 inches of surface mix at the completion of the project. For a rigid pavement section, we recommend 6 inches of 4,000 psi compressive strength concrete overlying at least 4 inches of compacted crushed stone in the roadway areas.

Aggregate base course materials beneath pavements should be compacted to at least 98 percent of their modified Proctor maximum dry density (ASTM D 1557).

Regardless of the section and type of construction utilized, saturation of the subgrade materials and asphalt pavement areas results in a softening of the subgrade material and shortened life span for the pavement. Therefore, we recommend that both the surface and subsurface materials for the pavement be properly graded to enhance surface and subgrade drainage. By quickly removing surface and subsurface water, softening of the subgrade can be reduced and the performance of the parking area can be improved. Site preparation for the parking areas should be similar to that for the building areas including stripping, proofrolling, and the placement of compacted structural fill.

Please note that large, front-loading trash dumpsters frequently impose concentrated front-wheel loads on pavements during loading. This type of loading typically results in rutting of bituminous pavements and ultimately pavement failures and costly repairs. Concrete pavements should be properly jointed and reinforced as needed to help reduce the potential for cracking and to permit proper load transfer.

**Weather Restrictions:** In this region, asphalt plants may close during the months of December, January, and/or February if particularly cold weather conditions prevail. However, this can change based on year to year temperature fluctuations. Daily temperatures from December to February will often stay below 40°F, limiting the days that asphalt placement can occur.

# 6.0 SITE CONSTRUCTION RECOMMENDATIONS

## 6.1 SUBGRADE PREPARATION

### 6.1.1 Stripping and Grubbing

The subgrade preparation should consist of stripping all vegetation, rootmat, topsoil, existing fill, existing footing and pavement, and any other soft or unsuitable materials from the 10-foot expanded building foot prints and 5-foot expanded pavement limits and to 5 feet beyond the toe of structural fills. ECS should be called on to verify that topsoil and unsuitable surficial materials have been completely removed prior to the placement of Structural Fill or construction of the roadways. Existing ABC stone on the site can be stockpiled for re-use. Existing asphalt ground down to an ABC gradation and used in lifts of 4 inches below the ABC stone at the site.

### 6.1.2 Proofrolling

After removing all unsuitable surface materials, cutting to the proposed grade, and prior to the placement of any structural fill or other construction materials, the exposed subgrade should be examined by the Geotechnical Engineer or authorized representative. The exposed subgrade should be thoroughly proofrolled with previously approved construction equipment having a minimum axle load of 10 tons (e.g. fully loaded tandem-axle dump truck). The areas subject to proofrolling should be traversed by the equipment in two perpendicular (orthogonal) directions with overlapping passes of the vehicle under the observation of the Geotechnical Engineer or authorized representative. This procedure is intended to assist in identifying any localized yielding materials. In the event that unstable or "pumping" subgrade is identified by the proofrolling, those areas should be marked for repair prior to the placement of any subsequent structural fill or other construction materials. Methods of repair of unstable subgrade, such as undercutting or moisture conditioning, should be discussed with the Geotechnical Engineer to determine the appropriate procedure with regard to the existing conditions causing the instability. Test pits may be excavated to explore the shallow subsurface materials in the area of the instability to help in determined the cause of the observed unstable materials and to assist in the evaluation of the appropriate remedial action to stabilize the subgrade.

## 6.2 EARTHWORK OPERATIONS

### 6.2.1 Structural Fill Materials

**Product Submittals:** Prior to placement of structural fill, representative bulk samples (about 50 pounds) of on-site and off-site borrow should be submitted to ECS for laboratory testing, which will include Atterberg limits, natural moisture content, grain-size distribution, and moisture-density relationships for compaction. Import materials should be tested prior to being hauled to the site to determine if they meet project specifications.

**Satisfactory Structural Fill Materials:** Materials satisfactory for use as structural fill should consist of inorganic soils classified as SM, SC, SW, SP, GW, GP, GM and GC, or a combination of these group symbols, per ASTM D 2487. Natural fine-grained soils classified as clays or silts (CL, ML) should generally not be considered for use as engineered fill, but may be evaluated by the geotechnical engineer to determine their suitability at the contractor's request. The materials should be free of organic matter, debris, and should contain no particle sizes greater than 4 inches in the largest dimension. Open graded materials, such as gravels (GW and GP), which contain void space in their mass should not be used in structural fills unless properly encapsulated with filter fabric. Suitable structural fill material should have the index properties shown in Table 6.2.1.1.

**Table 6.2.1.1 Structural Fill Index Properties**

| Location | LL | PI | Max % Fines Passing # 200 Sieve |
|---|---|---|---|
| Building Areas | 35 max | 9 max | 20 |
| Pavement Areas | 35 max | 9 max | 20 |

**Unsatisfactory Materials:** Materials that should not be used as engineered fill include topsoil, organic materials (OH, OL), and high plasticity clays and silts (CH, MH). Such materials removed during grading operations should be either stockpiled for later use in landscape fills, or placed in approved on or off-site disposal areas.

**On-Site Borrow Suitability:** The on-site near surface SANDS (SM, SP) in the upper 8 feet across the site with fines contents less than 20 percent and free of deleterious material and roots should be suitable for re-use as structural fill.

The soils encountered at depths of 1.5 to 3.0 feet in the hand augers performed on January 31, 2019, had natural moisture contents that ranged from 4.6 to 8.6 percent. Based on the standard proctor test performed on the bulk sample obtained on site at depths of 1 to 3 feet the optimum moisture is 12.8 percent with a maximum dry density of 108.0 pounds per cubic foot. Moisture conditioning by means of adding water to the soil should be anticipated for the soils to achieve the optimum moisture content for fill placement.

### 6.2.2 Compaction

**Structural Fill Compaction:** Structural fill within the expanded buildings, pavement, and embankment limits should be placed in maximum 8-inch loose lifts, moisture conditioned as necessary to within -3 and +3 % of the soil's optimum moisture content, and be compacted with suitable equipment to a dry density of at least 98% of the standard Proctor maximum dry density (ASTM D-698). Beyond these areas, compaction of at least 95% should be achieved. ECS should be called on to document that proper fill compaction has been achieved.

**Fill Compaction Control:** The expanded limits of the proposed construction areas should be well defined, including the limits of the fill zones for buildings, pavements, and slopes, etc., at the time of fill placement. Grade controls should be maintained throughout the filling operations. All filling operations should be observed on a full-time basis by a qualified representative of the construction testing laboratory to determine that the minimum compaction requirements are being achieved. Field density testing of fills will be performed at the frequencies shown in Table 6.2.2.1, but not less than 1 test per lift.

**Table 6.2.2.1 Frequency of Compaction Tests in Fill Areas**

| Location | Frequency of Tests |
|---|---|
| Building Areas | 1 test per 2,500 sq. ft. per lift |
| Pavement Areas | 1 test per 10,000 sq. ft. per lift |
| Utility Trenches | 1 test per 200 linear ft. per lift |

**Compaction Equipment:** Compaction equipment suitable to the soil type being compacted should be used to compact the subgrades and fill materials. A vibratory steel drum roller should be used for compaction of coarse-grained soils (Sands) as well as for sealing compacted surfaces.

**Fill Placement Considerations:** Fill materials should not be placed on frozen soils, on frost-heaved soils, and/or on excessively wet soils. Borrow fill materials should not contain frozen materials at the time of placement, and all frozen or frost-heaved soils should be removed prior to placement of Structural Fill or other fill soils and aggregates. Excessively wet soils or aggregates should be scarified, aerated, and moisture conditioned.

At the end of each work day, all fill areas should be graded to facilitate drainage of any precipitation and the surface should be sealed by use of a smooth-drum roller to limit infiltration of surface water. During placement and compaction of new fill at the beginning of each workday, the Contractor may need to scarify existing subgrades to a depth on the order of 4 inches so that a weak plane will not be formed between the new fill and the existing subgrade soils.

Drying and compaction of wet soils is typically difficult during the cold, winter months. Accordingly, earthwork should be performed during the warmer, drier times of the year, if practical. Proper drainage should be maintained during the earthwork phases of construction to prevent ponding of water which has a tendency to degrade subgrade soils.

Where fill materials will be placed to widen existing embankment fills, or placed up against sloping ground, the soil subgrade should be scarified and the new fill benched or keyed into the existing material. Fill material should be placed in horizontal lifts. In confined areas such as utility trenches, portable compaction equipment and thin lifts of 3 inches to 4 inches may be required to achieve specified degrees of compaction.

We recommend that the grading contractor have equipment on site during earthwork for both drying and wetting fill soils. We do not anticipate significant problems in controlling moisture within the fill during dry weather, but moisture control may be difficult during winter months or extended periods of rain. The control of moisture content of higher plasticity soils is difficult when these soils become wet. Further, such soils are easily degraded by construction traffic when the moisture content is elevated.

## 6.3 FOUNDATION OBSERVATIONS

**Protection of Foundation Excavations:** Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time. Therefore, foundation concrete should be placed the same day that excavations are made. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, a 2 to 3-inch thick “mud mat” of “lean” concrete should be placed on the bearing soils before the placement of reinforcing steel.

**Footing Subgrade Observations:** The preparation of fill subgrades, as well as proposed building subgrades, should be observed on a full-time basis by ECS personnel. These observations should be performed by an experienced geotechnical engineer or qualified person to ensure that unsuitable materials have been removed and that the prepared subgrade meets project requirements for support of the proposed construction and/or fills.

## 6.4 UTILITY INSTALLATIONS

**Utility Subgrades:** Most of the soils encountered in our exploration are expected to be suitable for support of utility pipes. The pipe subgrade should be observed and probed for stability by ECS to evaluate the suitability of the materials encountered. Any loose or unsuitable materials encountered at the utility pipe subgrade elevation should be removed and replaced with suitable compacted Structural Fill or pipe bedding material.

**Utility Backfilling:** The granular bedding material should be at least 4 inches thick, but not less than that specified by the project drawings and specifications. Fill placed for support of the utilities, as well as backfill over the utilities, should satisfy the requirements for structural fill given in this report. Compacted backfill should be free of topsoil, roots, ice, or any other material designated by ECS as unsuitable. The backfill should be moisture conditioned, placed, and compacted in accordance with the recommendations of this report.

**Utility Excavation Dewatering:** It is possible that perched water may be encountered by utility excavations which extend below existing grades. It is expected that removal of perched water which seeps into excavations could be accomplished by pumping from sumps excavated in the trench bottom and which are backfilled with DOT Size No. 57 Stone or open graded bedding material. Should water conditions beyond the capability of sump pumping be encountered, the contractor should submit a Dewatering Plan in accordance with project specifications.

## 6.5 GENERAL CONSTRUCTION CONSIDERATIONS

**Moisture Conditioning:** During the cooler and wetter periods of the year, delays and additional costs should be anticipated. At these times, reduction of soil moisture may need to be accomplished by mechanical manipulation, in order to lower moisture contents to levels appropriate for compaction. Alternatively, during the drier times of the year, such as the summer months, moisture may need to be added to the soil to provide adequate moisture for successful compaction according to the project requirements.

**Subgrade Protection:** Measures should also be taken to limit site disturbance, especially from rubber-tired heavy construction equipment, and to control and remove surface water from development areas, including structural and pavement areas. It would be advisable to designate a haul road and construction staging area to limit the areas of disturbance and to prevent construction traffic from excessively degrading sensitive subgrade soils and existing pavement areas. Haul roads and construction staging areas could be covered with excess depths of aggregate to protect those subgrades. The aggregate can later be removed and used in pavement areas.

**Surface Drainage:** Surface drainage conditions should be properly maintained. Surface water should be directed away from the construction area, and the work area should be sloped away from the construction area at a gradient of 1 percent or greater to reduce the potential of ponding water and the subsequent saturation of the surface soils. At the end of each work day, the subgrade soils should be sealed by rolling the surface with a smooth drum roller to minimize infiltration of surface water.

**Excavation Safety:** All excavations and slopes should be made and maintained in accordance with OSHA excavation safety standards. The contractor is solely responsible for designing and constructing stable, temporary excavations and slopes and should shore, slope, or bench the sides of the excavations and slopes as required to maintain stability of both the excavation sides and bottom. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. ECS is providing this information solely as a service to our client. ECS is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

**Excavation Considerations:** Based on the results of the soundings, we expect that the natural Coastal Plain soils encountered on this site can be excavated with conventional earth moving equipment such as loaders, bulldozers, rubber tired backhoes, etc.

The site soils are OSHA Type C soils for the purpose of temporary excavation support. Excavations should be constructed in compliance with current OSHA standards for excavation and trenching safety. Excavations should be observed by a "competent person," as defined by OSHA, who should evaluate the specific soil type and other conditions, which may control the excavation side slopes or the need for shoring or bracing. Regardless, site safety shall be the sole responsibility of the contractor and their subcontractors. Exposed earth slopes shall be protected during periods of inclement weather.

**Erosion Control:** The surface soils may be erodible. Therefore, the Contractor should provide and maintain good site drainage during earthwork operations to maintain the integrity of the surface soils. All erosion and sedimentation controls should be in accordance with sound engineering practices and local requirements.

## 7.0 CLOSING

ECS has prepared this report of findings, evaluations, and recommendations to guide geotechnical-related design and construction aspects of the project.

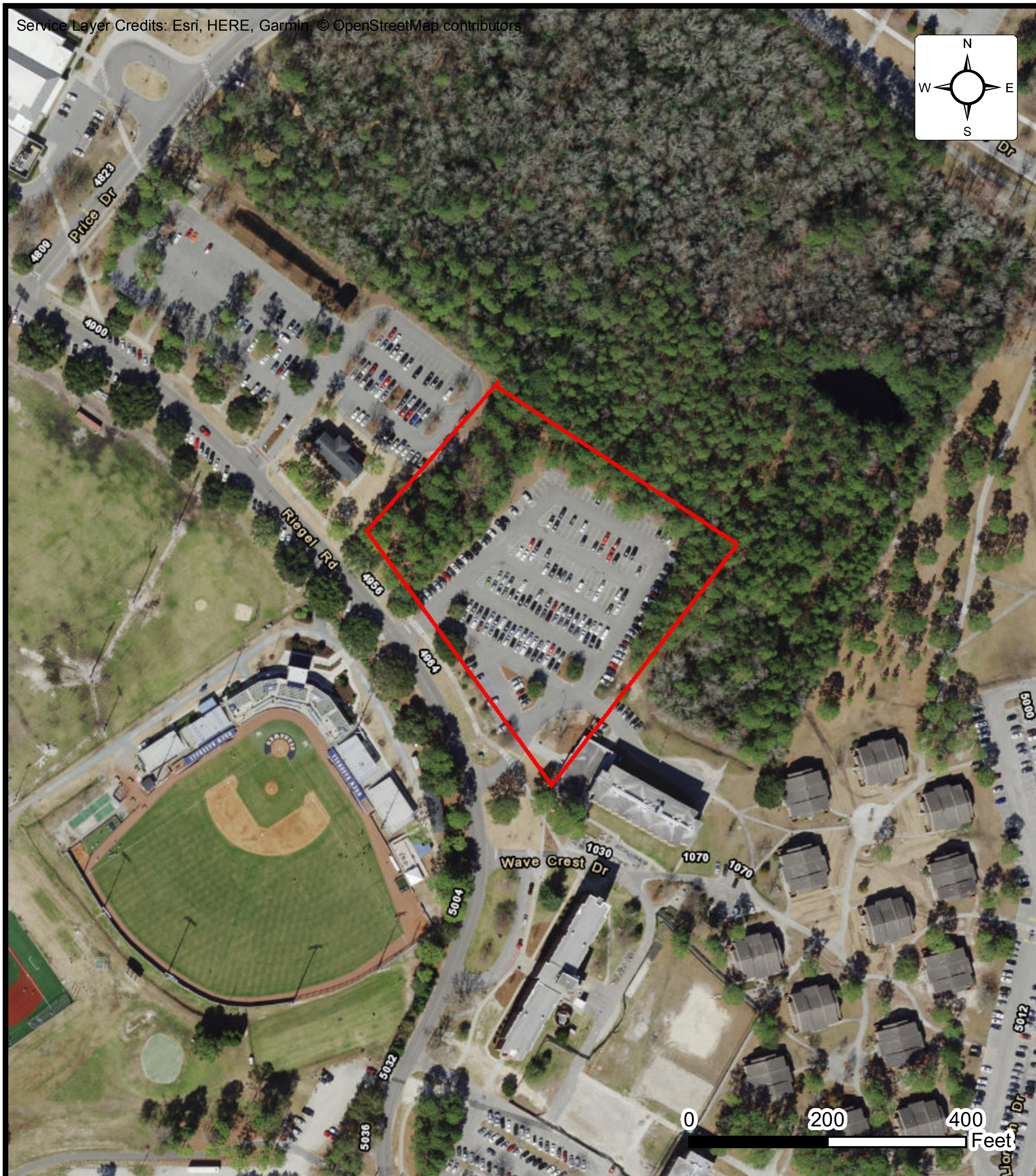
The description of the proposed project is based on information provided to ECS by Mr. Richard Collier of McKim & Creed. If any of this information is inaccurate, either due to our interpretation of the documents provided or site or design changes that may occur later, ECS should be contacted immediately so that we can review the report in light of the changes and provide additional or alternate recommendations as may be required to reflect the proposed construction.

We recommend that ECS be allowed to review the project's plans and specifications pertaining to our work so that we may ascertain consistency of those plans/specifications with the intent of the geotechnical report.

Field observations, monitoring, and quality assurance testing during earthwork and foundation installation are an extension of and integral to the geotechnical design recommendation. We recommend that the owner retain these quality assurance services and that ECS be allowed to continue our involvement throughout these critical phases of construction to provide general consultation as issues arise. ECS is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

## APPENDIX A – Drawings & Reports

Site Location Diagram
Exploration Location Diagram




# Site Location Diagram
# UNCW PARKING DECK

**RIEGEL ROAD, WILMINGTON, NC**

**MCKIM & CREED**

| | |
|---|---|
| ENGINEER | WEG |
| SCALE | 1 " = 200 ' |
| PROJECT NO. | 22:27313 |
| SHEET | 1 OF 2 |
| DATE | 12/11/2018 |

# EXPLORATION LOCATION DIAGRAM

## UNCW Parking Deck

Wilmington, North Carolina

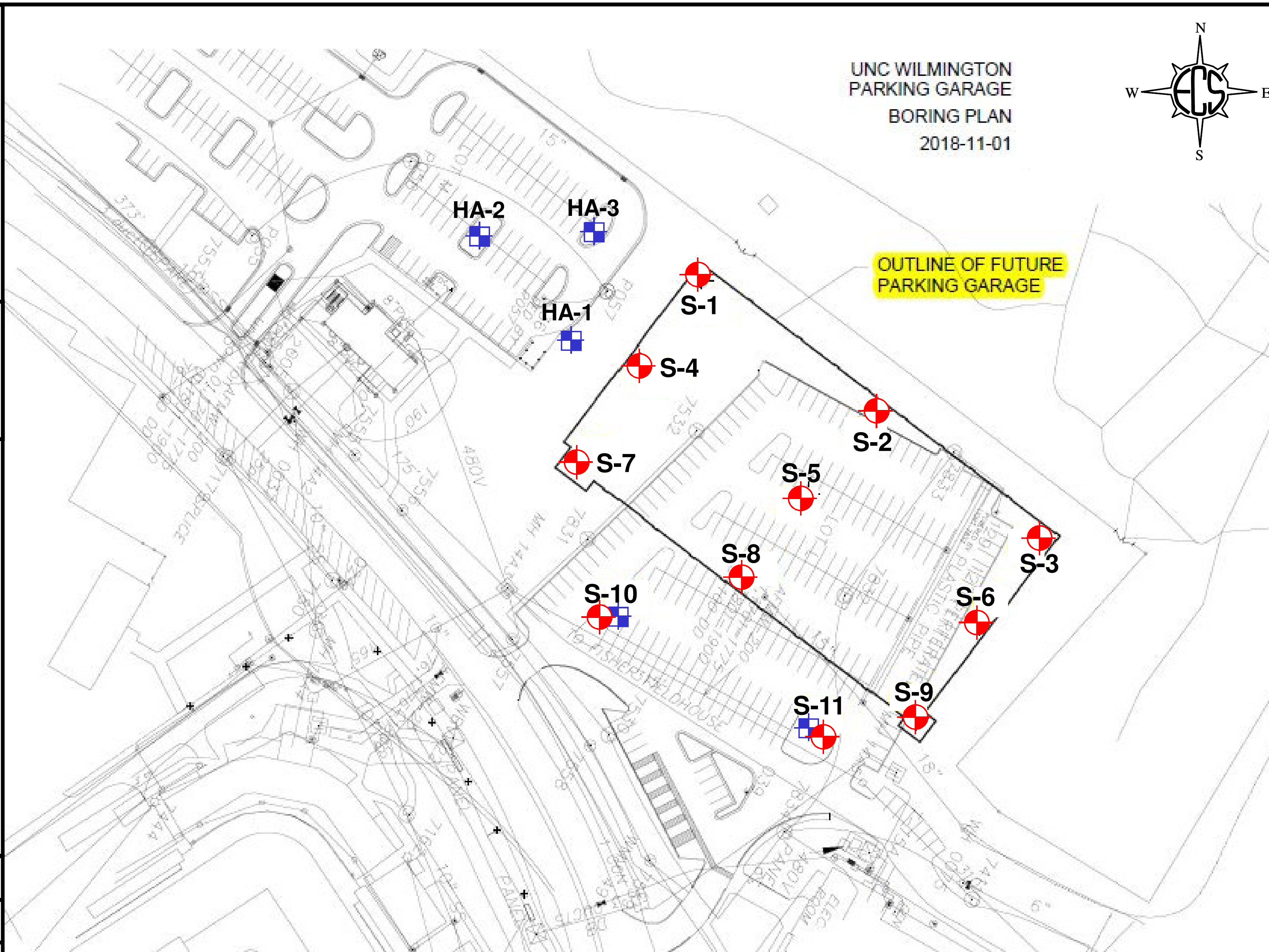
| ENGINEER | DRAFTING |
|---|---|
| WEG | ACC |

SCALE: NTS

Project NO. 22-27313R1

SHEET 2 of 2

DATE 2/4/2018




DENOTES APPROXIMATE LOCATION OF CPT SOUNDING

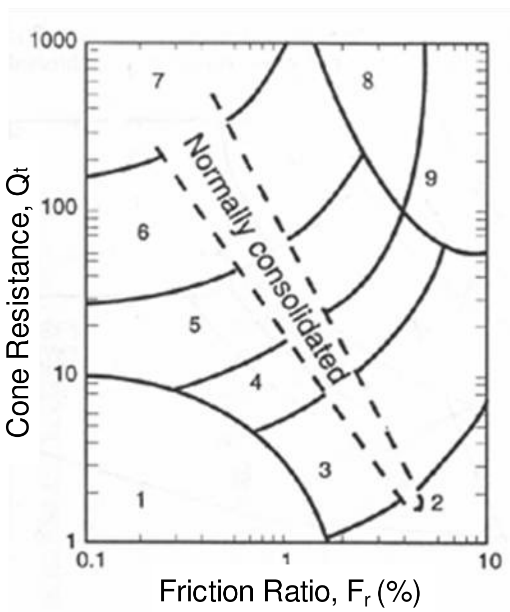
DENOTES APPROXIMATE LOCATION OF HAND AUGER BORINGS WITH KESSLER DCP

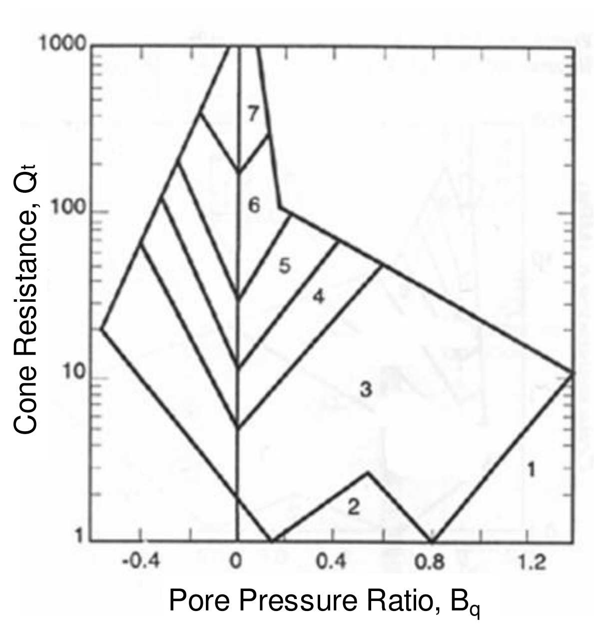
## APPENDIX B – Field Operations

Reference Notes for Sounding Logs
CPT Sounding Logs S-1 through S-11
Hand Auger Boring Logs
Kessler DCP Test Results

## REFERENCE NOTES FOR CONE PENETRATION TEST (CPT) SOUNDINGS

In the CPT sounding procedure (ASTM-D-5778), an electronically instrumented cone penetrometer is hydraulically advanced through soil to measure point resistance ($q_c$), pore water pressure ($u_2$), and sleeve friction ($f_s$). These values are recorded continuously as the cone is pushed to the desired depth. CPT data is corrected for depth and used to estimate soil classifications and intrinsic soil parameters such as angle of internal friction, preconsolidation pressure, and undrained shear strength. The graphs below represent one of the accepted methods of CPT soil behavior classification (Robertson, 1990).




1. Sensitive, Fine Grained
2. Organic Soils-Peats
3. Clays; Clay to Silty Clay
4. Clayey Silt to Silty Clay
5. Silty Sand to Sandy Silt
6. Clean Sands to Silty Sands
7. Gravelly Sand to Sand
8. Very Stiff Sand to Clayey Sand
9. Very Stiff Fine Grained

The following table presents a correlation of corrected cone tip resistance ($q_c$) to soil consistency or relative density:

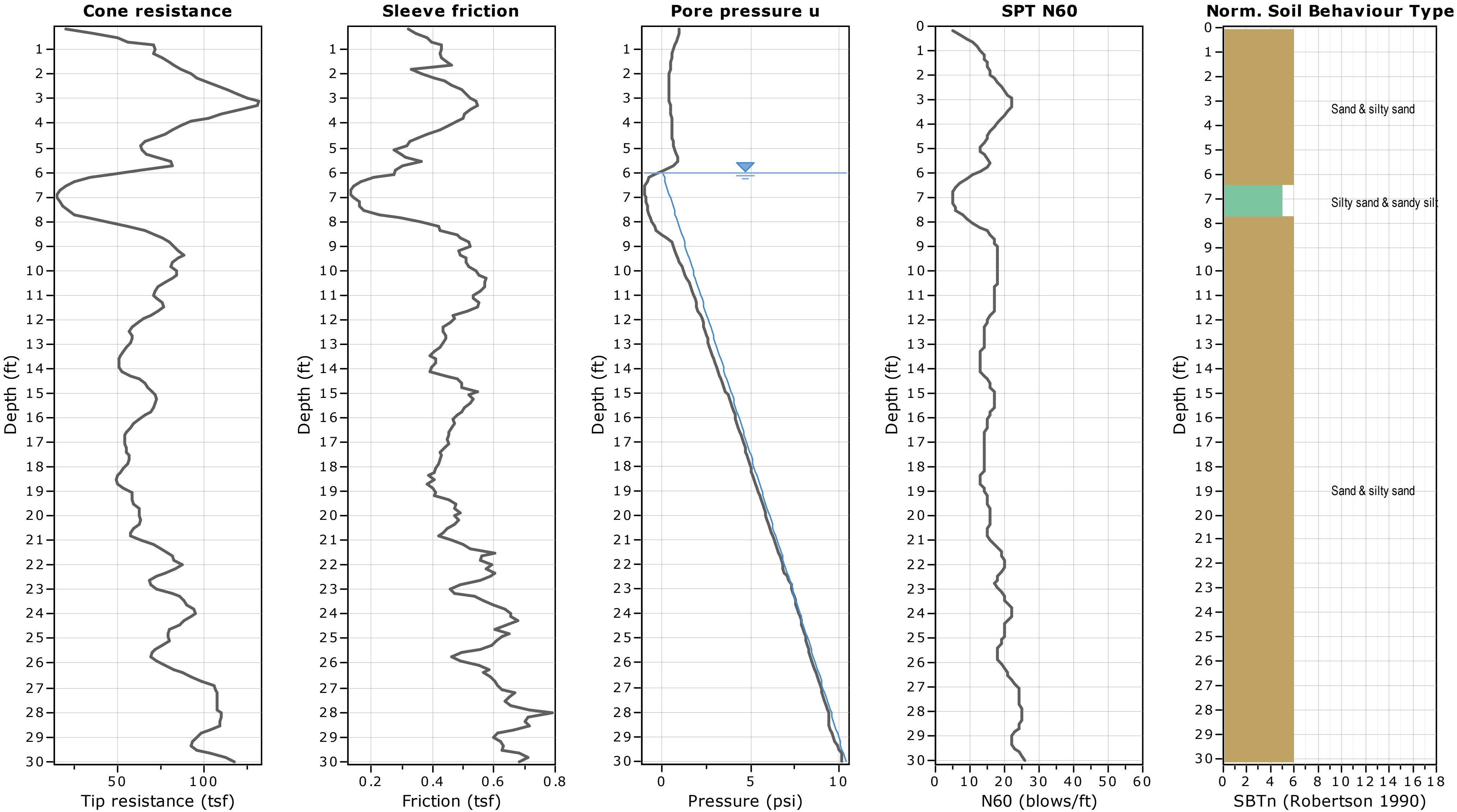
| SAND | | SILT/CLAY | |
|---|---|---|---|
| **Corrected Cone Tip Resistance ($q_c$) (tsf)** | **Relative Density** | **Corrected Cone Tip Resistance ($q_c$) (tsf)** | **Relative Density** |
| <20 | Very Loose | <5 | Very Soft |
| 20-40 | Loose | 5-10 | Soft |
| 40-120 | Medium Dense | 10-15 | Medium Stiff |
| | | 15-30 | Stiff |
| 120-200 | Dense | 30-45 | Very Stiff |
| >200 | Very Dense | 45-60 | Hard |
| | | >60 | Very Hard |

**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-1**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 30.02 ft, Date: 11/21/2018
Cone Operator: Cory Robison

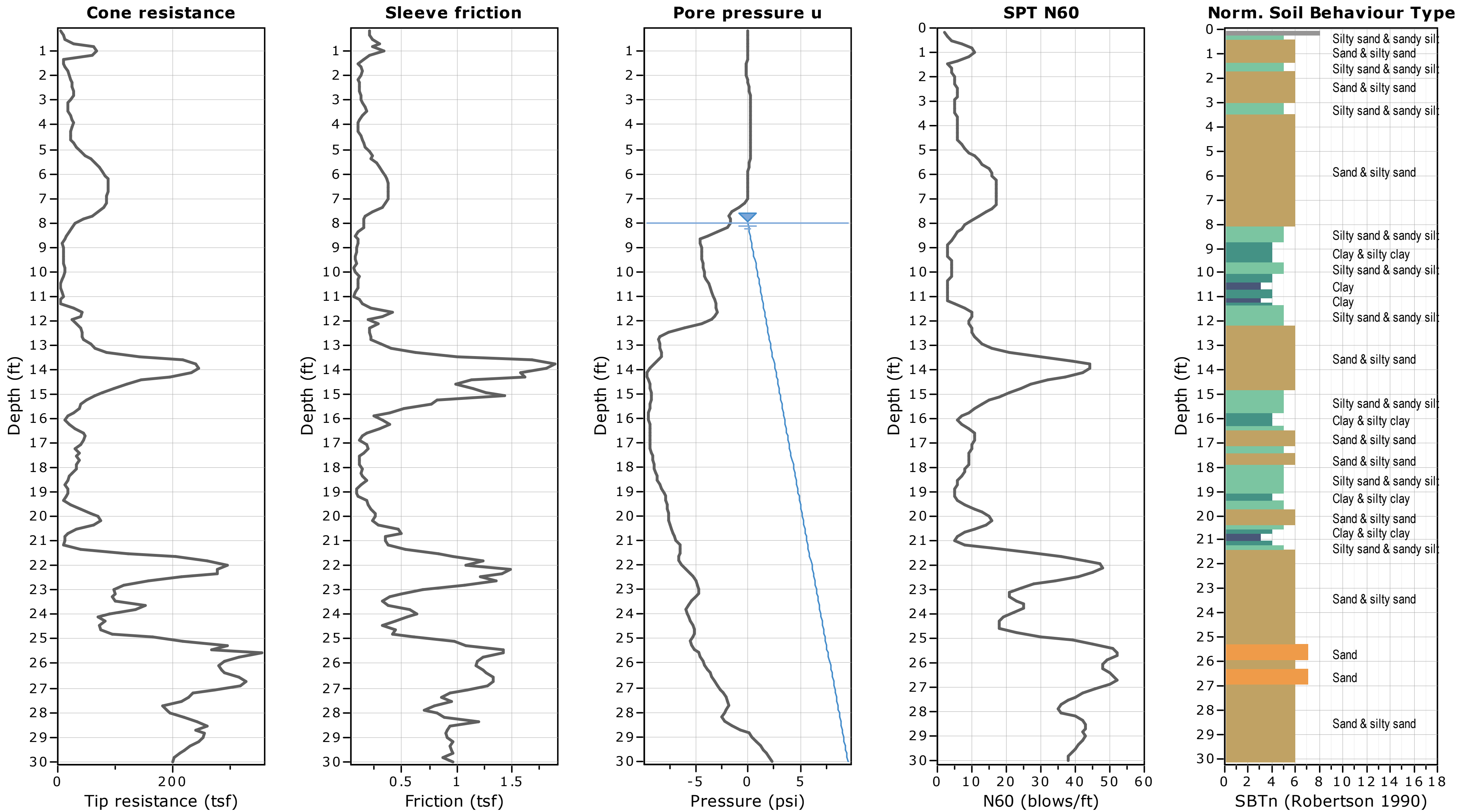


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-2**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 30.02 ft, Date: 11/21/2018
Cone Operator: Cory Robison



**ECS Southeast, LLP**
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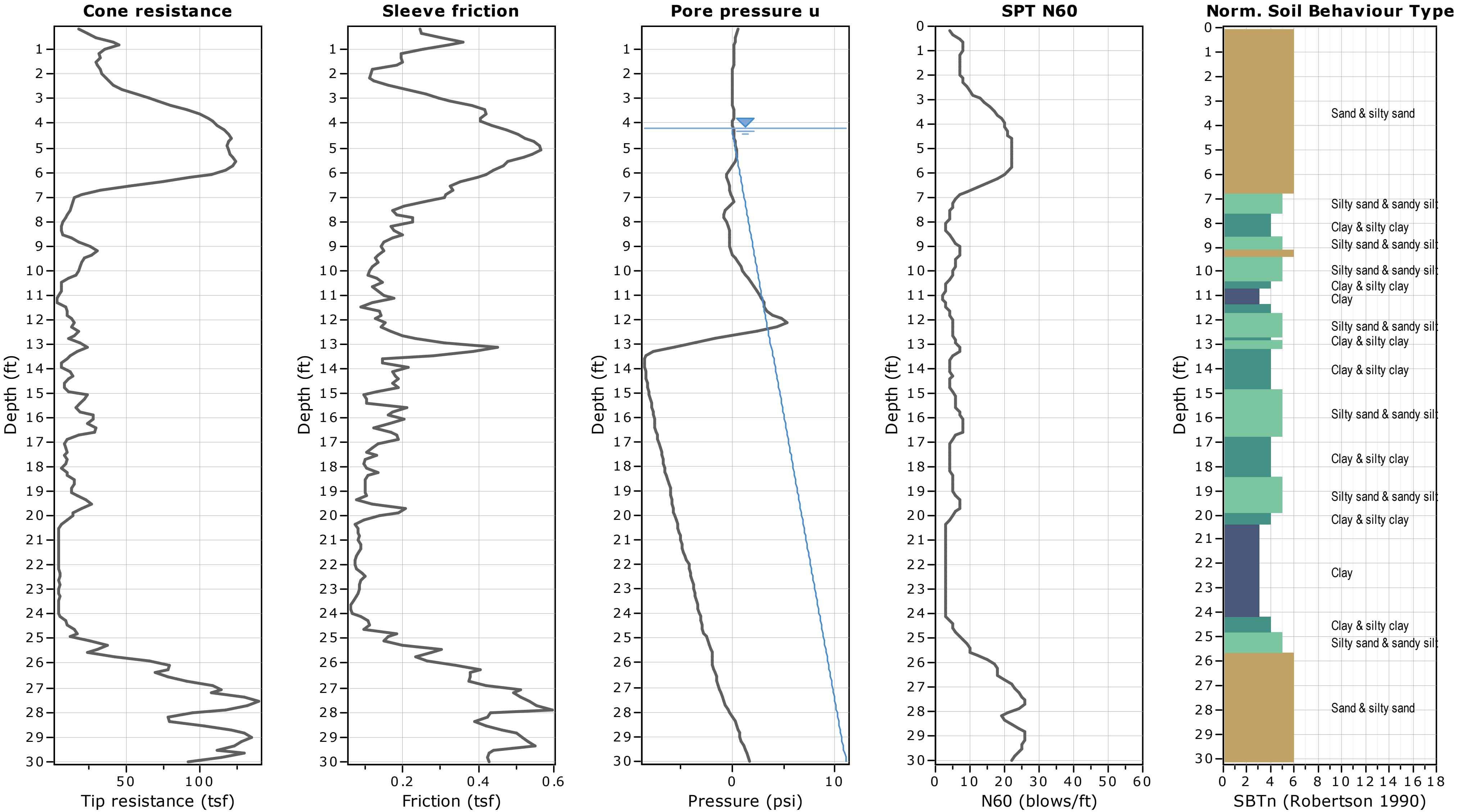
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-3**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 30.02 ft, Date: 11/21/2018
Cone Operator: Cory Robison

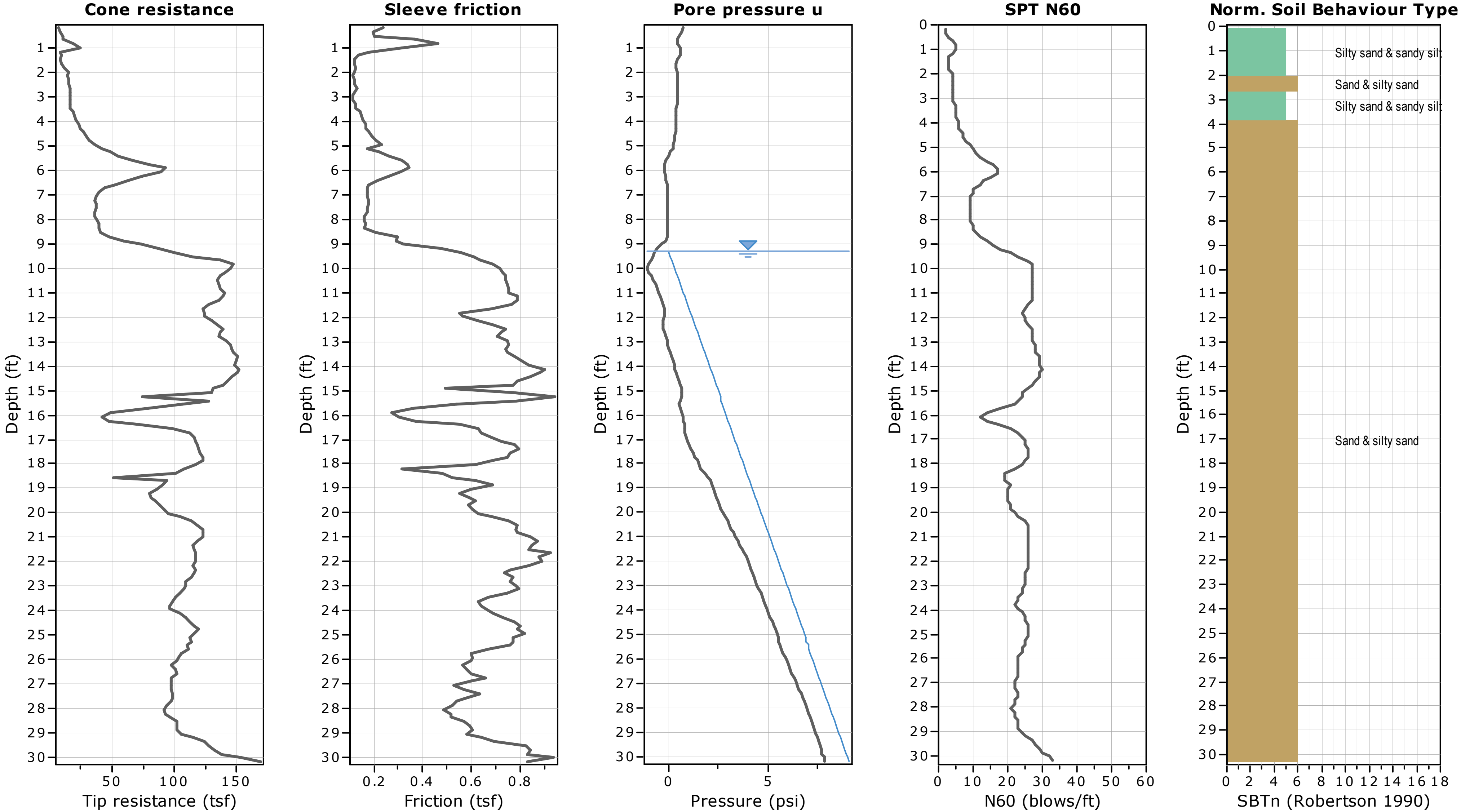


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-4**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 30.18 ft, Date: 11/21/2018
Cone Operator: Cory Robison



**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

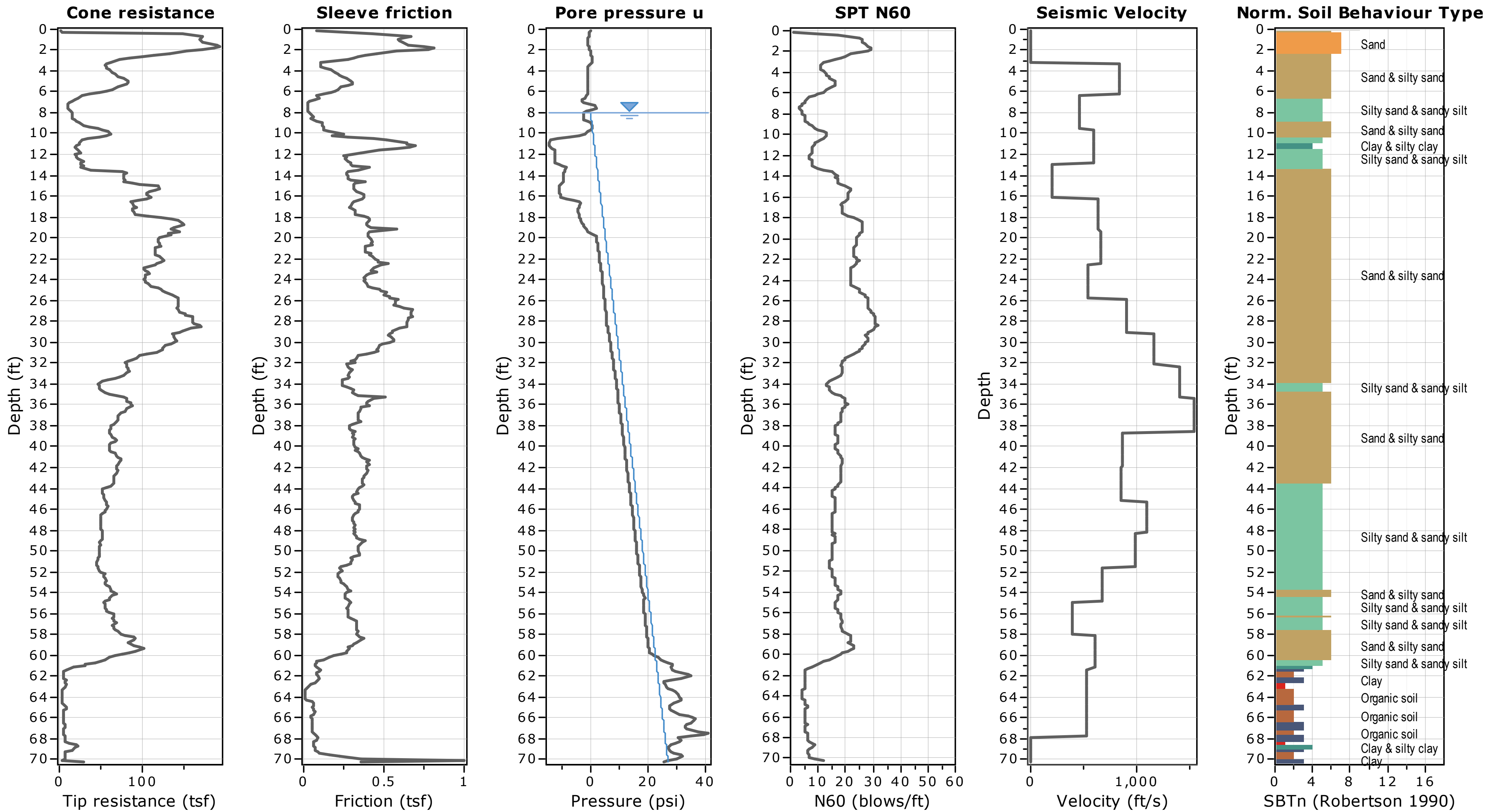
**CPT: S-5**

Total depth: 70.21 ft, Date: 11/21/2018

Cone Operator: Austin Fowler

**Project:** **UNCW Parking Deck**

**Location:** **Wilmington, New Hanover County, North Carolina**

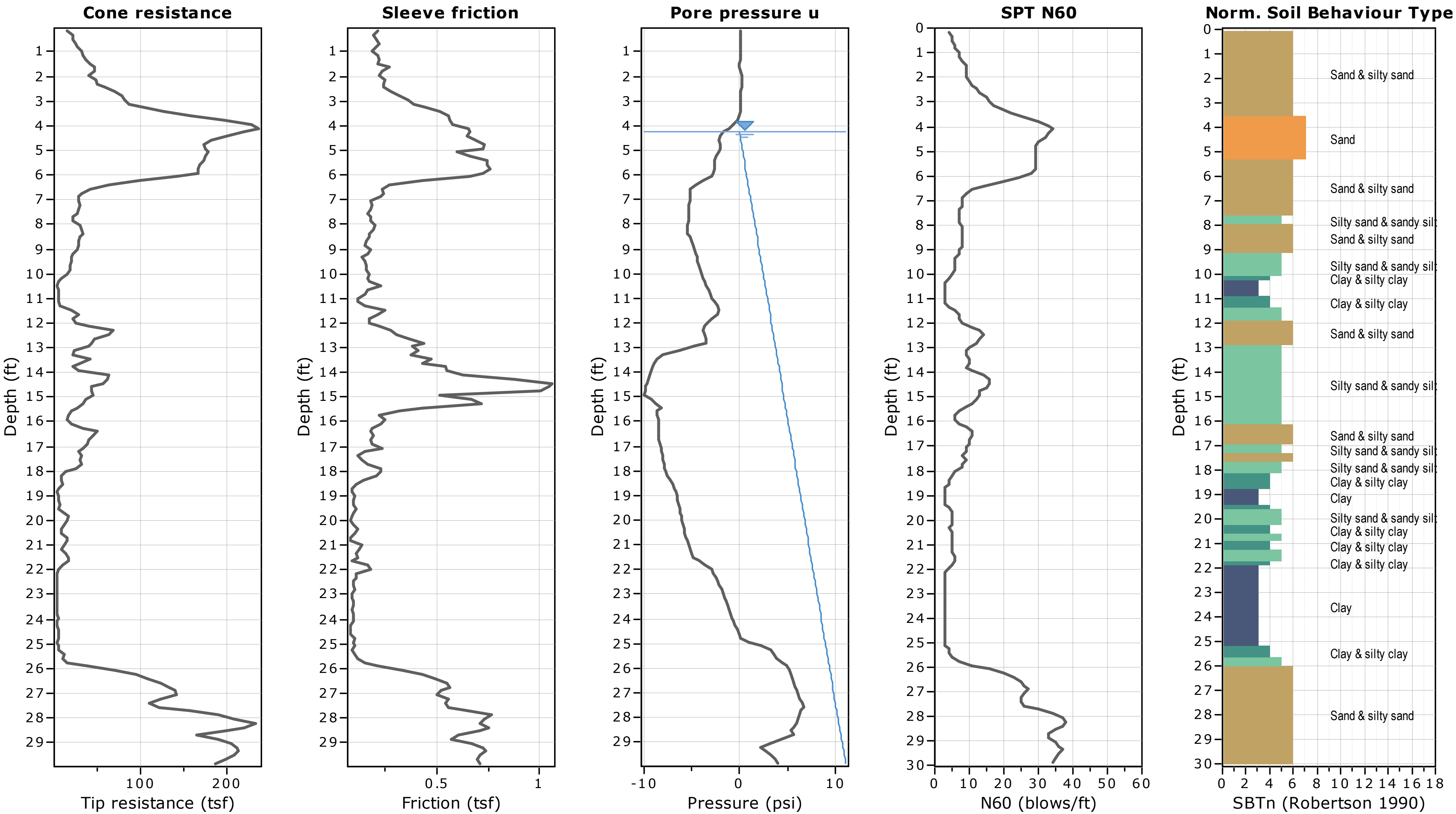


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-6**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 29.86 ft, Date: 11/21/2018
Cone Operator: Cory Robison

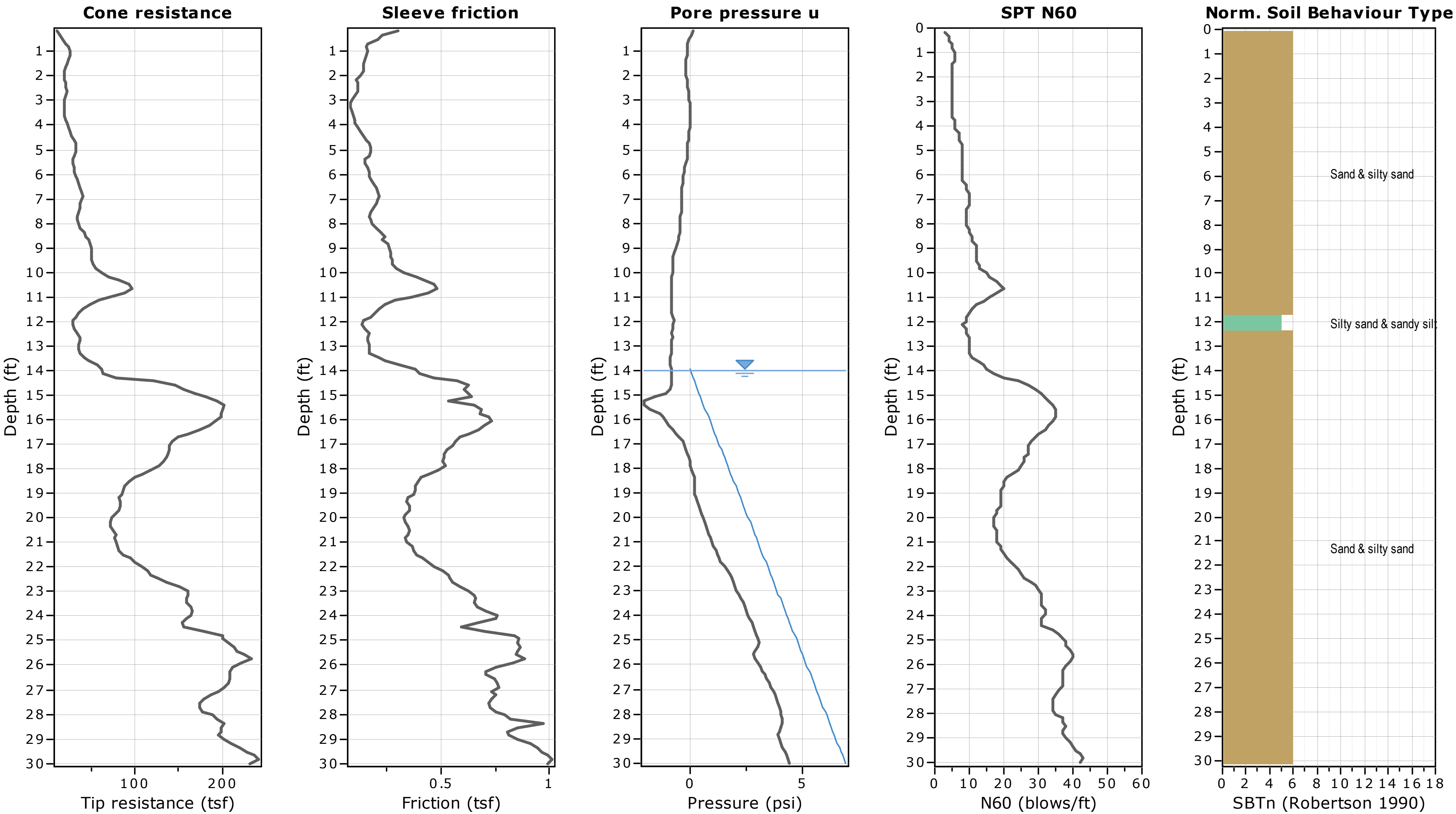


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-7**

Total depth: 30.02 ft, Date: 11/21/2018
Cone Operator: Cory Robison

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**



**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

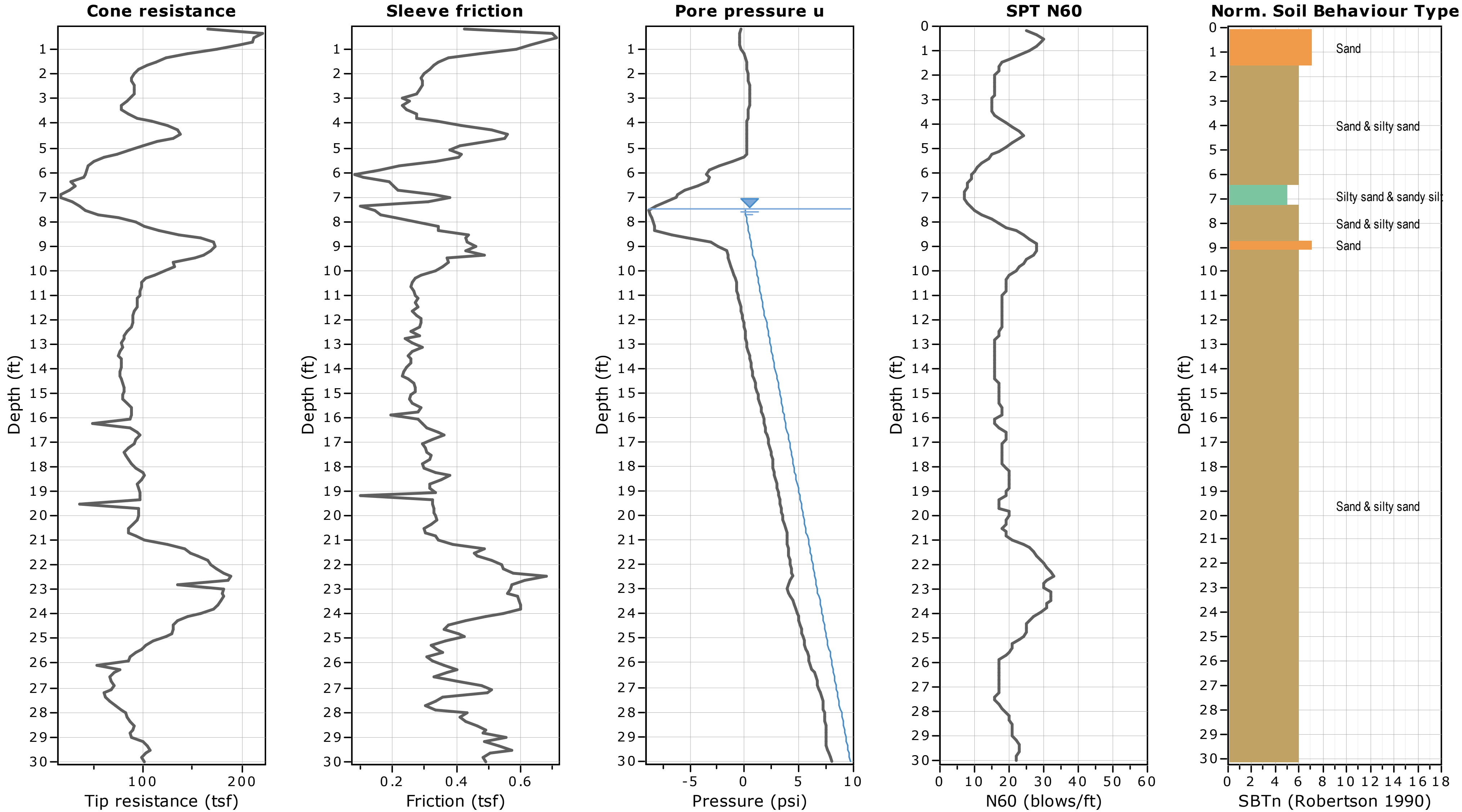
**CPT: S-8**

**Project:** **UNCW Parking Deck**

**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 30.02 ft, Date: 11/21/2018

Cone Operator: Austin Fowler

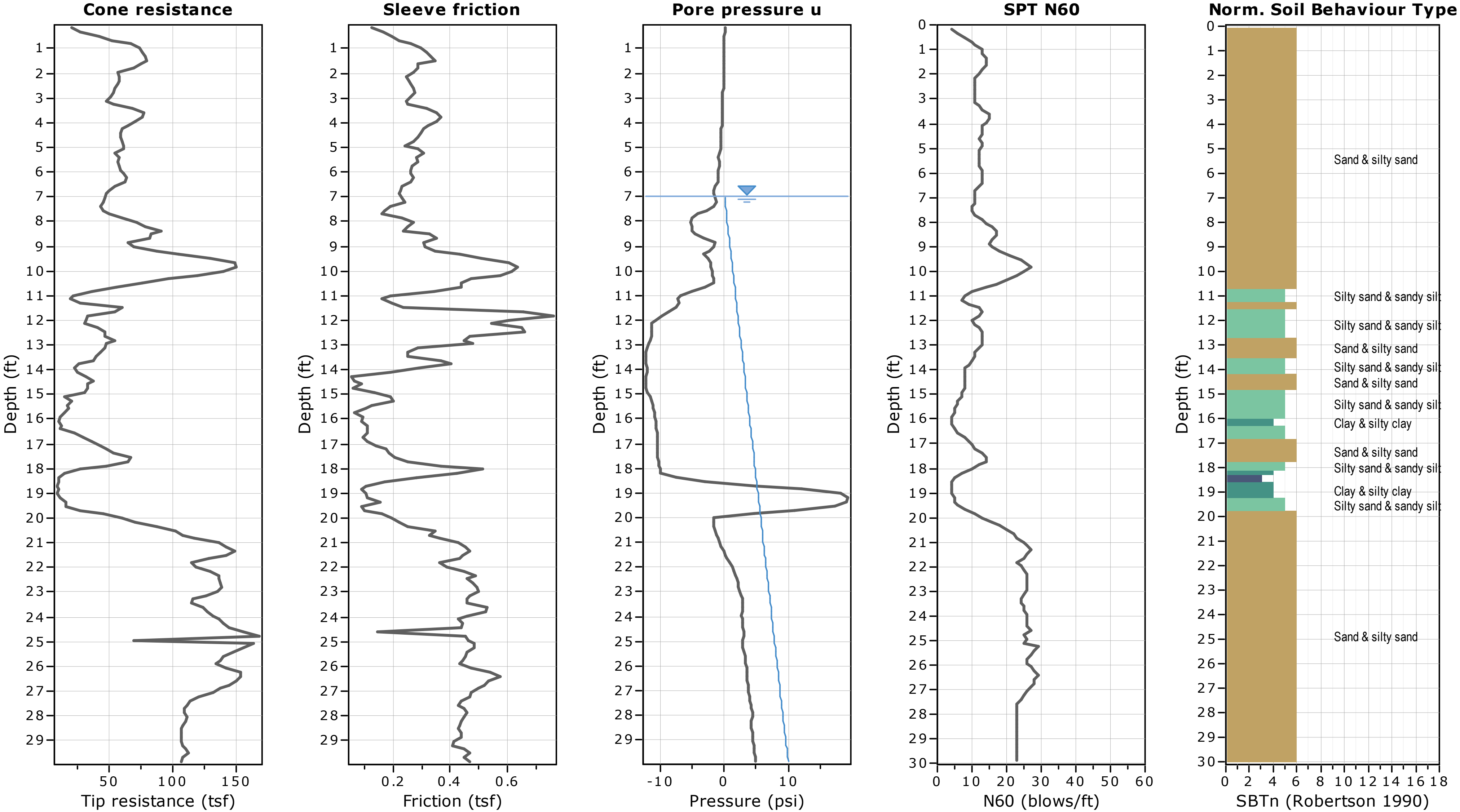


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-9**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 29.86 ft, Date: 11/21/2018
Cone Operator: Cory Robison

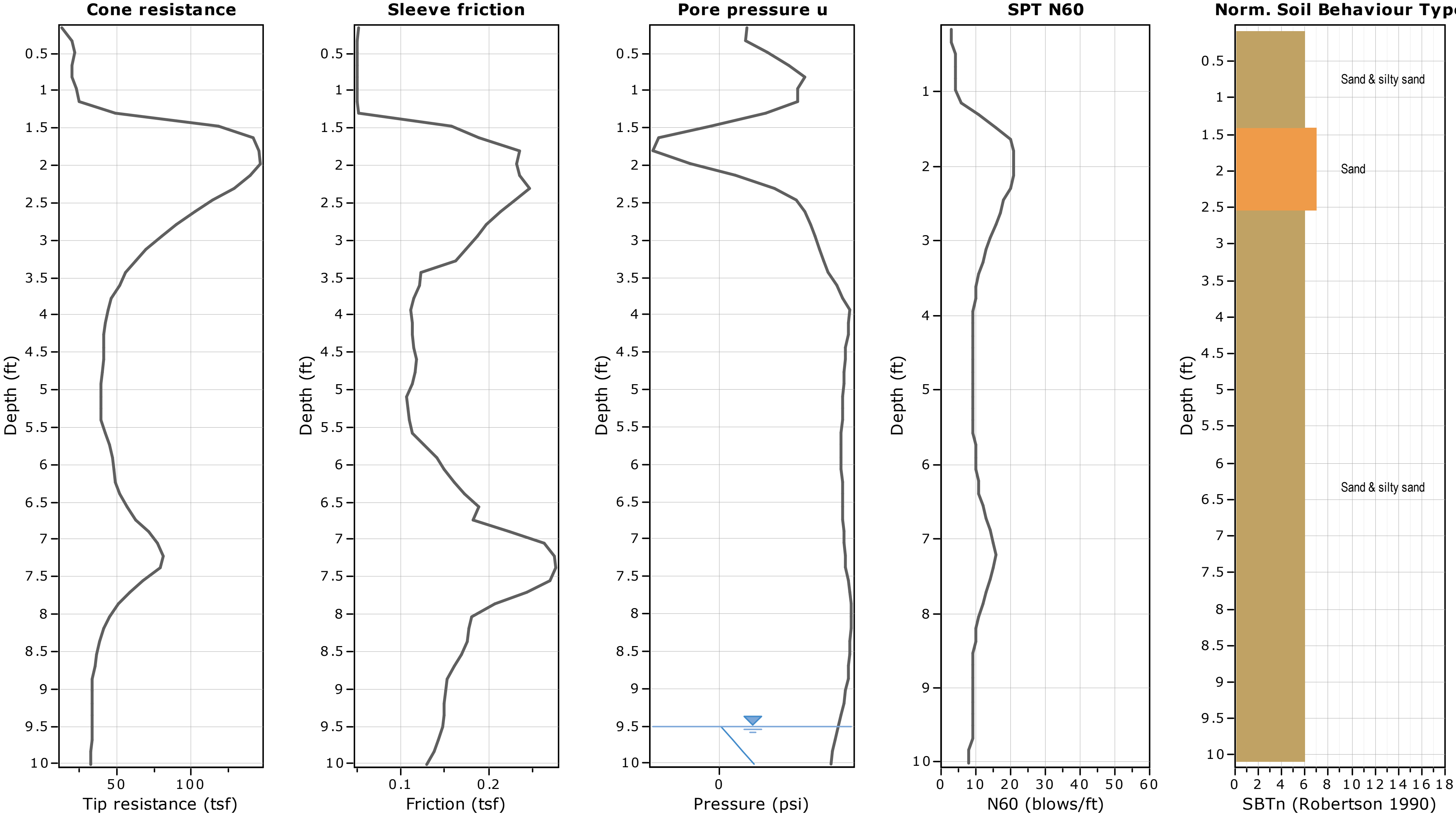


ECS Southeast, LLP
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-10**

**Project:** UNCW Parking Deck
**Location:** Wilmington, New Hanover County, North Carolina

Total depth: 10.01 ft, Date: 11/21/2018
Cone Operator: Austin Fowler

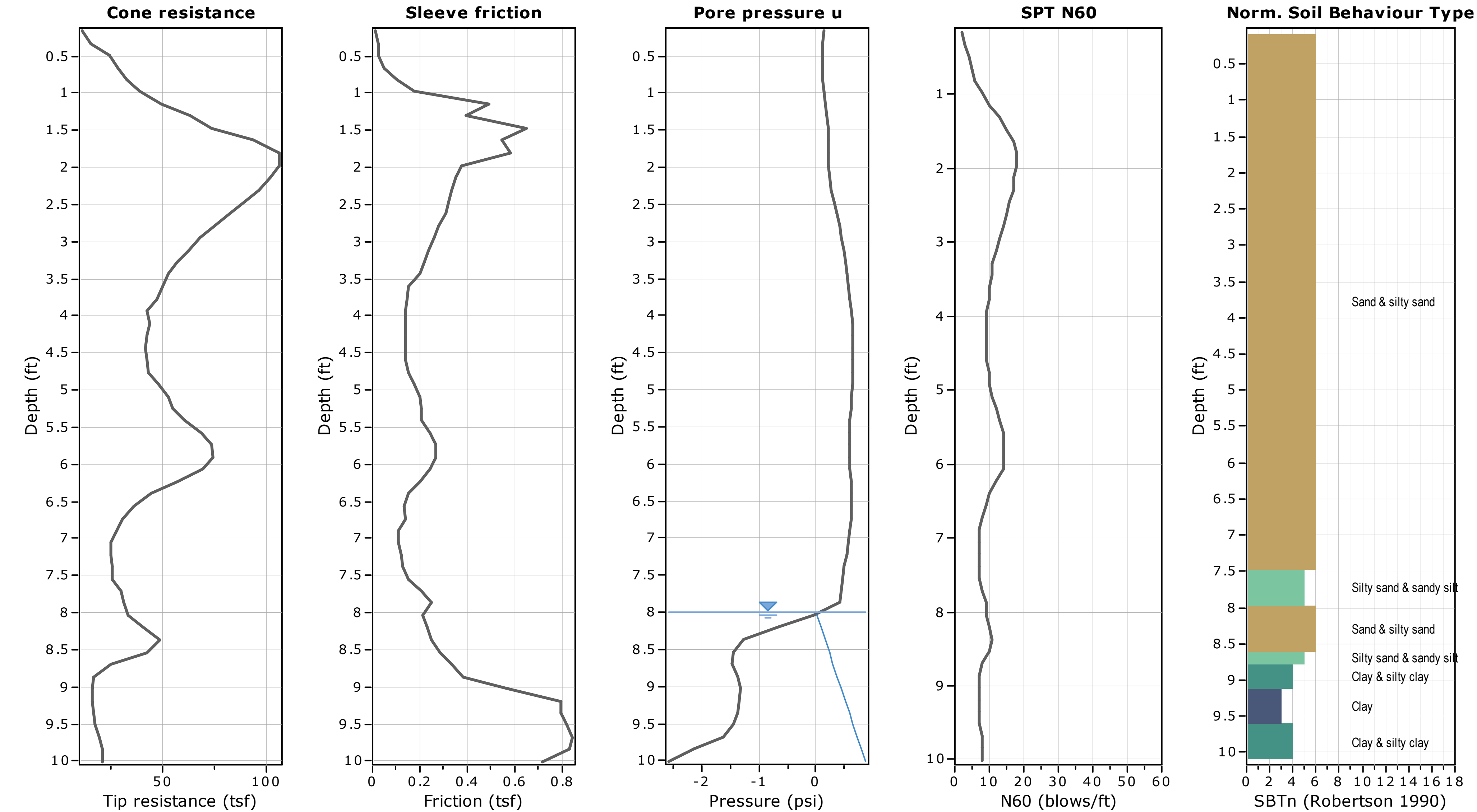


**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
ECS Project # 22-27313

**CPT: S-11**

**Project:** **UNCW Parking Deck**
**Location:** **Wilmington, New Hanover County, North Carolina**

Total depth: 10.01 ft, Date: 11/21/2018
Cone Operator: Austin Fowler



| CLIENT | Job #: | BORING # | SHEET |
|---|---|---|---|
| McKim & Creed | 22:27313 | S-10 | 1 OF 1 |

PROJECT NAME: UNCW Parking Deck

ARCHITECT-ENGINEER:

SITE LOCATION: Riegel Road, Wilmington, New Hanover County, North Carolina

NORTHING | EASTING | STATION

| DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | RECOVERY (IN) | DESCRIPTION OF MATERIAL (ENGLISH UNITS) | WATER LEVELS | ELEVATION (FT) | BLOWS/6" |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | Topsoil/Rootmat Depth [3"] | | | |
| | | | | | (SP FILL) FINE SAND PROBABLE FILL, Brown, Moist | | | |
| 1 | | | | | (SP FILL) FINE SAND PROBABLE FILL, Tan, Moist | | | |
| 4 | | | | | END OF BORING @ 4.0' | | | |

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION

Legend: CALIBRATED PENETROMETER TONS/FT$^2$; ROCK QUALITY DESIGNATION & RECOVERY — RQD% – – –, REC% ———; PLASTIC LIMIT% — WATER CONTENT% — LIQUID LIMIT%; ⊗ STANDARD PENETRATION BLOWS/FT

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

| | | |
|---|---|---|
| WL  WS ☐  WD ☐ | BORING STARTED 11/20/18 | CAVE IN DEPTH |
| WL(SHW)  WL(ACR) | BORING COMPLETED 11/20/18 | HAMMER TYPE |
| WL | RIG  FOREMAN | DRILLING METHOD Hand Auger |

# DCP TEST DATA

**Project:** *UNCW Parking Deck* **Date:** 20-Nov-18

**Location:** S-10 **Soil Type(s):** SAND (SP, SP FILL)

Hammer
- ◯ 10.1 lbs.
- ◉ 17.6 lbs.
- ◯ Both hammers used

Soil Type
- ◯ CH
- ◯ CL
- ◉ All other soils

| No. of Blows | Cumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 5 | 290 | 1 |
| 5 | 452 | 1 |
| 5 | 500 | 1 |
| 5 | 538 | 1 |
| 5 | 577 | 1 |
| 5 | 612 | 1 |
| 5 | 655 | 1 |
| 5 | 696 | 1 |
| 5 | 742 | 1 |
| 5 | 798 | 1 |
| 5 | 861 | 1 |
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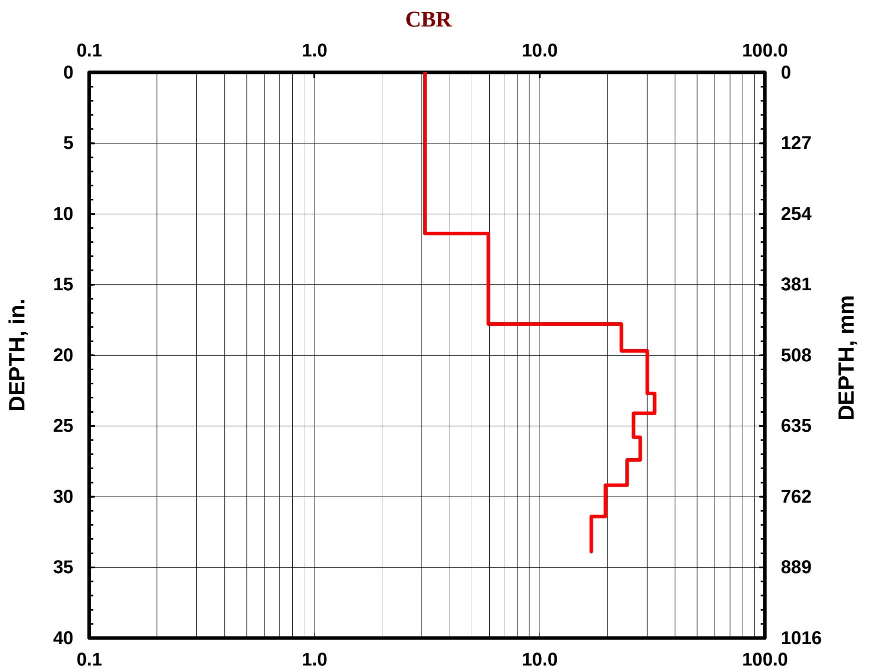


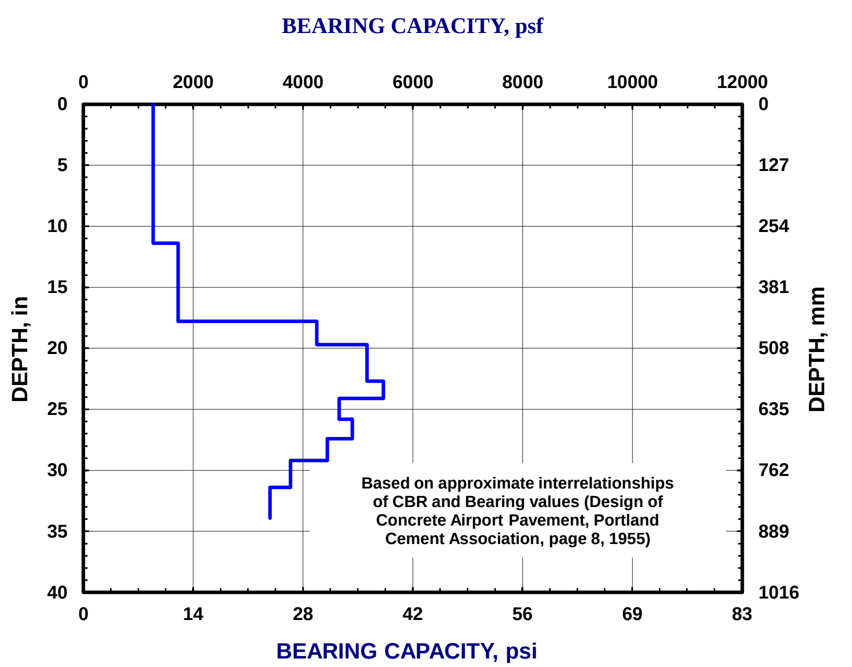

| CLIENT | Job #: | BORING # | SHEET |
|---|---|---|---|
| McKim & Creed | 22:27313 | S-11 | 1 OF 1 |

PROJECT NAME: UNCW Parking Deck

ARCHITECT-ENGINEER:

SITE LOCATION: Riegel Road, Wilmington, New Hanover County, North Carolina

NORTHING | EASTING | STATION

-○- CALIBRATED PENETROMETER TONS/FT$^2$

ROCK QUALITY DESIGNATION & RECOVERY
RQD% – — – REC% ———

PLASTIC LIMIT% × — WATER CONTENT% ● — LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION

| DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | RECOVERY (IN) | DESCRIPTION OF MATERIAL | WATER LEVELS | ELEVATION (FT) | BLOWS/6" |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | Topsoil/Rootmat Depth [3"] | | | |
| | | | | | (SP FILL) FINE SAND PROBABLE FILL, Brown, Moist | | | |
| 1 | | | | | (SP FILL) FINE SAND PROBABLE FILL, Tan, Moist, Contains Gravel | | | |
| 4 | | | | | END OF BORING @ 4.0' | | | |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

| | | |
|---|---|---|
| WL WS☐ WD☐ | BORING STARTED 11/20/18 | CAVE IN DEPTH |
| WL(SHW) WL(ACR) | BORING COMPLETED 11/20/18 | HAMMER TYPE |
| WL | RIG FOREMAN | DRILLING METHOD Hand Auger |

# DCP TEST DATA

**Project:** *UNCW Parking Deck*
**Date:** 20-Nov-18
**Location:** S-11
**Soil Type(s):** SAND (SP, SP FILL)

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

| No. of Blows | Cumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 5 | 160 | 1 |
| 5 | 267 | 1 |
| 5 | 310 | 1 |
| 5 | 361 | 1 |
| 5 | 406 | 1 |
| 5 | 445 | 1 |
| 5 | 495 | 1 |
| 5 | 546 | 1 |
| 5 | 597 | 1 |
| 5 | 673 | 1 |
| 5 | 739 | 1 |
| 5 | 848 | 1 |
| | | 1 |
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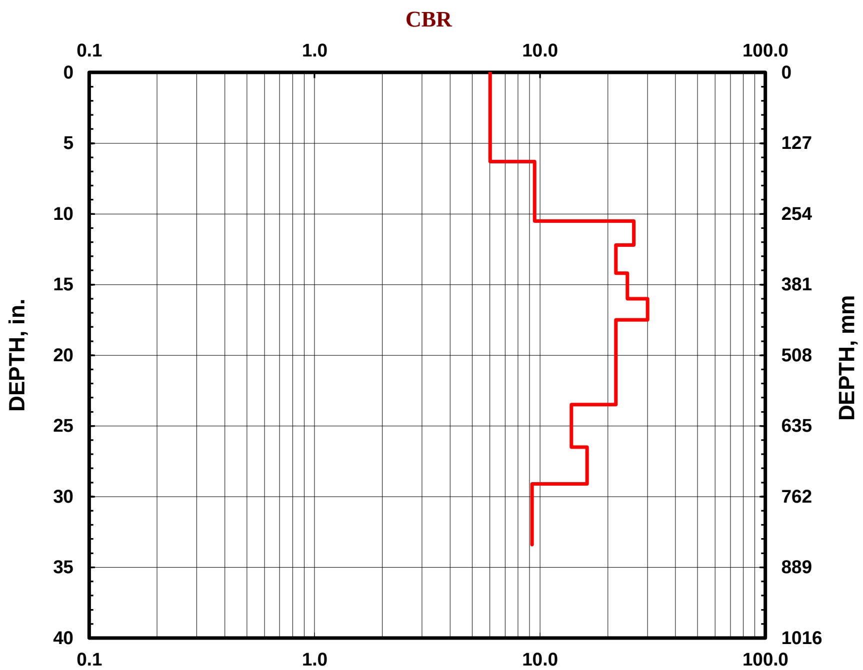
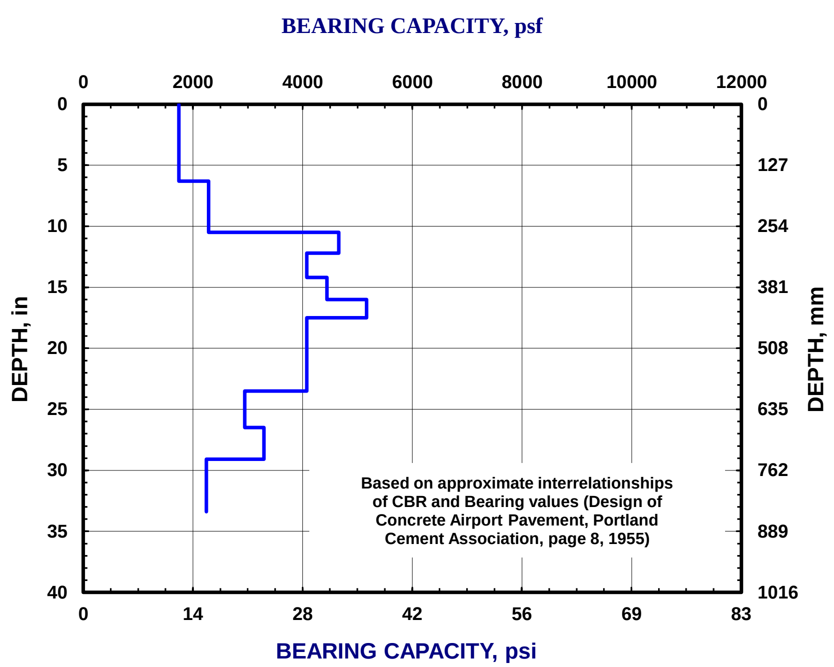


| CLIENT | Job #: | BORING # | SHEET |
|---|---|---|---|
| UNCW | 22:27497 | HA-1 | 1 OF 1 |

PROJECT NAME: UNCW Parking Deck - West Lot

ARCHITECT-ENGINEER:

SITE LOCATION: Wilmington, North Carolina

NORTHING: | EASTING: | STATION:

—○— CALIBRATED PENETROMETER TONS/FT$^2$

ROCK QUALITY DESIGNATION & RECOVERY: RQD% – – – REC% ———

PLASTIC LIMIT% × — WATER CONTENT% ● — LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

| DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | RECOVERY (IN) | DESCRIPTION OF MATERIAL (ENGLISH UNITS; BOTTOM OF CASING; LOSS OF CIRCULATION; SURFACE ELEVATION) | WATER LEVELS | ELEVATION (FT) | BLOWS/6" | WATER CONTENT % |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | (SM) SILTY FINE TO MEDIUM SAND, gray/ brown, moist, trace gravel | | | | |
| 0.5 | | | | | (SP) FINE TO MEDIUM SAND, light gray/tan, moist | | | | 8.6 |
| 4 | | | | | END OF BORING @ 4.0' | | | | |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

| | | |
|---|---|---|
| WL DRY WS☐ WD☐ | BORING STARTED 1/31/2019 | CAVE IN DEPTH |
| WL(SHW) WL(ACR) | BORING COMPLETED 1/31/2019 | HAMMER TYPE |
| WL | RIG FOREMAN | DRILLING METHOD Hand Auger |

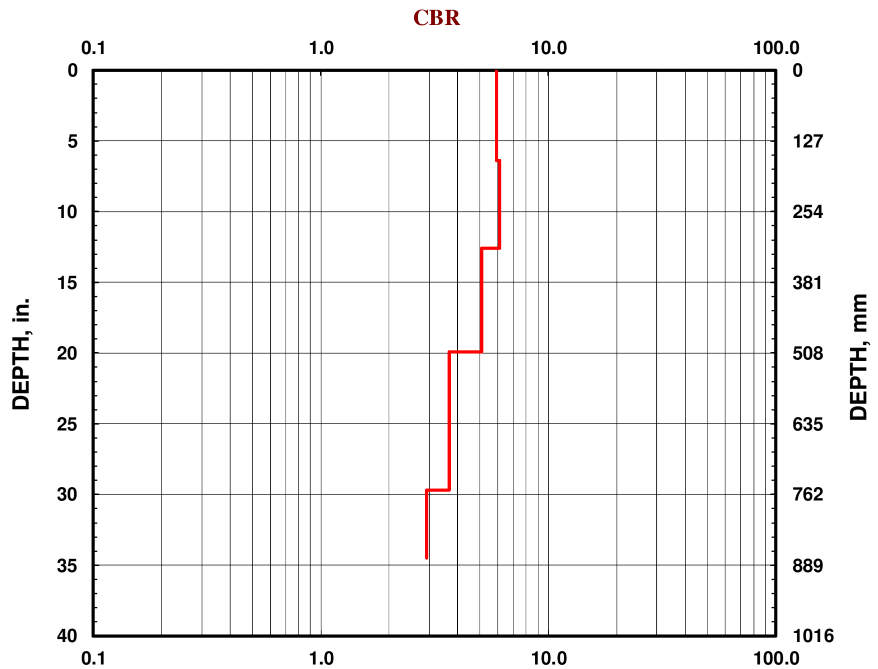
# DCP TEST DATA

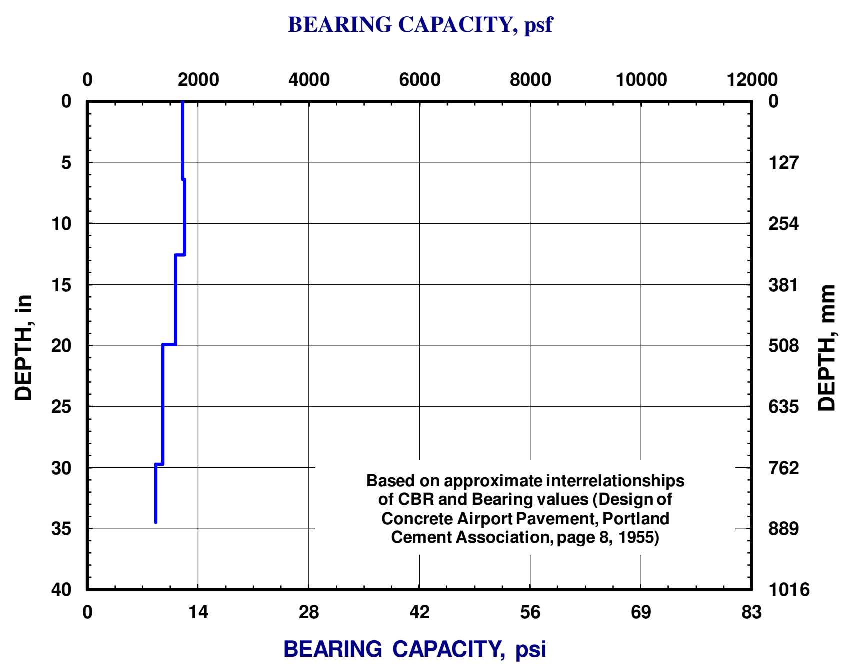
**Project:** UNCW Parking Deck - West Lot  **Date:** 31-Jan-19
**Location:** HA-1  **Soil Type(s):** SAND (SM, SP)

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Cumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 5 | 163 | 1 |
| 5 | 320 | 1 |
| 5 | 505 | 1 |
| 5 | 754 | 1 |
| 2 | 876 | 1 |



| CLIENT | Job #: | BORING # | SHEET |
|---|---|---|---|
| UNCW | 22:27497 | HA-2 | 1 OF 1 |

| PROJECT NAME | ARCHITECT-ENGINEER |
|---|---|
| UNCW Parking Deck - West Lot | |

| SITE LOCATION |
|---|
| Wilmington, North Carolina |

| NORTHING | EASTING | STATION |
|---|---|---|
| | | |

─◯─ CALIBRATED PENETROMETER TONS/FT$^2$

ROCK QUALITY DESIGNATION & RECOVERY
RQD% – — –  REC% ———

PLASTIC LIMIT% × ——— WATER CONTENT% ● ——— △ LIQUID LIMIT%

⊗ STANDARD PENETRATION BLOWS/FT

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION

| DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | RECOVERY (IN) | DESCRIPTION OF MATERIAL | WATER LEVELS | ELEVATION (FT) | BLOWS/6" | WATER CONTENT% |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | TOPSOIL/ROOTMAT [2"] | | | | |
| | | | | | (SM) SILTY FINE TO MEDIUM SAND, dark gray, moist, trace roots | | | | |
| 2 | | | | | (SP) FINE TO MEDIUM SAND, light gray, moist | | | | 4.6 |
| 4 | | | | | END OF BORING @ 4.0' | | | | |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

| | | |
|---|---|---|
| WL DRY WS☐ WD☐ | BORING STARTED 1/31/2019 | CAVE IN DEPTH |
| WL(SHW) WL(ACR) | BORING COMPLETED 1/31/2019 | HAMMER TYPE |
| WL | RIG FOREMAN | DRILLING METHOD Hand Auger |

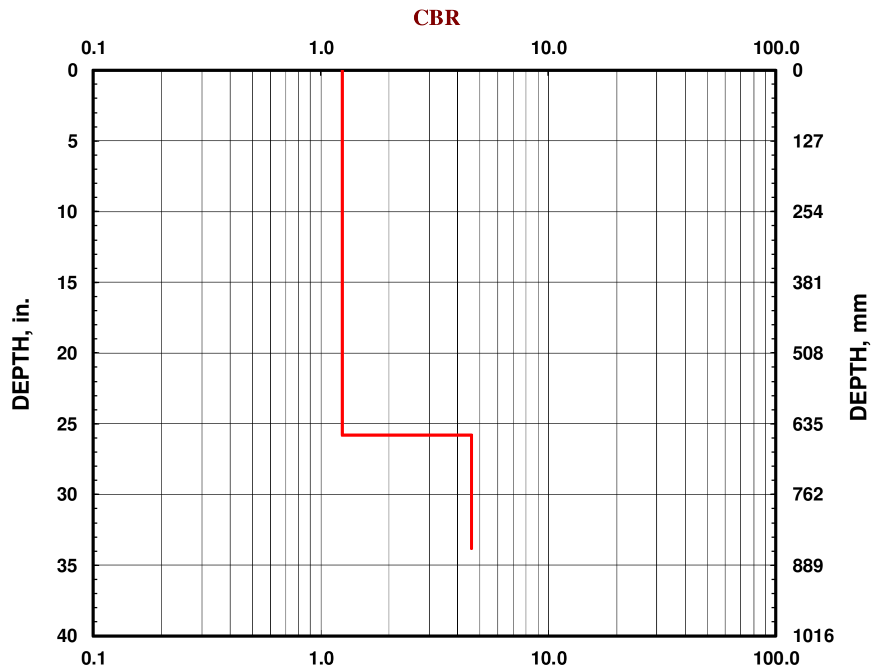
# DCP TEST DATA

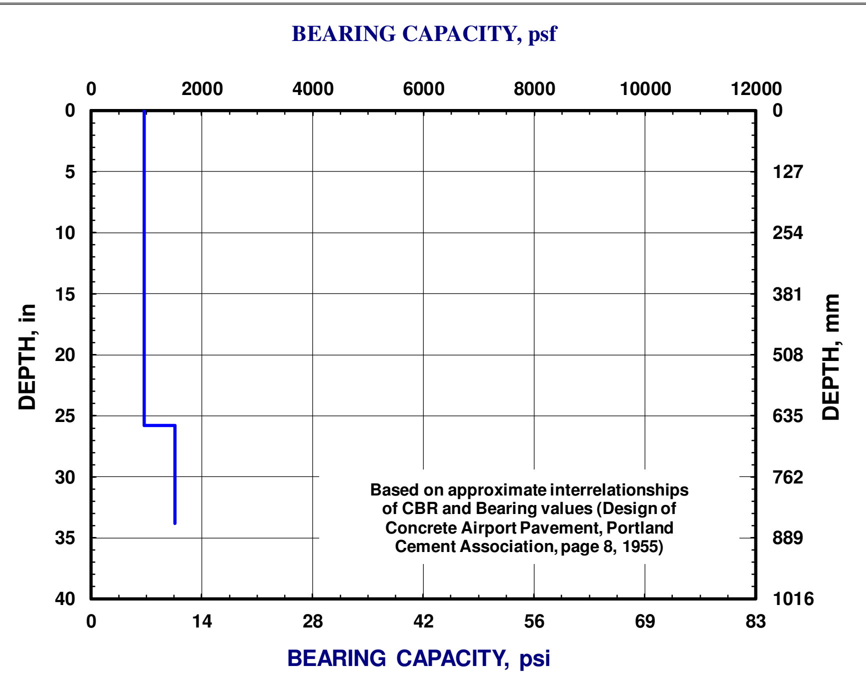
Project: UNCW Parking Deck - West Lot
Date: 31-Jan-19
Location: HA-2
Soil Type(s): SAND (SM, SP)

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Cumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 5 | 655 | 1 |
| 5 | 859 | 1 |



| CLIENT | Job #: | BORING # | SHEET |
|---|---|---|---|
| UNCW | 22:27497 | HA-3 | 1 OF 1 |

PROJECT NAME: UNCW Parking Deck - West Lot

ARCHITECT-ENGINEER:

SITE LOCATION: Wilmington, North Carolina

NORTHING: | EASTING: | STATION:

Legend: CALIBRATED PENETROMETER TONS/FT$^2$; ROCK QUALITY DESIGNATION & RECOVERY (RQD% – – –, REC% ———); PLASTIC LIMIT%, WATER CONTENT%, LIQUID LIMIT%; STANDARD PENETRATION BLOWS/FT

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION

| DEPTH (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DIST. (IN) | RECOVERY (IN) | DESCRIPTION OF MATERIAL | WATER LEVELS | ELEVATION (FT) | BLOWS/6" | WATER CONTENT% |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | TOPSOIL/ROOTMAT [6"] | | | | |
| | | | | | (SM) SILTY FINE TO MEDIUM SAND, dark gravel brown/dark gray, moist, trace gravel from 1' to 2' | | | | |
| 2 | | | | | (SP) FINE TO MEDIUM SAND, light gray, moist | | | | 6.9 |
| 3 | | | | | (SM) SILTY FINE TO MEDIUM SAND, dark brown, wet | | | | |
| 4 | | | | | END OF BORING @ 4.0' | | | | |

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

| | | |
|---|---|---|
| WL DRY WS☐ WD☐ | BORING STARTED 1/31/2019 | CAVE IN DEPTH |
| WL(SHW) WL(ACR) | BORING COMPLETED 1/31/2019 | HAMMER TYPE |
| WL | RIG FOREMAN | DRILLING METHOD Hand Auger |

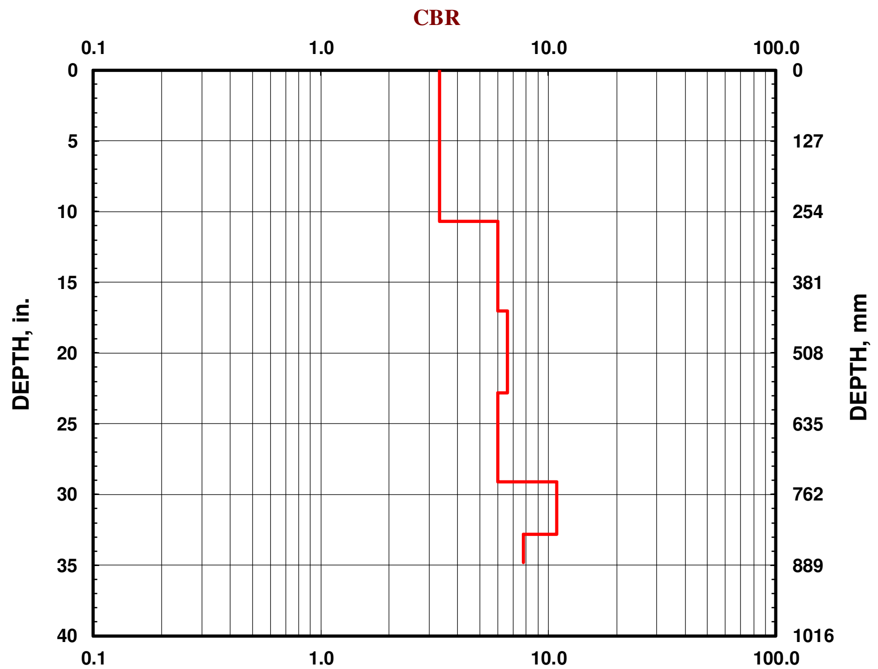
# DCP TEST DATA

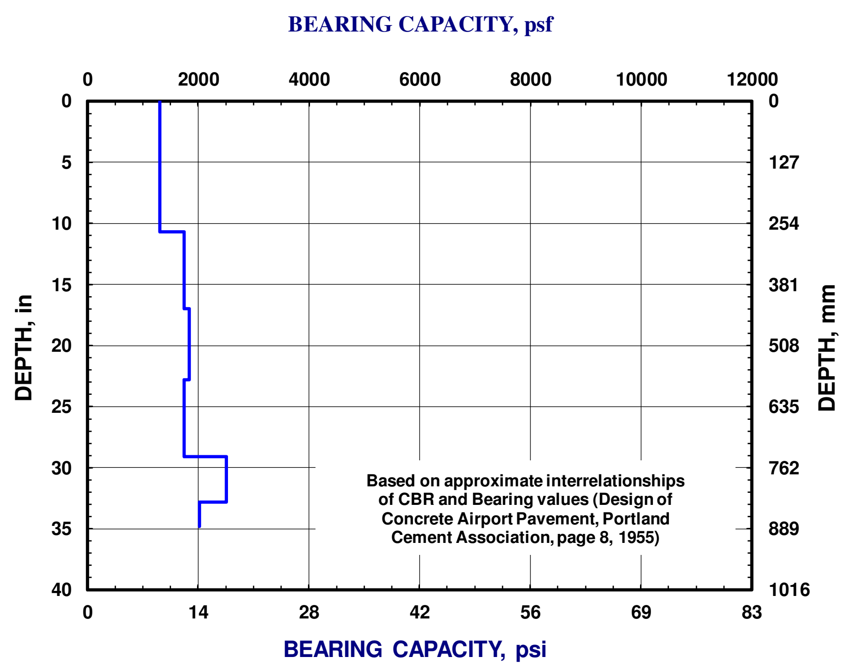
**Project:** UNCW Parking Deck - West Lot
**Date:** 31-Jan-19
**Location:** HA-3
**Soil Type(s):** SAND (SM, SP)

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

| No. of Blows | Cumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 0 | 0 | 1 |
| 5 | 272 | 1 |
| 5 | 432 | 1 |
| 5 | 579 | 1 |
| 5 | 739 | 1 |
| 5 | 833 | 1 |
| 2 | 884 | 1 |



## APPENDIX C – Laboratory Testing

Laboratory Test Results Summary
Natural Moisture Content Results
Moisture-Density Relationship Curves

# Laboratory Testing Summary



| Sample Source | Sample Number | Depth (feet) | MC[1] (%) | Soil Type[2] | Atterberg Limits[3] LL | PL | PI | Percent Passing No. 200 Sieve[4] | Moisture - Density (Corr.)[5] Maximum Density (pcf) | Optimum Moisture (%) | CBR Value[6] | Other |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **HA-1** | S-1 | 1.5-2.0 | 8.6 | SP | | | | | | | | |
| **HA-2** | S-2 | 2.0-2.5 | 4.6 | SP | | | | | | | | |
| **HA-3** | S-3 | 2.5-3.0 | 6.9 | SP | | | | | | | | |
| **Bulk-1** | | 1.0-3.0 | | SP | | | | | 108 | 12.8 | | |

**Notes:** 1. ASTM D 2216, 2. ASTM D 2487, 3. ASTM D 4318, 4. ASTM D 1140, 5. See test reports for test method, 6. See test reports for test method

**Definitions:** MC: Moisture Content, Soil Type: AASHTO, LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, CBR: California Bearing Ratio, OC: Organic Content

| | |
|---|---|
| **Project No.** | **22.27313** |
| **Project Name:** | **UNCW Parking Deck** |
| **PM:** | **Annemarie Crumrine** |
| **PE:** | **Winslow Goins** |
| **Printed On:** | **2/4/19** |

**ECS Southeast, LLP**
6714 Netherlands Drive
Wilmington, NC 28405
Phone: (910) 686-9114

**ECS Southeast, LLP**
**6714 Netherlands Drive**
**Wilmington, NC 28405**
**910-686-9114**

# Moisture Content Worksheet (ASTM D2216)

**Project Name** UNCW Parking Deck - West Lot **Project #** 27497 **Date** 2/1/2019

| Sample ID | Depth (ft.) | Wet Soil + Tare Weight | Dry Soil + Tare Weight | Tare Weight | Water | Dry Soil | Moisture Content (%) |
|---|---|---|---|---|---|---|---|
| HA-1, S-1 | 1.5-2.0 | 134.48 | 124.46 | 8.37 | 10.02 | 116.09 | 8.6 |
| HA-2, S-2 | 2.0-2.5 | 193.05 | 184.89 | 8.64 | 8.16 | 176.25 | 4.6 |
| HA-3, S-3 | 2.5-3.0 | 220.28 | 206.58 | 8.40 | 13.70 | 198.18 | 6.9 |

# COMPACTION TEST REPORT

**Curve No.: S-1**

**Project No.:** 27497 **Date:** 2/1/19

**Project:** UNCW Parking Deck - West Lot

**Client:** McKim & Creed

**Location:** Onsite

**Sample Number:** S-1

**Remarks:**

**MATERIAL DESCRIPTION**

**Description:** Dark Brown, Silty, Fine to Medium Sand, Trace Clay

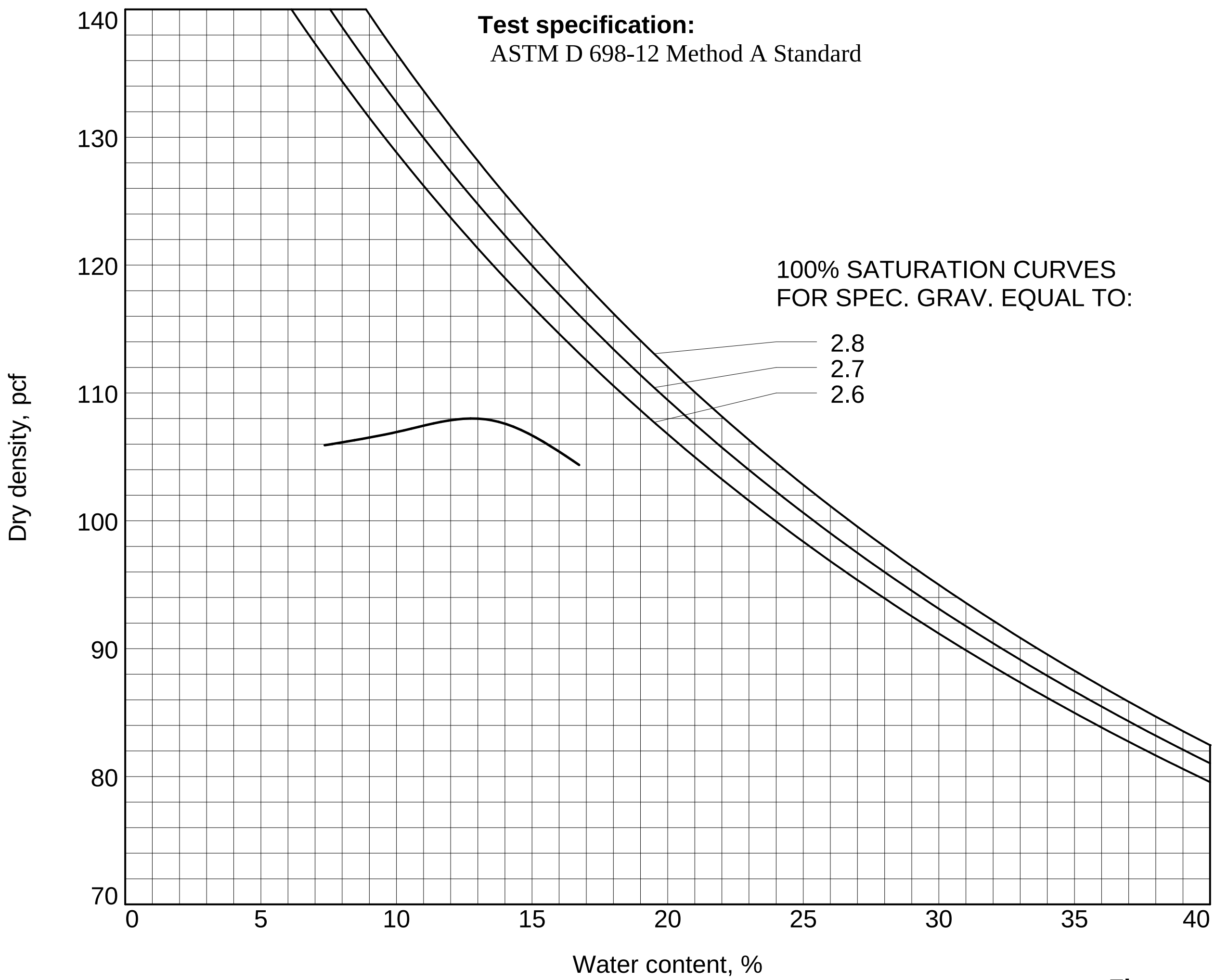
**Classifications -** **USCS:** **AASHTO:**

**Nat. Moist. =** **Sp.G. =** 2.60

**Liquid Limit =** **Plasticity Index =**

**% < No.200 =**

| TEST RESULTS |
|---|
| Maximum dry density = 108.0 pcf |
| Optimum moisture = 12.8 % |




**Figure**

**ECS Southeast, LLP**

**Tested By:** EG **Checked By:** KEL

## APPENDIX D – Seasonal High Water Table and Infiltration Testing Report

Seasonal High Water Table and Infiltration Testing Results

February 4, 2019

Mr. Richard Collier, P.E.
McKim & Creed
243 North Front Street
Wilmington, North Carolina 28401

Reference: Report of Seasonal High Water Table Estimation and Infiltration Testing
UNCW Parking Deck - West
Wilmington, New Hanover County, North Carolina
ECS Project No. 22.27497

Dear Mr. Collier:

ECS Southeast, LLP (ECS) recently conducted a seasonal high water table (SHWT) estimation and infiltration testing within the stormwater control measure (SCM) area(s) at Parking Lot L on the campus of UNCW in Wilmington, New Hanover County, North Carolina. This letter, with attachments, is the report of our testing.

**Field Testing**

On January 31, 2019, ECS conducted an exploration of the subsurface soil and groundwater conditions, in accordance with the NCDEQ Stormwater Design Manual section A-2, at one requested location shown on the attached Boring Location Plan (Figure 1). ECS used GPS equipment in order to determine the boring location. The purpose of this exploration was to obtain subsurface information of the in situ soils for the SCM area(s). ECS explored the subsurface soil and groundwater conditions by advancing one hand auger boring into the existing ground surface at each of the requested boring location. ECS visually classified the subsurface soils and obtained representative samples of each soil type encountered. ECS also recorded the SHWT and groundwater elevation observed at the time of the hand auger boring. The attached Infiltration Testing Form provides a summary of the subsurface conditions encountered at the hand auger boring locations.

The SHWT and groundwater elevation was estimated at the boring location below the existing grade elevation. A summary of the findings are as follows:

| Location | SHWT | Groundwater |
|---|---|---|
| I-1 | 36 inches | 55 inches |

ECS has conducted two infiltration tests utilizing a compact constant head permeameter near the hand auger borings in order to estimate the infiltration rate for the subsurface soils. Infiltration tests are typically conducted at two feet above the SHWT or in the most restrictive soil horizon. Tests in clayey conditions are conducted for durations of up to 30 minutes. If a more precise hydraulic conductivity value is desired for these locations, then ECS recommends collecting samples by advancing Shelby tubes and performing laboratory permeability testing.

**Field Test Results**

Below is a summary of the infiltration test results:

| Location | Description | Depth | Inches/ hour |
|---|---|---|---|
| I-1 | Dark gray silty SAND | 16 inches | 0.21 |

Infiltration rates and SHWT may vary within the proposed site due to changes in elevation and subsurface conditions.

**Closure**

ECS's analysis of the site has been based on our understanding of the site, the project information provided to us, and the data obtained during our exploration. If the project information provided to us is changed, please contact us so that our recommendations can be reviewed and appropriate revisions provided, if necessary. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our review, analysis and revision of our recommendations, if necessary. The assessment of site environmental conditions for the presence of pollutants in the soil and groundwater of the site is beyond the scope of this geotechnical exploration.

ECS appreciates the opportunity to provide our services to you on this project. If you have any questions concerning this report or this project, please contact us.

Respectfully,

**ECS SOUTHEAST, LLP**

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Attachments: Figure 1 - Boring Location Plan
Infiltration Testing Form


**APPROXIMATE BORING LOCATIONS**

**SCALE SHOWN ABOVE**

N
W E
S

**UNCW Parking Deck - West**
**Wilmington, New Hanover County, North Carolina**

**ECS Project # 22.27497**
**January 31, 2019**
**KBW**

**Figure 1– Boring Location Plan**

**Provided by: Google Earth**

Infiltration Testing Form
UNCW Parking Deck - West
Shallotte, Brunswick County, North Carolina
ECS Project No. 22.27497
January 31, 2019

| Location | Depth | USCS | Soil Description |
|---|---|---|---|
| I-1 | 0-6” | SM | Gray silty SAND |
| | 6”-16” | SP | Tan fine SAND |
| | 16”-20” | SM | Dark gray silty SAND |
| | 20”-36” | SP | Gray fine SAND |
| | 36”-60” | SM | Black silty SAND (hardpan) |

Seasonal High Water Table was estimated to be at 36 inches below the existing grade elevation.
Groundwater was encountered at 55 inches below the existing grade elevation.
Test was conducted at 16 inches below existing grade elevation
Infiltration Rate: 0.21 inches per hour

## APPENDIX E – Supplemental Report Documents

GBA Document

# Important Information about This

# Geotechnical-Engineering Report

**Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.**

**While you cannot eliminate all such risks, you can manage them. The following information is provided to help.**

**The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.**

## Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

## Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

## You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

## This Report May Not Be Reliable

*Do not rely on this report* if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

## Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

## This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation*.

## This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

## Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

## Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

## Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

## Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***


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