April 6, 2009

Ms. Nancy Jenkins, AIA, LEED AP
Lindbergh & Associates, LLC.
2170 Ashley Phosphate Road
Charleston, SC 29406
JenkinNL@lindbergh-assoc.com

Geotechnical Investigation
Ralph H. Johnson VA Medical Center - Addition
Charleston, South Carolina
WPC Project WPC1209.00081

Dear Ms. Jenkins:

WPC has completed the geotechnical exploration for the above referenced project to be constructed on the grounds of the Ralph H. Johnson VA Medical Center in Charleston, South Carolina. The purpose of this exploration was to determine the subsurface conditions at the site with respect to the proposed construction. The following paragraphs present our understanding of the proposed project, describe our exploratory procedures, discuss the subsurface conditions encountered, and present our recommendations for site preparation and foundation support. The provided recommendations are based upon our understanding of the proposed construction, our test data, and our experience with similar projects and conditions.

EXECUTIVE SUMMARY

- The subsurface lithology of the proposed addition is comprised of approximately 2 inches of topsoil followed by medium dense to dense sand to a depth of approximately 19 feet. Underlying the sand is a very soft marine silt that extends to an average depth of approximately 55½ feet. The soft marine silt transitions into a medium dense to dense sand that extends to an average depth of approximately 65 feet. Underneath the sand is a firm to stiff sandy silt that extends to the termination of the deepest boring/sounding at a depth of 90 feet.

- The site is classified as Seismic Design Category E/F based on the procedures outlined in the IBC 2006/ASCE 7-05 for the presence of approximately 35 feet of very soft soil and potentially liquefiable soils. Liquefaction induced settlements associated with a design seismic event may approach 2½ inches. A Site Specific Seismic Evaluation (SSSE) was conducted for this project.

- Based on the potential for excessive primary and secondary consolidation induced settlements, the proposed addition should be supported on a deep foundation system such as drilled shafts or driven piles bearing within the Cooper Marl Formation.

- If significant volumes of fill are expected to establish nominal construction grade, downdrag on foundation elements of both the proposed and the existing structure may need to be addressed.

PROJECT INFORMATION

Overview

The proposed addition to the Ralph H. Johnson VA Medical Center is located at 109 Bee Street, Charleston, South Carolina, as seen in Figure 1. The proposed addition is expected to have a footprint of approximately 6,000 square feet and will initially be a one to two story structure. However, it is our understanding that the structure is to have a foundation system (in addition to other structural components) that can accommodate a completed structure of up to five (5) stories. At this time, building loads are unknown, but based on the type of structure and anticipated final building height, we assume column loads are likely to approach **500 kips** or more with floor loads ranging up to **250 pounds per square foot (psf)**. We anticipate the addition will be designed in general accordance with International Building Code (IBC) 2006 Edition. Based on visual observation, fill requirements for this project are expected to be minimal.

Currently, the proposed location of the addition is used as a small garden area consisting of a manicured lawn, concrete path, and several trees and shrubs.

If building or fill loads significantly differ from what was assumed above, WPC should be contacted to review and revise these recommendations presented herein as necessary.

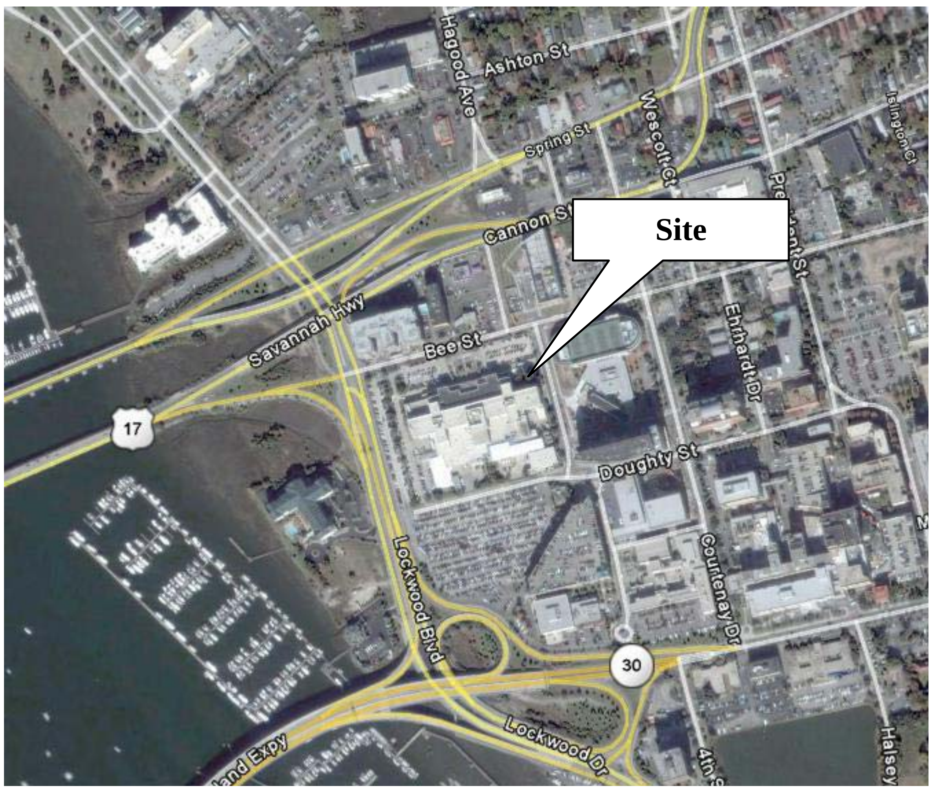


Figure 1. Approximate Location of Project Site

REGIONAL GEOLOGY

South Carolina is composed of two broad physiographic regions, the Atlantic Coastal Plain and the Piedmont Provinces. "The wide Coastal Plain belt generally consists of sedimentary deposits of Late Cretaceous to Holocene in age, extending from New Jersey to Texas. These sedimentary rocks, deposited mostly in a marine environment, were later uplifted and now tilt seaward. Coastal Plain deposits overlap the older, more distorted, Paleozoic and Precambrian rocks immediately to the north and west"[1] of the Piedmont Uplands. The dividing line between the two geomorphic provinces is generally considered the Fall Line, which indicates a change from a thin (relatively speaking) surficial veneer of sediments overlying the crystalline bedrock of the Piedmont Province to deeper unconsolidated thick layers of sediments that lie within the Coastal Plains. The basement rock, which is visible on the ground surface as outcroppings in the Piedmont, gradually drops down in a southeasterly direction at an approximate rate of 25 feet per mile. The sediments of the Coastal plains range in thickness from a feather edge at the Fall Line to depths approaching 3,000 feet before encountering basement rock near the coast. Figure 2 presents the site location relative to the generalized South Carolina geology.

Soils of the Lower Coastal Plain typically consist of interbedded layers of silts, sands, and low permeability clays deposited in a marine environment prior to being uplifted during a geologic event. The coarse grained soils typically classified as SP to SC whereas the fine-grained soils generally classify as CL to CH in the Unified Soil Classification System (USCS).

[1] A Tapestry of Time and Terrain: A Union of Two Maps, Geology and Topography. Retrieved on August March 18, 2009 from the World Wide Web:http://tapestry.usgs.gov/features/13coastalplain.html

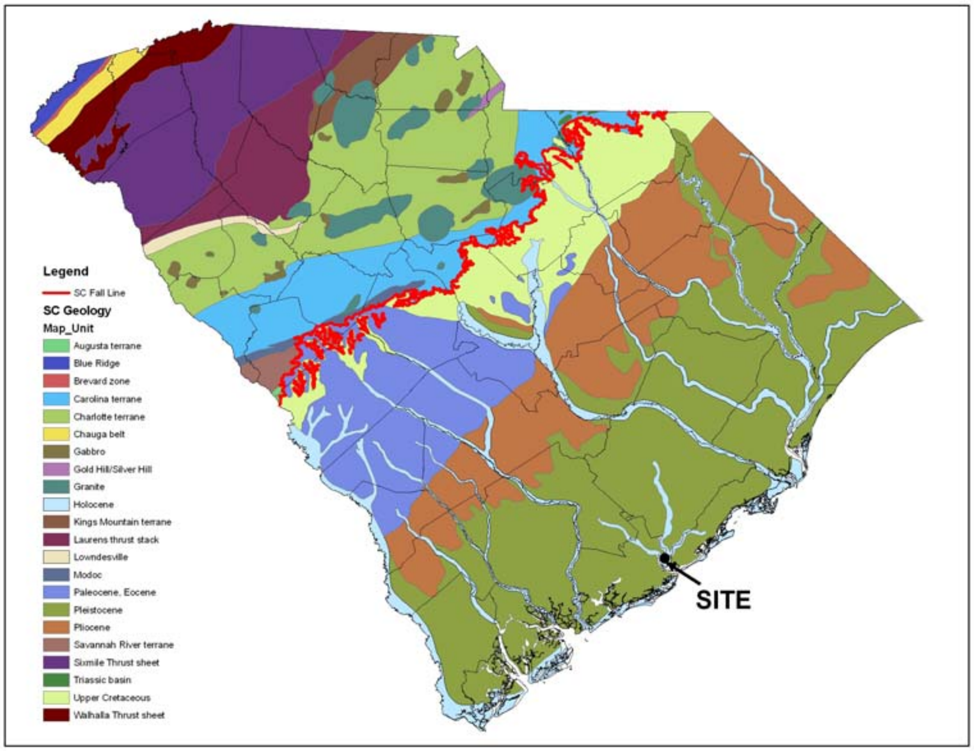


Figure 2. Generalized Geologic Map of South Carolina

EXPLORATION PROCEDURES

Overview

Our field investigation within the study area consisted of one (1) Piezocone Penetration Test (CPTu) (ASTM D5778) and one (1) Soil Test Boring (STB) which were conducted within the footprint of the proposed addition. Within the CPTu sounding, simultaneous seismic testing was performed to collect shear wave velocities utilized for seismic site classification. The CPTu sounding was terminated at a depth of approximately 72 feet below the ground surface whereas the STB terminated at a depth of approximately 70 feet.

Seismic Piezocone Penetration Test (SCPTu)

The Piezocone Penetration Test (SCPTu) hydraulically pushes an instrumented cone through the soil while continuous readings are recorded to a portable computer. The instrumented cone has a cross-sectional area of 10 square centimeters (cm^2) with a 60° conical tip. The cone is advanced through the ground at a constant rate of 2 centimeters per second (2 cm/sec). No soil samples are gathered through this subsurface investigation technique. However, insitu measurements of tip and side resistance and porewater pressure are taken every 2 centimeters (approximately 1 inch). Porewater pressure measurements are taken directly behind the tip, while a load cell located above the cone tip takes side friction measurements. In addition, soil shear wave measurements were conducted at 1 meter (3.3 ft) depth increments. The CPTu test was conducted in accordance with ASTM D5778 "*Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils*".

The CPTu log in the report Appendix graphically illustrates the relative strength of the soils encountered and provides an approximate soil stratigraphy. Stratification lines on the CPTu log represent approximate boundaries between soil types based on behavioral characteristics. Soil behavior is based on currently accepted correlations between the tip, side, and porewater pressure measurements. A detailed explanation of these correlations can be found in the Appendix.

Soil Test Borings

The Soil Test Boring (STB) was completed using mud-rotary techniques in accordance with ASTM D 5783, "*Standard Guide for Use of Direct Rotary Drilling with Water-Based Drilling Fluid for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices*". Samples were collected using a standard Split Spoon Sampler according to the methods outlined in ASTM D1586, "*Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*". Split spoon samples were collected continuously in the upper 10 feet and then at five (5) foot intervals thereafter.

The STB record graphically illustrates Standard Penetration Test (SPT) values, groundwater levels, soil descriptions, and classification of the subsurface conditions based upon visual examination of the split-spoon samples by a geotechnical engineer using the procedures outlined in ASTM D2487 *"Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)"*. Stratification lines on these records represent approximate boundaries between soil types; however, the actual transition may be gradual. Details of the subsurface conditions encountered by the STB are included in the Soil Test Boring Log located in the Appendix of this report. SPT N-values are used in all forms of geotechnical analysis and design. N-values are used to determine soil properties, such as relative density and friction angle, through well-documented empirical correlations. N-values are also used in the design of shallow and deep foundations, evaluation of liquefaction potential, settlement analysis, and other geotechnical design parameters.

Hand Auger Boring (HAB)

Hand Auger Boring (HAB) allow for physical sampling of the subgrade soils for visual classification, laboratory testing, and site grading purposes. The HAB log is presented in the report Appendix.

Laboratory Testing

Laboratory soil testing consisted of natural moisture content (ASTM D2216 "*Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"*), grain size analysis (ASTM D422/SC T-34, "*Standard Test Method for Particle-Size Analysis of Soils*"), and Atterberg Limits testing (ASTM D4318, "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*") on selected samples.

LABORATORY TEST RESULTS

Soil Properties and Classification

Index testing consisting of sieve analysis, Atterberg Limits, and moisture content were performed on select samples collected from the project site. The total fines content (i.e. the amount passing the #200 sieve) for the sands tested was approximately 14.0%. The plasticity of the fine grained soils range from low to high plasticity silt. The laboratory test results are summarized in the following table with the individual test results presented in the report Appendix.

Table 1. Soil Properties and Classification

Sample Source	Sample Depth (ft)	USCS[1] (classification)	Passing #200 (%)	Moisture Content (%)	PI (%)	LL (%)
B-1	4 to 6	SM	14.0	14.4		
B-1	18.5 to 20	MH		102.6	29	62
B-1	38.5 to 40	MH[2]		102.1		
B-1	58.5 to 60	MH[2]		23.1		
B-1	78.5 to 80	ML		58.4	6	42

NOTES:

1. United Soil Classification System designation
2. Classification based in part on visual observation

GEOTECHNICAL FINDINGS

Subsurface Conditions

The subsurface lithology of the proposed addition comprised of approximately 2 inches of topsoil followed by medium dense to dense sand to a depth of approximately 19 feet. Underlying the sand is a very soft marine silt that extends to an average depth of approximately 55½ feet. The soft marine silt transitions into a medium dense to dense sand that extends to an average depth of approximately 65 feet below the existing ground surface. Underneath the sand is a firm to stiff sandy silt that extends to the termination of the deepest boring/sounding at a depth of 90 feet. This soil stratum is locally referred to as the Cooper Marl Formation. The Cooper Marl Formation underlying the Charleston area is typically 100 to 200 feet thick. This formation is often used as a bearing stratum for a deep foundation system and the basis for our seismic analysis.

Groundwater

At the time of our exploration, the water table was encountered at a depth of approximately 8 feet below the ground surface. The groundwater depth was determined from calculating the hydrostatic line (height of water below the ground surface) on the penetration porewater pressure (U) graph in the Piezocone Penetration Logs. No groundwater reading was collected within the STB. Rainfall events, drainage constraints, and seasonal weather patterns can vary with time and influence the level of the groundwater table.

Seismic Evaluation

Due to the high seismicity of the Coastal South Carolina area, WPC has performed a liquefaction potential analysis for the site. Ground shaking at the foundation of structures and liquefaction of the soil under the foundation are the principle seismic hazards to be considered in design of earthquake-resistant structures. Liquefaction occurs when a rapid buildup in water pressure, caused by the ground motion, pushes sand particles apart, resulting in a loss of strength and later densification as the water pressure dissipates. This loss of strength can cause bearing capacity failure while the densification can cause excessive settlement. Potential earthquake damage can be mitigated by structural and/or geotechnical measures or procedures common to earthquake resistant design.

According to the International Building Code year 2006 edition (IBC 2006), structures are required to be designed to a design earthquake from a 50 year exposure period with a 2% Probability of Exceedance (PE) (i.e. a 2475 year design earthquake). The 2% PE in 50 year design earthquake has a Moment Magnitude (M_w) of 7.3 and a Peak Ground Acceleration (PGA) of **0.29g**, as determined from data provided by the IBC 2006 Code. The IBC 2006 Seismic design code is based on the 2004 National Earthquake Hazards Reduction Program (NEHRP) *Recommended Provisions for Seismic Regulations for New Building and Other Structures, Part 1 and 2- Commentary* (FEMA 450) and the 2004 USGS National Seismic Hazard Mapping Project. Based on the presence of ±35 feet of continuous, very soft soils (i.e. pluff mud) the project site classifies as a Site Class E. In addition to the soft soils, sands located below the ground surface to a depth of approximately 20 feet of the ground surface have the potential to liquefy during a design event the project site classifies as a Site Class F.

Based on this site classification and soil profile, and the assumption the building will likely have a fundamental period greater than 0.5 seconds, we conducted a Site Specific Seismic Evaluation (SSSE) for this project. Site specific seismic design parameters are as follows:

$$\mathbf{S_{DS} = 0.72,\ S_{D1} = 0.49,\ S_{MS} = 1.02,\ \text{and}\ S_{M1} = 0.74}$$

Figure 3 presents the Design Response Spectrum for this site based on procedures outlined in IBC 2006 and ASCE 7-05.

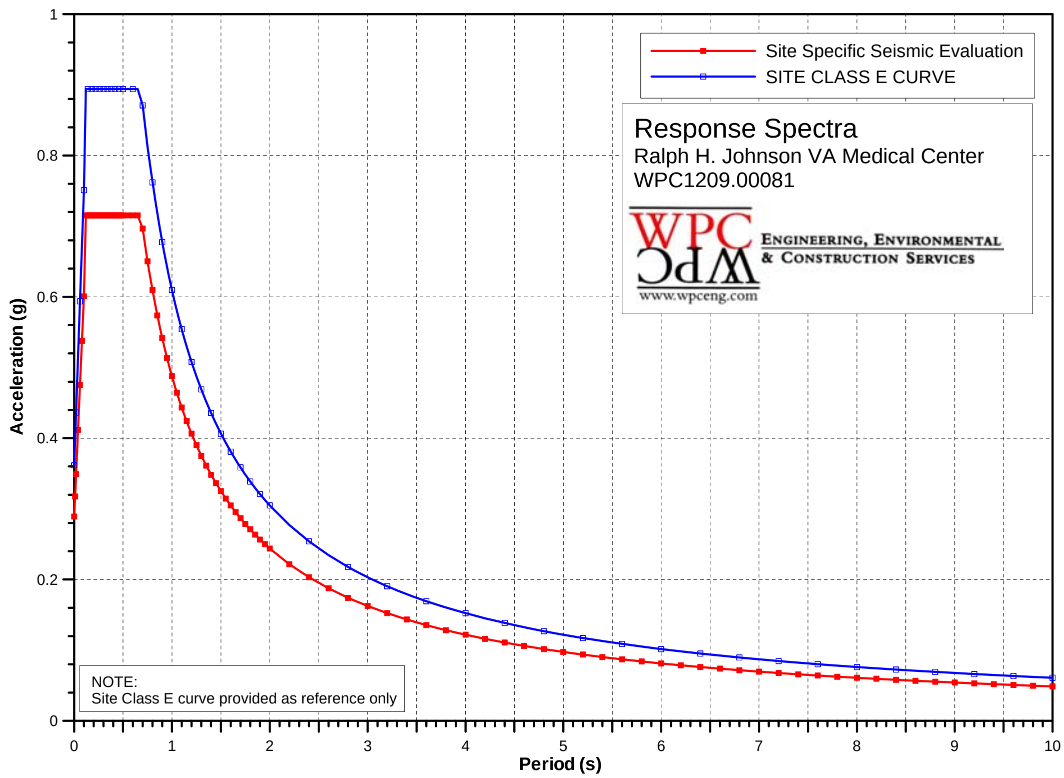


Figure 3. Site Specific Design Response Spectrum.

While the amount of the settlement is dependent on the magnitude and distance from the seismic event, we estimate that settlements from the design earthquake may range up to 2½ inches within the proposed addition footprint. Differential settlement may range up to 50 to 75 percent of the total settlement depending on depth and amount of liquefaction, and location relative to a seismic event epicenter.

RECOMMENDATIONS

Overview

Given the nature of the proposed addition (i.e. hospital), the expected loading, and the soft ground conditions, we recommend the addition be supported on a deep foundation system bearing within the Cooper Marl Formation. We have examined several common deep foundation options that have been used successfully in the Charleston area. We recommend that either drilled shafts or driven piles be utilized to support the proposed addition. If a driven pile foundation option is selected, we recommend the use of pre-stressed concrete (PSC) piles. PSC piles are commonly used throughout the Charleston area. Low displacement piles such as steel pipe piles may also be used to support the planned construction. Alternatively, Augered Cast in Place (ACIP) piles may be considered for this project. Discussions on the advantages and disadvantages of various deep foundation options discussed in the report text are presented in the report Appendix.

Our deep foundation analysis included the effects of downdrag resulting from the self consolidation of ±35 feet of very soft marine silt on the axial compressive capacity of the various recommended foundation elements. Additionally, our analysis included capacity provided by the sand that directly overlies the Cooper Marl Formation. No allowance to axial capacity was given to the upper sand (i.e. soils located within 19 feet of the existing ground surface).

Additionally, since the existing hospital will be in operation during construction, and the proposed addition will be located in close proximity to rooms occupied with patients, there is some concern in regards to the both the noise and vibration levels generated during construction. Although it will be impossible to completely eliminate noise and vibrations associated with construction activities, we can provide several options that can limit their impacts.

Driven Pile Foundations

We have analyzed two driven pile options in various sizes to support the proposed structure: 12 inch and 14 inch square pre-stressed concrete (PSC) piles and steel pipe piles in 12 inch and 18 inch diameters. Figure 4 presents the allowable axial compressive capacities with depth for both pile types. A Factor of Safety (FS) of 2.25 was used to determine the allowable (i.e. design) capacities. Note, this factor of safety assume that a pile load test program utilizing a Pile Dynamic Analyzer (PDA) is conducted to verify/modify the pile design, otherwise, a larger factor of safety will be necessary and production pile lengths may vary resulting in addition expense. The designer should factor in final finished grades of the site when determining final pile lengths.

The PSC piles should conform to the guidelines specified in ACI 543R-74 Recommendations for Design, Manufacture, and Installation of Concrete Piles and PCI JR-382 Recommended Practice for Design, Manufacture, and Installation of Pre-Stressed Concrete Piling.

Pile Installation

Based on our experience with similar projects, air, diesel, or hydraulic hammers with rated energies of approximately **35 ft-kips to 65 ft-kips** should be appropriate for pile installation. Based on the subsurface profile (i.e. medium dense to dense sand overlying the Cooper Marl Formation), we recommend that the pile locations be pre-augered prior to installation to limit vibrations to within acceptable levels. Pre-augering should be advanced to the Cooper Marl Formation, estimated to be encountered at an average depth of approximately 65 feet below the existing ground surface. This pre-augering depth can be modified for the production piles by the geotechnical engineer based on the test pile program results. The outer diameter of the auger used should not exceed the outside diameter of the piles.

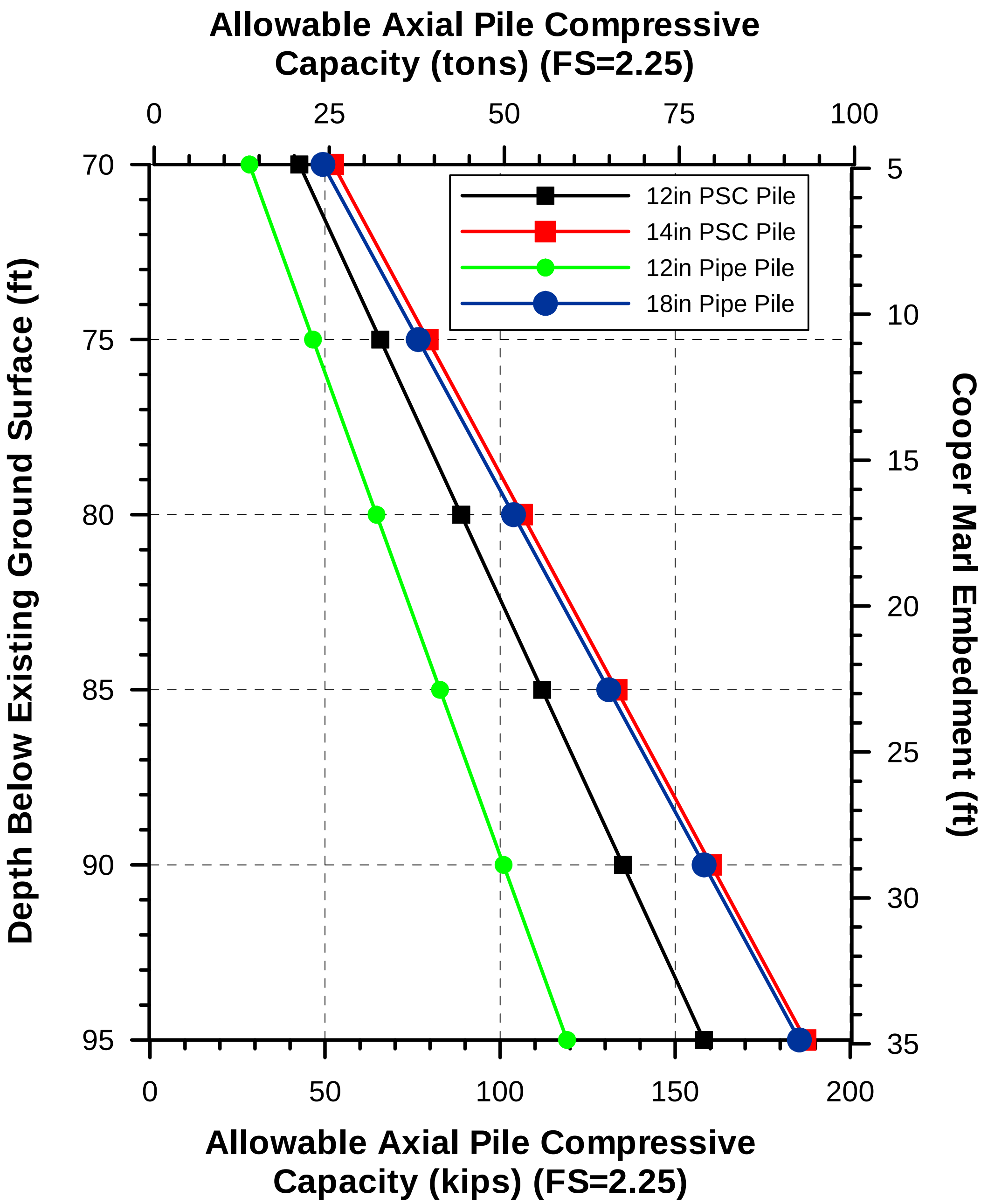


Figure 4. Allowable Axial Compressive Capacity (Driven Piles)

Upon selection of the pile size and the contractor's driving system, a wave equation analysis of piles (WEAP) of the hammer-pile-soil system should be conducted. The WEAP analysis will determine if the selected hammer has sufficient energy to install the selected pile size to the required depth, if the driving stresses (both compressive and tensile) during installation are within acceptable limits, and provide pile driving criteria. Hammer and/or pile sizes can be varied until an acceptable hammer-pile system is found. Upon request, WPC can provide assistance in evaluating the selected hammer and determining the pile driving criteria.

Driven Pile Test Program

We recommend that three (3) test piles be installed within the footprint of the proposed addition to determine final production pile lengths. Test piles may be installed at production locations. The geotechnical engineer should select the test pile locations in conjunction with the structural engineer. In addition, the geotechnical engineer or their representative should be present during the installation of the test piles. Hammer restrikes should be performed on each of the test piles a minimum of seven (7) days after installation to determine final axial capacity. This wait period will account for the time dependent pile capacity gain (i.e. pile "setup" or "freeze") characteristics of the Cooper Marl.

The piles should be dynamically monitored during installation and hammer restrikes in accordance with ASTM D4945, *"Standard Test Method for High-Strain Dynamic Testing of Piles"*. Test pile lengths should allow for a minimum of five (5) feet of the pile to extend above the ground surface after installation to facilitate gage attachment for the dynamic load tests.

Driven Pile Quality Control

An engineering technician, supervised by a professional engineer licensed in the state of South Carolina, should monitor and document the production pile installations. A pile driving record should be kept for each individual production pile. The individual pile driving records should have the following minimum information:

- Pile size
- Final pile embedment depth

- Pile tip and head elevation (if applicable)
- Pile installation date and time
- Pre-augering information
- Pile blow counts per one (1) foot interval
- Relevant Hammer and Cushion Information
- Hammer Stroke
- Installation notes (as required)

Drilled Shaft Capacity and Installation

Drilled shafts should be installed into the Cooper Marl Formation. Allowable axial compressive capacities of drilled shafts with 36 inch and 48-inch diameters are presented in Figure 5. A Factor of Safety (FS) of 2.0 was used to determine the allowable (i.e. design) capacities. Note, this factor of safety assume that a static load test is conducted as described in a subsequent section, otherwise, a larger factor of safety will be necessary and production shaft lengths may vary resulting in addition expense. Note, the capacity presented in the following chart is based on a straight shaft, larger capacities can be generated with the use of a belled shaft embedded within the Cooper Marl Formation. To take advantage of a shaft bell, the shaft should be extended a minimum of 10 to 15 feet within the Cooper Marl Formation. Final capacity will be highly dependant on marl embedment depth, bell diameter, and the required capacity.

Drilled shafts can be installed utilizing either the dry or wet methods as described by SCDOT Supplemental Specification "*Section 712 – Drilled Shafts and Drilled Pile Foundations*". The selection of either method will be highly dependent on ground and water conditions encountered.

The "dry" installation method is used when soil and groundwater conditions will allow for construction and inspection of the shaft in a relatively dry condition. For this method of drill shaft construction to be viable in the downtown Charleston area would require the use of steel casing socketed into the Cooper Marl Formation. The socket should penetrate the Cooper Marl Formation a minimum of 5 feet. Steel casing is typically installed using a vibratory hammer.

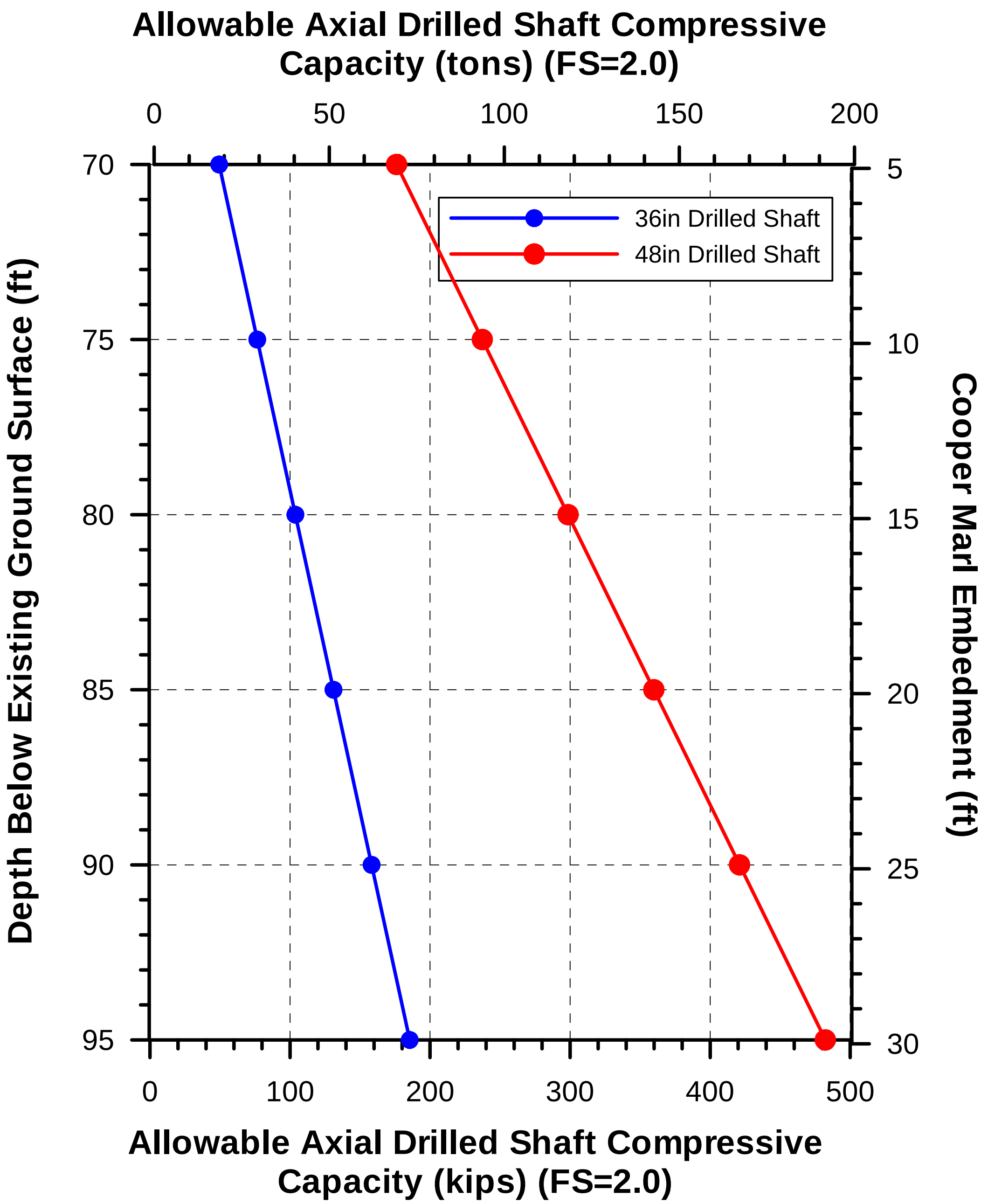


Figure 5. Allowable Axial Compressive Capacity (Drilled Shafts)

Note the installation of temporary construction casing may generate significant vibrations during construction. This may be especially critical when the casing is vibrated through the dense deeper sand located between 55 and 67 feet below the existing ground surface. These vibrations may not be particularly evident on the ground surface during installation (as the overlying soil mass will dampen the vibrations). However, as the existing structure is supported on a deep foundation system where vibrations may travel up the existing foundation elements, potentially affecting the building occupants as the effects of soil dampening will be limited.

The second method of shaft construction utilizes the “wet” installation method. Typically, in the wet construction method, water or slurry is pumped into the excavation as the shaft is advanced to completion depth to maintain shaft wall stability. To maintain the integrity of the upper section of the shaft, steel casing is drilled (screwed) into place to depths of 10 to 20 feet. Vibrations from the installation of "short" temporary casing for this method of construction are not expected to be as significant as the dry construction method.

Drilled Shaft Load Testing

To verify the axial design compressive capacity, a static load test should be performed on a test shaft 2 feet in diameter installed at a non-production location. A static load test will require the use of a reaction frame which typically consist of reaction shafts. Final production shaft capacity can be scaled from the test shaft results. WPC can provide recommendations for reaction frame on request. Larger (production) shafts may be tested, but this would likely require a significant expenditure and/or generate high vibrations during testing (Osterberg Load Cell/Statnamic Load Test).

The geotechnical and structural engineer should select the test shaft location that is representative of the site. A static load test should be performed using the Quick Test Procedure in accordance to the standards described in ASTM D1143, *"Standard Test Method for Deep Foundations Under Static Axial Compressive Load"*. Installation methods for the test shaft should be the same as those used during construction. Non-destructive integrity testing (NDT), such as cross-hole sonic logging (CSL) testing, should be performed on the shaft prior to performing the load test.

Augered Cast-in-Place (ACIP) Piles

As discussed previously, Augered Cast-in-Place (ACIP) piles may provide an economical alternative to driven piles and/or drilled shafts. ACIP piles utilize continuous flight hollow stem augers, which have been modified to deliver grout under pressure to form a continuous concrete pile. ACIP piles typically range from 12 to 36 inches in diameter. ACIP piles are constructed by advancing a hollow stem auger to the appropriate depth. Grout is then continuously pumped down through the annulus of the auger and forced into the surrounding soil mass as the auger is removed forming a continuous pile. After the auger has been removed a single reinforcing bar is then typically lowered into the wet grout (as necessary). However, depth and time constraints can be a limiting factor if cage reinforcement is required as concrete/grout near the base of the pile may set or be to dense to allow for complete installation of reinforcement. Additionally, in soil conditions where a significant deposit if soft soils is present as is the case for this project site, grout pumped under pressure may induce bulging of the pile walls. This will result in large quantities of grout being pumped into the ACIP. Casing may be necessary to limit grout loss. As with drilled shafts, ACIP piles will generate a significant quantity of spoil that will require disposal. A close cousin to ACIP piles is Auger Cast Displacement Piles (ACDP) which are similar in construction to the ACIP, however, the auger cuttings are forced into the surrounding soil mass. Thus no significant cuttings are spoiled at the ground surface with this method of construction. Use of either construction method would require a significant test pile and quality control plan.

WPC can provide additional recommendations for ACIP/ACDP piles on request.

Construction Vibration Considerations

Ground vibrations, which can be a concern to the existing adjacent structures, should be monitored during driven pile/drilled shaft casing installation. WPC can conduct vibration monitoring in conjunction with deep foundation installation monitoring. Vibration monitoring should also be undertaken inside the existing facility, especially if temporary steel casing is vibrated into the Cooper Marl Formation (i.e. dry shaft construction method). In addition, we recommend that a pre and post condition survey be performed of the adjacent structures to document existing cracks and other significant defects on adjacent structures. The pre and post condition survey should extend a minimum of three (3) pile/shaft lengths from the building footprint.

WPC has significant experience with vibration monitoring in the Charleston area and has developed vibration monitoring criteria for structures based on our own research and review of other established research from the US Bureau of Mines, German standards, etc. It is anticipated the vibrations generated on the site will be tolerable to the existing surrounding structures, and our recommended criteria are included in Figure 6.

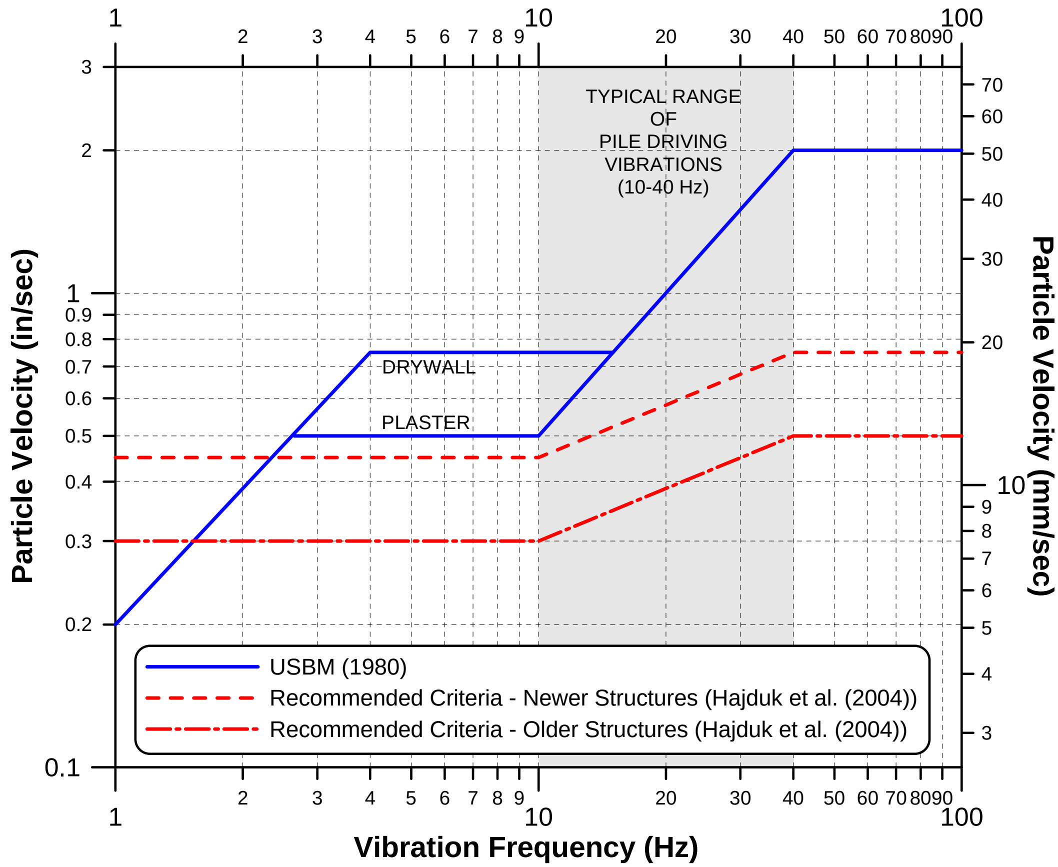


Figure 6. Recommended Vibration Monitoring Criteria

In addition to limiting vibrations during construction operations, it is our understanding that controlling the magnitude of noise generated will also be a significant consideration for this project as the proposed site will be located adjacent to rooms being actively used by patients. Although, noise cannot be totally eliminated from any construction activity, methods/equipment can be incorporated into pile installation to limit their effects, this is especially critical for pile driving operations where impact induced noise can be significant or the installation of temporary steel casing. A primary method of limiting noise generation (for pile installation) is to utilize a hydraulic hammer such as a Junnten Hammer. Utilizing a Junnten Hammer, with a "silenced" ram can significantly reduce noise levels as the ram is completely enclosed. Several contractors in the Charleston area have access to the Junntan Hammer. Other options may be available to limit the magnitude of construction generated noises. Further discussions with foundation contractors may disclose other methods employed elsewhere.

GENERAL SITE RECOMMENDATIONS

Site Preparations

The proposed addition footprint should be stripped of surface topsoil, trees, concrete, and other deleterious materials. Stripping should extend at least 5 feet beyond the footprint of the planned construction. All deep depressions resulting from the removal of root balls, rubble, etc. should be backfilled with Controlled Fill subsequently described.

After stripping, the subgrade within the proposed building footprint should be proofrolled. Proofrolling will help detect any isolated soft or loose areas that "pump", deflect or rut excessively. A fully loaded pneumatic tired tandem axle dump truck, capable of transferring a load of in excess of 20 tons, should be utilized for this operation. Proofrolling should be performed under the observation of the geotechnical engineer or their representative.

If encountered, the site work contractor should be prepared to undercut and remove larger pieces of debris that may be encountered within the proposed building footprint as they can impede, damage, or deflect deep foundation elements during the installation process. It may be prudent to consider surveying the building footprint with ground penetrating radar (GPR) prior to site work to locate near surface debris and potentially previously unidentified underground utilities (i.e. Subsurface Utility Engineering or SUE). A GPR survey has the potential to significantly reduce the volume of buried debris removed at the time of foundation installation.

Controlled Fill

Controlled Fill soils should be free of organics and debris. Fill soils should be sands classified as SP or SM according to the Unified Soil Classification System, with a Modified Proctor Maximum Dry Density of at least 100 pounds per cubic foot (pcf) (ASTM D1557). The fill should have a maximum fines content (i.e. percent passing a #200 sieve) of 15%. Controlled Fill should be placed in uniform lifts and compacted to at least 95% of its Modified Proctor Maximum Dry Density as determined by ASTM D1557.

On Grade Slabs

Based on the potential for significant primary and secondary settlements of the soils (approaching 2 inches from primary settlement with additional settlement associated with secondary consolidation) the use of slab on grade construction is not recommended for this project. We recommend the first floor slab be structurally supported on the deep foundation system.

Additional Considerations

Based on existing site grades, we estimate little if any fill will be necessary to establish nominal construction grades. However, if fills are required, WPC should be contacted to review and revise the recommendations presented herein, as necessary. Fill loads will generate additional downdrag forces, not only on the foundation elements of the proposed addition but also the existing elements, especially perimeter elements located in close proximity to the proposed construction. This additional loading may exceed the axial capacity of the existing foundation elements and should be examined. The use of lightweight fill or other remedial measures may be necessary if a significant quantity of fill is anticipated.

QUALITY ASSURANCE TESTING

WPC has been performing International Building Code (IBC) Chapter 17 Special Inspections since they were adopted by North Carolina in 2002 and has one of the largest pools of inspectors in the Southeast. WPC has assembled a team of professionals that has extensive experience with Special Inspections for projects including hospitals, schools, office parks, condominium/apartment projects, etc. The South Carolina Office of State Engineer has recognized WPC's expertise with Special Inspections by awarding WPC an "on-call" contract for Chapter 17 Special Inspection services. WPC excels in providing premium quality professional special inspection services such as:

- Site preparation
- Clearing
- Subgrade verification
- Fill placement and density testing
- Driven pile foundations (if applicable)
- Drilled shaft foundations (if applicable)
- ACIP foundations (if applicable)
- Deep foundation quality control (discussed in the elsewhere in the report text and Appendix)
- Concrete
- Structural steel and welds
- Reinforcing steel
- Post tensioning tendons
- Masonry
- Sprayed Fireproofing
- EIFS

WPC appreciates the opportunity to be of service to you on this project. This report is for the sole use of this project and should not be relied upon otherwise. If you have questions concerning the contents herein, please contact us.

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 7849 WILLIAM B. WRIGHT

Respectfully submitted,
WPC, Inc.

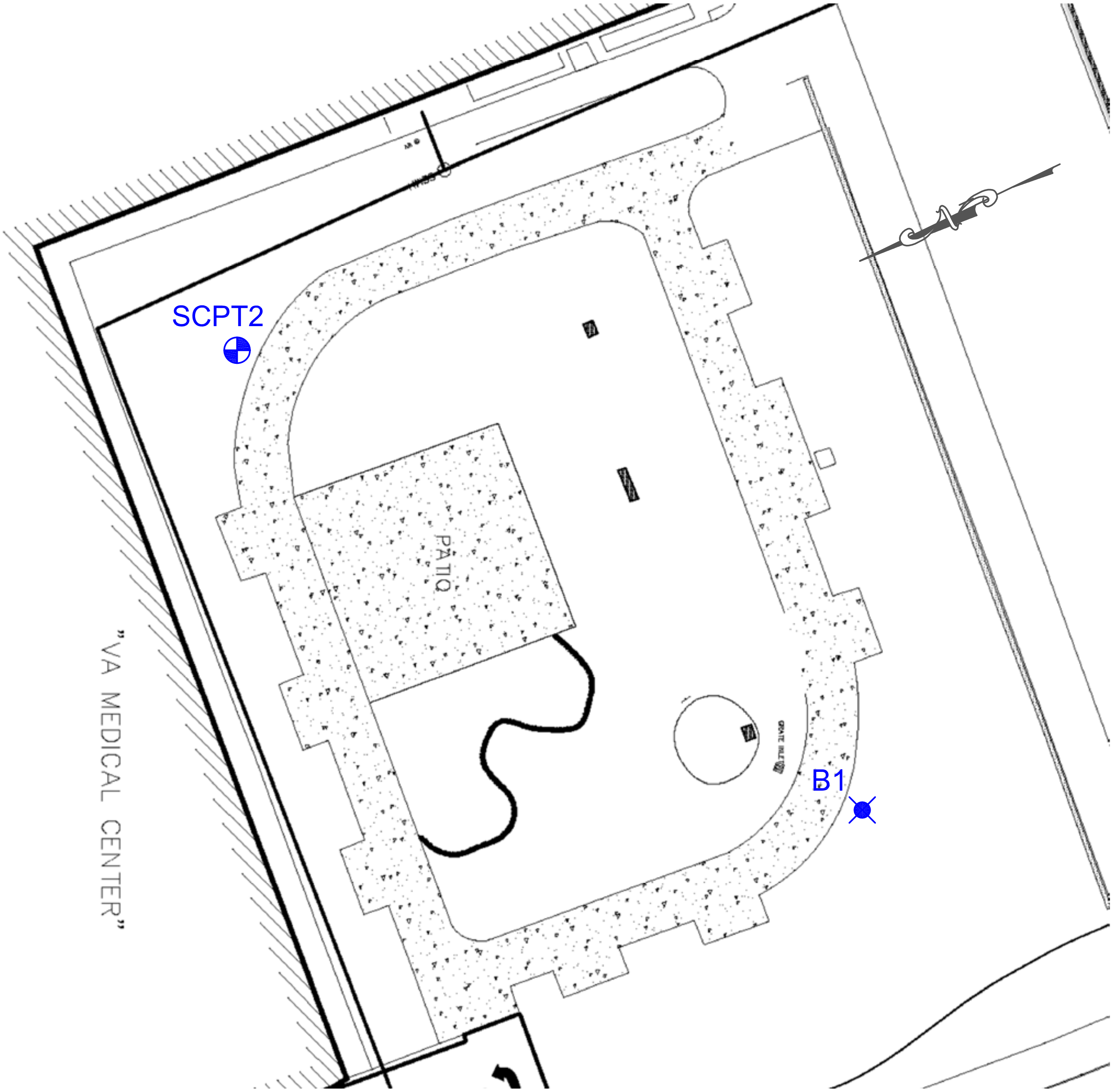
Kenneth J. Zur, P.E.
Geotechnical Engineer

William B. Wright, P.E. 4/6/09
Principal Engineer M. Sanders

SOUTH CAROLINA WRIGHT PADGETT CHRISTOPHER No. C00483 CERTIFICATE OF AUTHORIZATION

Attachments:
- Figure 1. Test Location Plan
- Piezocone Test Log
- Piezocone Penetration Classification
- Soil Test Boring Log
- Field Testing Procedures
- Hand Auger Boring Log
- Shear Wave Velocity Profile
- Laboratory Test Results
- Advantages and Disadvantages of Various Deep Foundation Systems
- Summary of Methods for Various Deep Foundation Systems

FIGURE 1. TEST LOCATION PLAN



LEGEND

- SEISMIC CONE PENETRATION TEST (SC-)
- SOIL BORING TEST (B-)

Drawn By: kjz
Approved By: kjz
Project Number: WPC1209.00081
Date: 04.01.09
Scale: NTS

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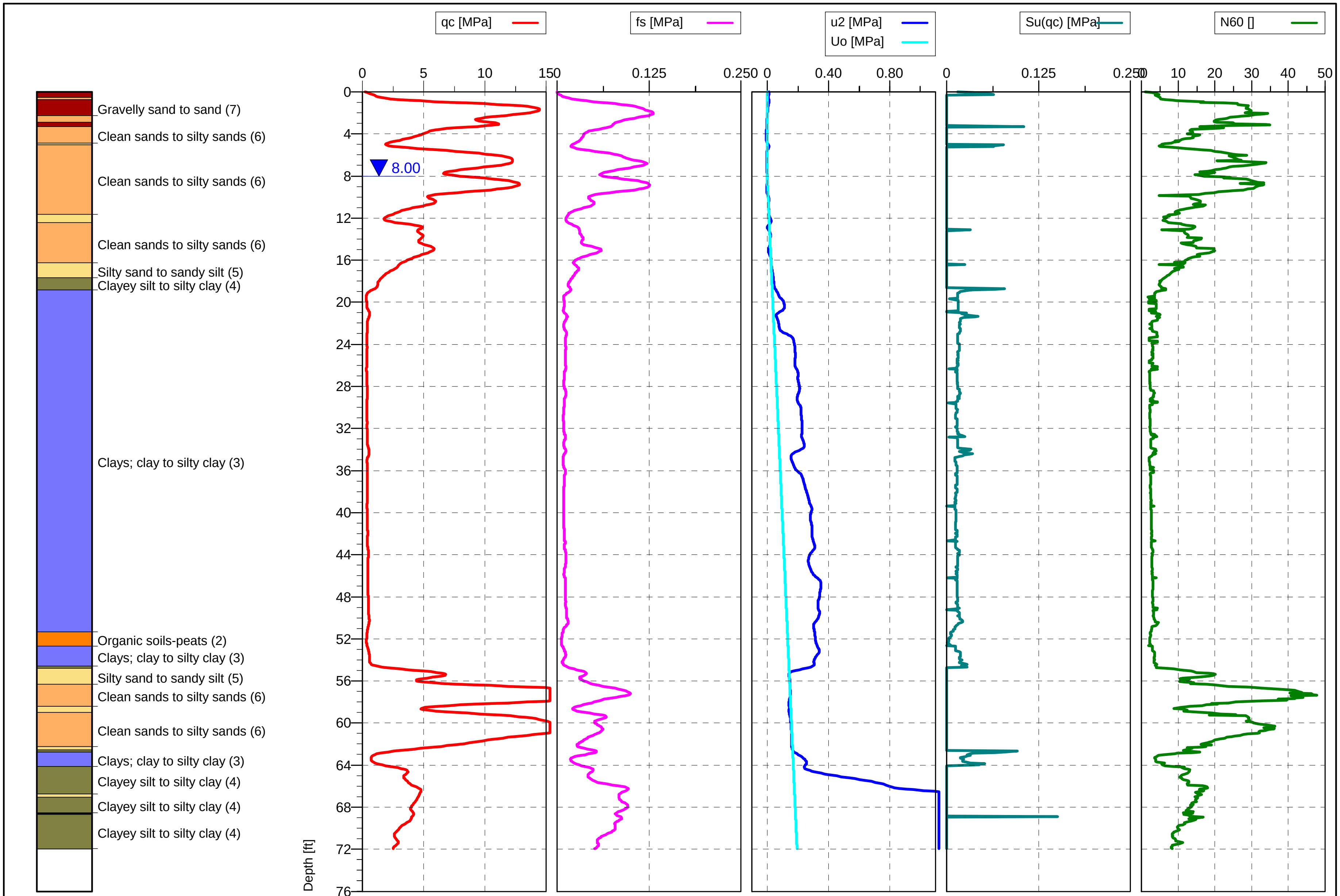
ENGINEERING, ENVIRONMENTAL
& CONSTRUCTION SERVICES
1450 Fifth Street West
North Charleston, SC 29405
tel. 843.884.1234
fax. 843.884.9234
www.wpceng.com

RALPH H. JOHNSON
VA MEDICAL CENTER
CHARLESTON, SC

FIGURE 1. TEST LOCATION PLAN

PIEZOCONE TEST LOG



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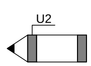


Cone No: 0
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: CHARLESTON, SC	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: SCPT2
Project ID: WPC1209.00081	Client: LINDBERGH & ASSOCIATES	Date: 3/20/2009	Scale:
Project: RALPH H. JOHNSON VA MEDICAL CENTER		Page: 1/2	Fig:
		File: SCPT1.cpd	

PIEZOCONE PENETRATION CLASSIFICATION

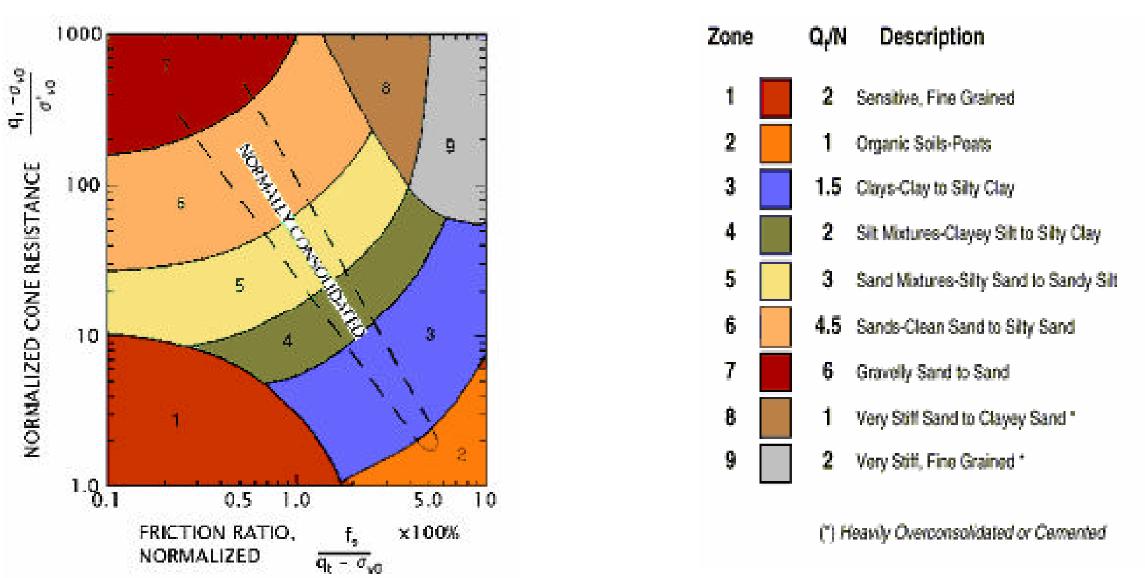
Cone Penetration Classification

The tip resistance (q_c) is measured as the maximum force over the projected area of the tip. It is a point stress related to the bearing capacity of the soil. The measured q_c must be corrected for porewater pressure effects (Lunne et al, 1997), especially in clays and silts where porewater pressures typically vary greatly from hydrostatic. This corrected value is known as q_t,, which is reported in the Piezocone Penetration Logs. The u_2 position element is required for the measurement of penetration porewater pressures and the correction of tip resistance. The sleeve friction (f_s) is used as a measure of soil type and can be expressed by friction ratio: **FR = f_s/q_t**.

The estimated stratigraphic profiles included in the Piezocone Penetration Logs are based on relationships between q_t, f_s, and U_2. The normalized friction ratio (FR_N)is calculated by using:

$$FR_N = \frac{f_s}{q_t - s_{vo}'} \times 100\%$$

and is indicative of soil behavior and is used to classify the soil behavior type. Typically, cohesive soils, such as plastic silts and clays, have high FR values, low q_t values, and generate large excess penetration porewater pressures. Cohesionless soils, such as sands, have lower FR's, high q_t values, and typically do not generate excess penetration porewater pressures. The following graph (Robertson, 1990) presents one of the accepted correlations used to classify soils behavior types.



SOIL TEST BORING LOGS

LOG OF BORING NO. B-1

PROJECT:	Ralph H. Johnson VA Medical Center Charleston, SC WPC1209.00081
DATE STARTED:	3/31/2009
DATE FINISHED:	3/31/2009
SURFACE ELEVATION:	11.17ft
DRILLING METHOD:	Mud Rotary
DRILLING RIG:	CME45C
CASING:	Not Used
DRILLER:	E. Vezie
WATER LEVEL (EOD unless otherwise stated):	@ 24 Hrs
LOGGER:	K. Zur

NOTES:
HAB conducted in advance of STB

CLIENT: Lingbergh & Associates

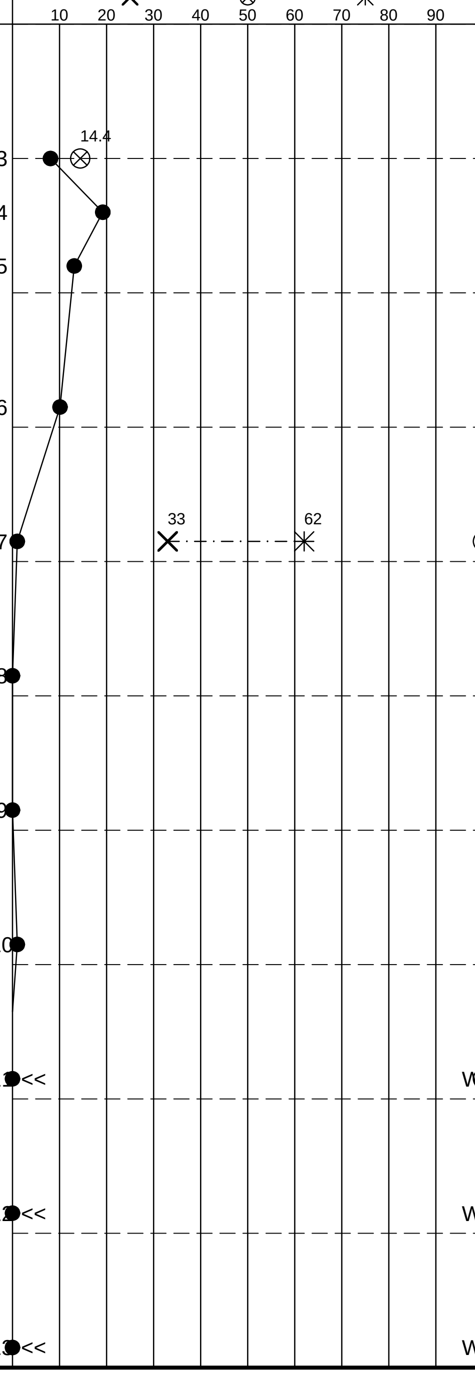
DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)
0–17	SAND	SAND (SP), medium dense, brown, moist, fine, with trace organics - pale yellow, cement seams, medium, between 2'0" and 4'0" - very dark grayish brown - light olive brown - clayey, dark greenish gray between 8'0" and 12'0" - very dark gray	
17–50	SILT	SILT (MH), very soft, dark greenish gray PLUFF MUD - very dark gray, trace organics below 48'6"	

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO/TYPE	N VALUE
5	6.2	SS-3	9
7		SS-4	20
9		SS-5	14
10	1.2		
14		SS-6	11
15	-3.8		
19		SS-7	2
20	-8.8		
24		SS-8	1
25	-13.8		
29		SS-9	1
30	-18.8		
34		SS-10	2
35	-23.8		
39		SS-11	WOH/18"
40	-28.8		
44		SS-12	WOH/18"
45	-33.8		
49		SS-13	WOH/18"

STANDARD PENETRATION TEST DATA (blows/foot)

MOISTURE CONTENT RESULTS

Plastic Limit — Moisture Content — Liquid Limit



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Groundwater at end of drilling

Groundwater 24 hours after drilling

LOG OF BORING NO. B-1

PROJECT: Ralph H. Johnson VA Medical Center
Charleston, SC
WPC1209.00081

DATE STARTED: 3/31/2009	DATE FINISHED: 3/31/2009	SURFACE ELEVATION: 11.17ft	NOTES: HAB conducted in advance of STB	
DRILLING METHOD: Mud Rotary	DRILLING RIG: CME45C			
CASING: Not Used	DRILLER: E. Vezie			
WATER LEVEL (EOD unless otherwise stated): @ 24 Hrs	LOGGER: K. Zur		CLIENT: Lingbergh & Associates	

STANDARD PENETRATION TEST DATA (blows/foot)
MOISTURE CONTENT RESULTS: Plastic Limit, Moisture Content, Liquid Limit

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet)	SAMPLE NO/TYPE	MOISTURE CONTENT RESULTS	N VALUE
55	- sandy below 53'6"	-43.8	SS-14	<<	WOH/18"
60	SAND (SP), medium dense, dark gray and brown, wet - trace organics below 58'6"	-48.8	SS-15	23.1	14
65	- no recovery between 63'6" and 65'0"	-53.8	SS-16		4
70	Sandy SILT (ML), stiff, light olive green, moist, with H2S odor	-58.8	SS-17		14
75	- firm below 73'6"	-63.8	SS-18		7
80	COOPER MARL FORMATION	-68.8	SS-19	36, 42, 58.4	4
85		-73.8	SS-29		6
90		-78.8	SS-21		5
	Bottom of Borehole				

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Groundwater at end of drilling

Groundwater 24 hours after drilling

FIELD TESTING PROCEDURES

FIELD TESTING PROCEDURES

Soil Test Borings

All boring and sampling operations were conducted in accordance with ASTM D-1586. Initially, the borings were advanced either mechanically or by wash boring through the soils. Where necessary, a heavy drilling fluid is used below the water table to stabilize the sides and bottom of the drill hole. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-barrel sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is designated the "Standard Penetration Resistance". The penetration resistance, when properly evaluated, is an index to the soil strength.

Soil Classifications

Soil Classification provides a general guide to the engineering properties of various soil types and enables the engineer to apply his experience to current situations. In our exploration, samples obtained during drilling operations are examined and visually classified (ASTM D-2488) by an engineer. The soils are classified according to color, texture, and relative density or consistency (based on standard penetration resistance). The relative density and consistency designations are as follows:

SANDS		SILTS AND CLAYS	
N(SPT	Relative Density	N(SPT)	Consistency
0-4	Very Loose	0-2	Very Soft
5-10	Loose	3-4	Soft
11-30	Medium Dense	5-8	Firm
31-50	Dense	9-15	Stiff
50+	Very Dense	16-30	Very Stiff
		31-50	Hard
		50+	Very Hard

Water Level Readings

Water table readings are normally taken in conjunction with borings and are recorded on the Soil Boring Records. These readings indicate the approximate location of the hydrostatic water table at the time of our field exploration. The groundwater table may be dependent upon the amount of precipitation at the site during a particular period of time. Fluctuations in the water table should also be expected with variations in surface run-off, evaporation and other factors.

Water level readings are generally obtained 24 hours after the borings are completed. The time lag of at least 24 hours is used to permit general stabilization of the groundwater table which has been disrupted by the drilling operation. The readings are taken by dropping a weighted line down the boreholes or using an electrical probe to detect a water-level surface.

Occasionally the boreholes will cave-in, preventing the water level readings from being obtained or trapping drilling water above the cave-in zone. The cave-in depth is measured and recorded on the Soil Boring Records. Water level readings taken during the field operations do not provide information on the long term fluctuations of the water table. When this information is required, piezometers are necessary to prevent boreholes from caving in.

HAND AUGER BORING LOG

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Hand Auger Boring Logs

WPC Project Ralph H. Johnson VA Medical Center
WPC Project No WPC1209.00081
Date 3/18/2009 ***Location*** Charleston, SC
Logger R. Frazier ***Review*** K. Zur

Test Number	Depth (in)		Soil Stratigraphy	USCS Classification
	from	to	Description and Remarks	
HA by SC2	0	2	Topsoil	
	2	24	Dark Tan Silty SAND w/ brick debris	SP
	24	48	Tan SAND w/ debris	SP
			No Groundwater Encountered	

SHEAR WAVE VELOCITY PROFILE

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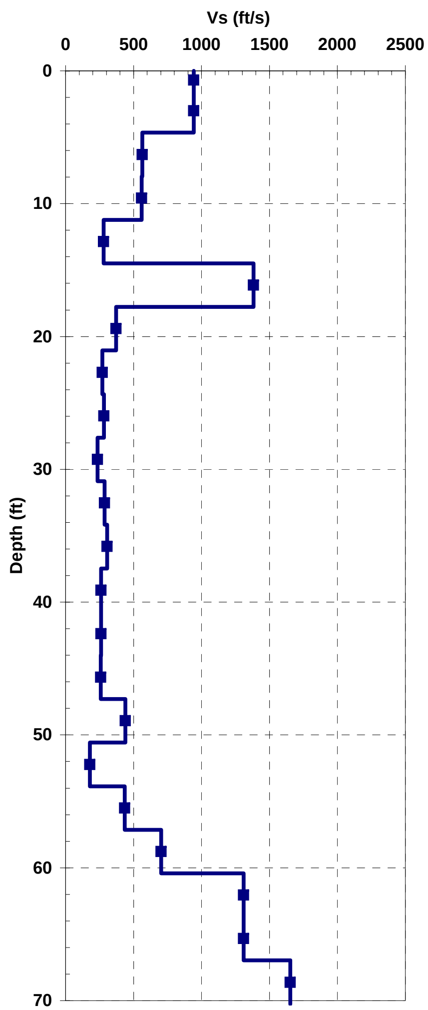
Test Site:	*VA Hospital*	Truck:	*Pagani 220-73*
Location:	*Charleston, SC*	Cone:	*Geotech AB 5 ton*
Client:	*Lindbergh & Associates*	Sounding:	*SC2*
Project:	*WPC1209.00081*	GWT (ft):	*8.0*
Latitude:	*32.78470*	ASTM:	*D 5778*
Longitude:	*-79.9534*	Engineer:	*K. Zur*
Elevation:	Date: *3/20/2009*	Operator:	*RF*



Tip to Geophone (ft): 0.98
Cone to Source (ft): 1.64

Depth	Vs
feet	ft/s
2.0	943
5.2	943
8.5	564
11.8	559
15.1	280
18.4	1384
21.6	371
24.9	269
28.2	282
31.5	234
34.8	287
38.0	305
41.3	260
44.6	260
47.9	259
51.2	439
54.4	179
57.7	434
61.0	702
64.3	1310
67.6	1310
70.8	1653

$\overline{V}_S$ =	*515*	*ft/s*
Site Class:*	*E*	

***Per IBC 2006 Weighted Shear Wave Velocity Criterion**
(Liquefaction, Soft Clay, etc, Not Considered In This Calculation)

LABORATORY TEST RESULTS

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LABORATORY TESTING SERVICES

Water Content of Soil ASTM D2216

Project Name:	Ralph H. Johnson VA Medical Center	WPC Project #:	WPC1209.00081
Date:	4.1.09	Project Manager:	Ken Zur

Sample #	Depth (feet)	Moisture Content (%)
B1 S3	4-6	14.4
B1 S7	18.5-20	102.6
B1 S11	38.5-40	102.1
B1 S15	58.5-60	23.1
B1 S19	78.5-80	58.4

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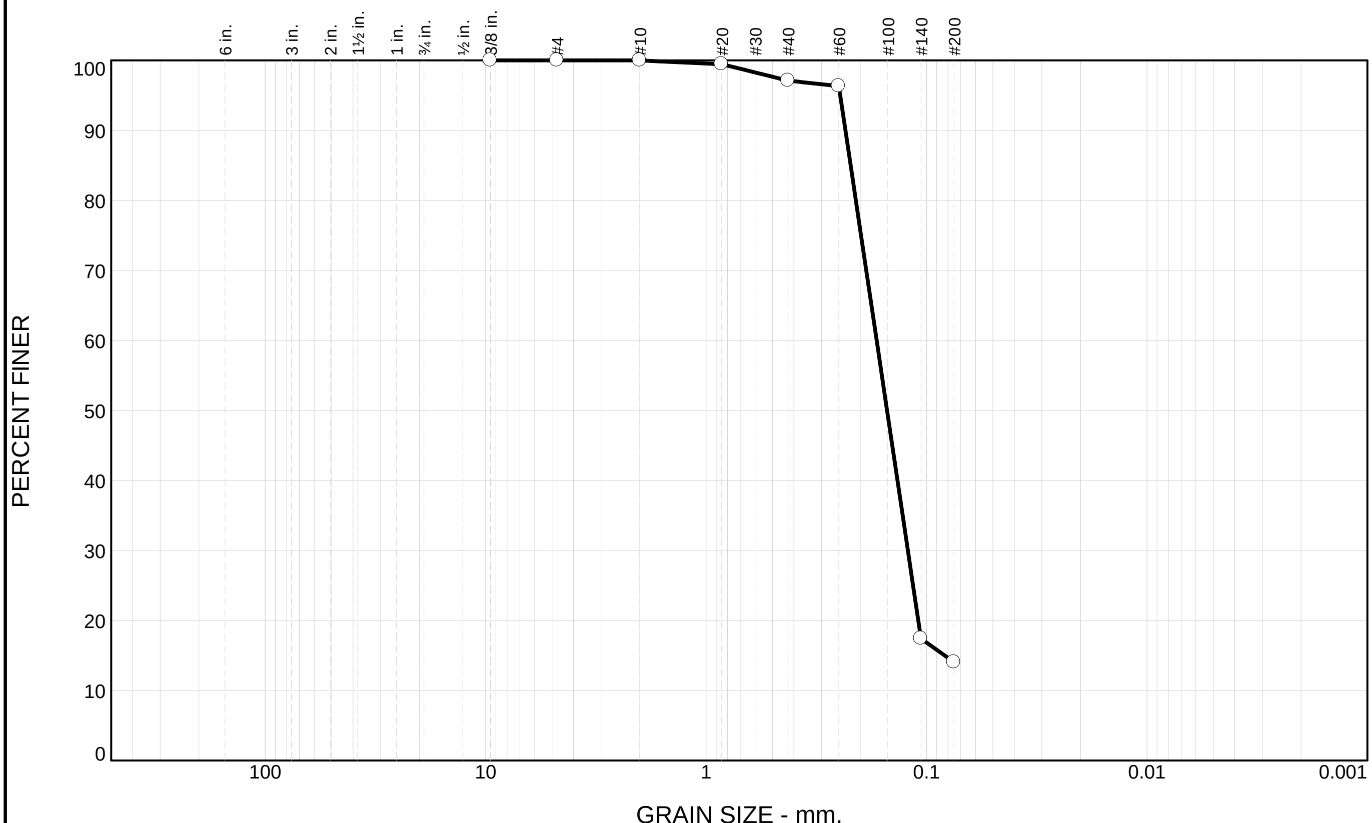
Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	2.9	83.1	14.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	99.5		
#40	97.1		
#60	96.3		
#140	17.4		
#200	14.0		

* (no specification provided)

Soil Description

Very Dark Gray Silty Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2334 D_{85}= 0.2210 D_{60}= 0.1684
D_{50}= 0.1511 D_{30}= 0.1215 D_{15}= 0.0827
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

NM = 14.4%

Source of Sample: Boring Samples **Depth:** 4'-6'
Sample Number: B1 S3 **Date:** 4.1.09

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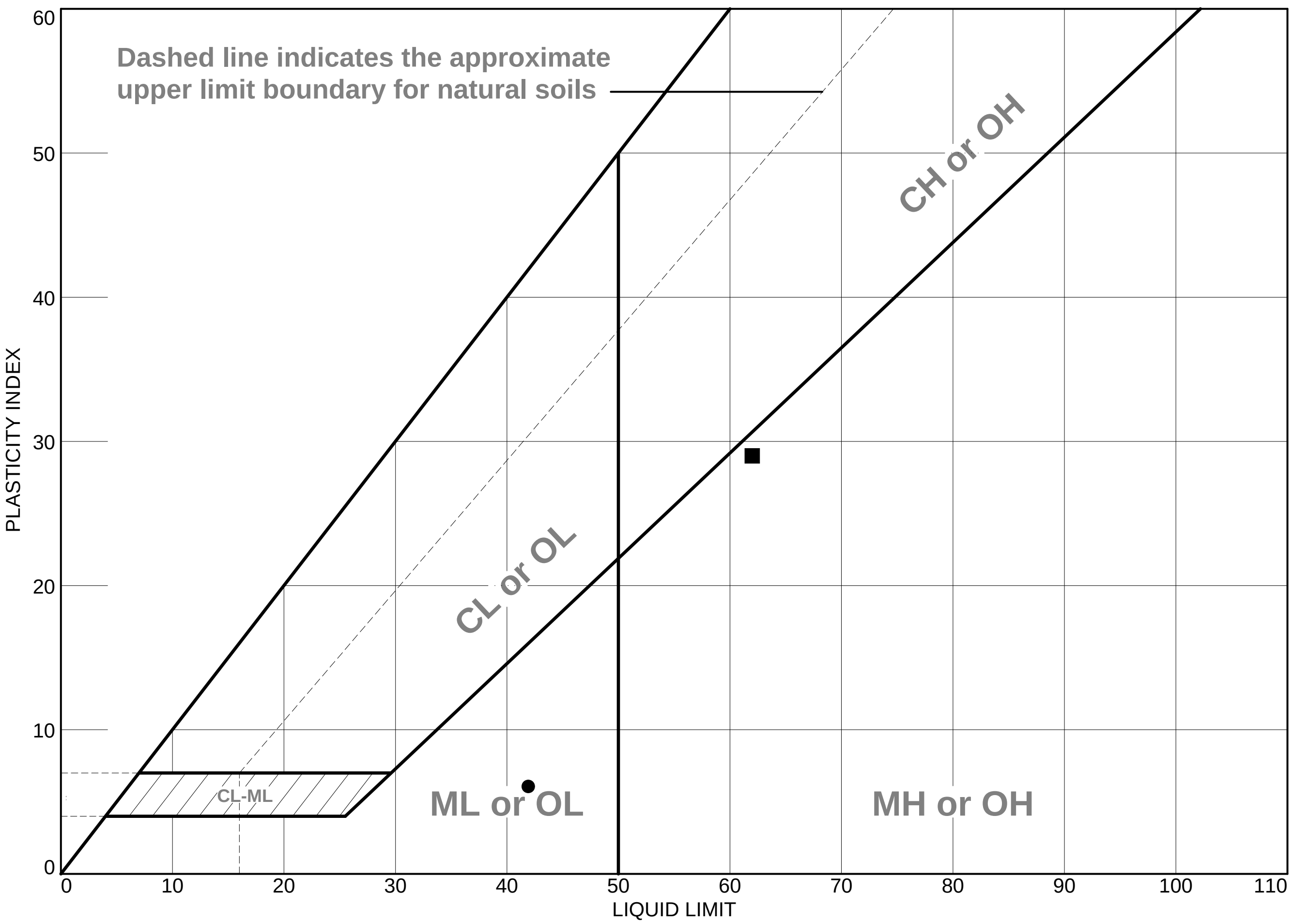
North Charleston, SC

Client: Lindbergh & Associates
Project: Ralph H. Johnson VA Medical Center

Project No: 1209.00081 **Figure**

Tested By: BC **Checked By:** KP

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive Brown Silt	42	36	6			ML
■	Very Dark Gray Elastic Silt	62	33	29			MH

Project No. 1209.00081 **Client:** Lindbergh & Associates

Project: Ralph H. Johnson VA Medical Center

● **Sample Source:** Boring Samples **Depth:** 78.5'-80' **Sample No.:** B1 S19

■ **Sample Source:** Boring Samples **Depth:** 18.5'-20' **Sample No.:** B1 S7

Remarks:

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North Charleston, SC

Figure

Tested By: BC **Checked By:** KP

ADVANTAGES AND DISADVANTAGES OF VARIOUS DEEP FOUNDATION SYSTEMS

Advantages and Disadvantages of Various Deep Foundation Systems.

Foundation System	Advantages	Disadvantages	Comments
Drilled Shaft - Straight	• Minimal vibrations • Can achieve high capacity • No impact noise generated	• Requires specialty contractor • Requires pile cap and grade beam below the ground surface • Inspection during and after construction is critical to ensure proper performance • Construction noise	• Typical diameters range from 30 to 60 inches • Construction noise associated with equipment not impact
Driven Steel H-piles & Pipe Piles	• Possible to splice short lengths together • Readily available	• More expensive material cost than concrete • Less vibrations produced than driven concrete piles but more than drilled shafts • Construction Noise	• May be best suited for medium to high capacity piles • Noise associated with hammer impact and construction equipment
Driven Pre-stressed Concrete (PSC) Piles	• Less expensive • Readily available	• Long piles may be difficult to transport • May generate significant vibrations during installation • Construction Noise	• Routinely used in the Charleston area. • Noise associated with hammer impact and construction equipment

SUMMARY OF METHODS
FOR VARIOUS DEEP FOUNDATION SYSTEMS

Summary of Quality Control Methods for Various Deep Foundation Systems.

Quality Control Technique	Foundation Type: Drilled Shafts	Foundation Type: Driven Piles
Visual	• Observe shaft cuttings to determine when Cooper Marl is encountered • Inspect shaft and bell prior to placement using equipment, such as downhole cameras, to ensure cuttings are removed prior to concrete placement	• Inspect piles for straightness and possible defects.
Documentation	• Record concrete quality prior to placement (slump testing). • Record strength of concrete cylinders from pours. • Monitor concrete quantity during placement. Comparison of actual to estimated concrete volume may determine shaft defects. Less concrete used indicates the drilled shaft may have collapsed. More concrete may mean a void was encountered.	• Record pre-augering depths, pile length, final pile tip and top elevations, and hammer type. • Record pile penetration on a blows per foot (bpf) basis during installation for each pile to ensure that the pile is meeting proper driving resistance.
Non-Destructive Testing (NDT)	• Placement of inspection tubes and use of cross-hole sonic logging (CSL) equipment to determine shaft integrity. • Use of small strain integrity testing equipment, such as the Pile Integrity Tester (PIT), on selected shafts.	• Using of high strain integrity testing equipment, such as the Pile Driving Analyzer, can determine pile integrity and capacity.
Vibration Monitoring	• Monitor vibrations at nearby structures during driven or vibrated steel casing installation.	• Monitor vibrations at nearby structures during driven pile installation.