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STEEL PIPE PILE DESIGN, INSTALLATION, AND DYNAMIC TESTING FOR A NEW PIER IN GEORGETOWN, SC

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ABSTRACT

A new pier extension was constructed for the South Carolina State Ports Authority (SCSPA) at Pier 31C in Georgetown, South Carolina. This pier was founded on 28 steel open ended pipe (OEP) piles driven into the underlying limestone and silts of the Pee Dee Formation. To account for the expected high lateral loads the new pier may experience during ship impacts and movements, both vertical and batter piles were installed. To the authors' knowledge, this is the first application of steel open ended pipe piles as a maritime foundation system in the Georgetown, South Carolina area.

The soil stratigraphy at the site consisted of soft river deposits overlaying interbedded limestone and silt layers of the Pee Dee Formation. Significant pile penetration into the cohesive deposits of the Pee Dee Formation was necessary to generate the required tension pile capacity. The results of the geotechnical investigation suggested that the intermittent limestone layers would cause difficult driving conditions for pre-stressed concrete piles, which are traditionally used in the area.

This paper provides an overview of the project, discusses the selection and design processes and installation of the steel OEP piles, and presents the results of the dynamic load testing program conducted to verify the pile design. In addition, measurements of time dependent pile capacity gain and pile plugging made during pile installation are also presented and discussed.

INTRODUCTION

Pier 31C of the Port of Georgetown, SC, located on the Sampit River, is a dedicated breakbulk and bulk cargo facility. The top commodities shipped through this pier are steel, salt, cement, aggregates and forest products. The existing Pier 31C had a berth of 500 feet. Ships calling on this facility were routinely exceeding this 500 foot length and often required moving the ship to allow full access for unloading and loading cargo. The owner/operator of the pier, the South Carolina State Ports Authority (SCSPA), wanted to expand the existing pier by 100 feet to accommodate these larger ship sizes. As the foundation system for the pier expansion, twenty eight (28) steel open ended pipe (OEP) piles were installed into the underlying limestone and silts of the Pee Dee Formation. A layout of the new pier, showing the pile locations relative to the existing pier, is provided in Figure 1.

The typical soil profile at new pier location consisted of very soft sandy, clayey silt (ML) starting at 10.1m (34ft) and ranging to 10.7m (35ft) below mean sea level (MSL). Underlying this silt layer is limestone with interbedded silt layers which extends to an elevation of -16.5m (-54ft) from MSL. Typical standard penetration test N values within the limestone were 50 blows per 0.15m (6 inches). Following the limestone is a sandy, clayey silt (ML-MH) with cemented seams. The limestone and sandy, clayey silt are part of the Pee Dee Formation. The Pee Dee formation was deposited during the Cretaceous period in an open marine environment and consists of interbedded clayey sand, impure limestone, and massive dark clays (Cooke, 1936). Figure 2 presents a typical soil profile of the site.

PILE DESIGN

The hard upper limestone layer presented certain difficulties for installing piles for the pier extension. Previous experience with installing piles through this layer has shown that typical pre-stressed, pre-cast concrete (PPC) piles can experience practical refusal or damage during installation due to high impact stresses. Therefore, several pile alternatives where evaluated to determine if they could provided adequate capacity.

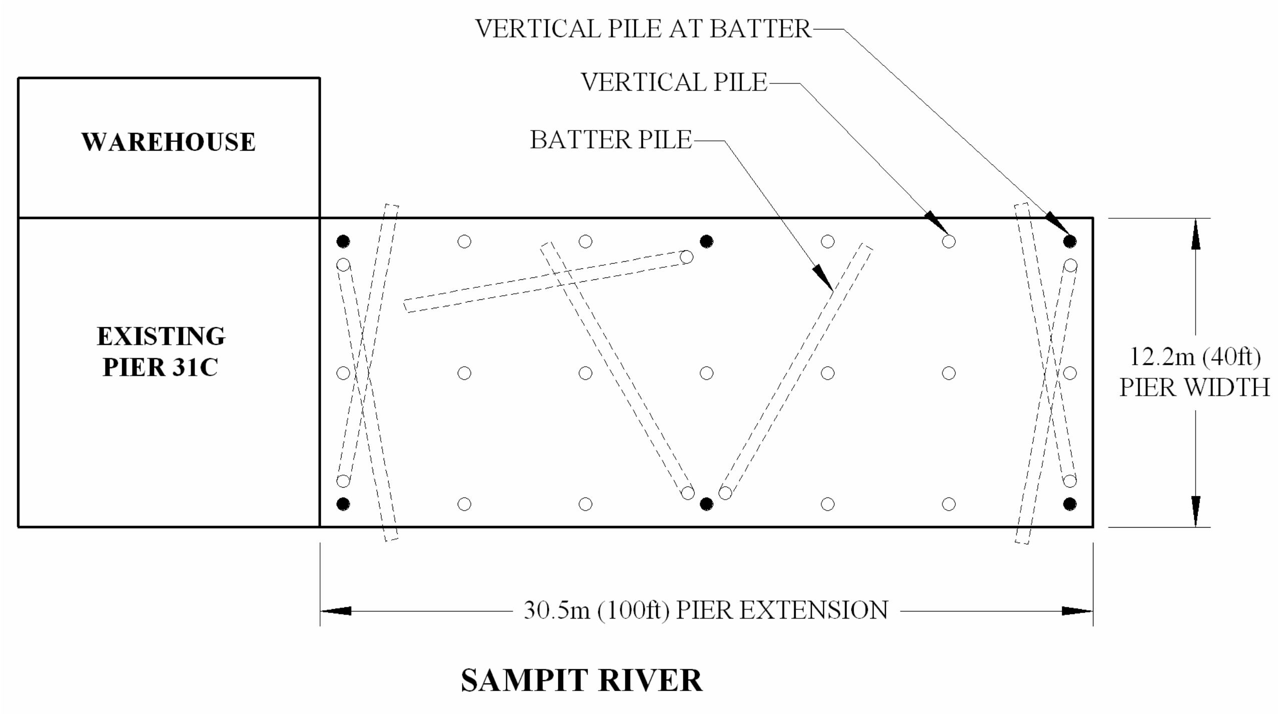

Fig. 1. New Pier Extension Pile Layout.

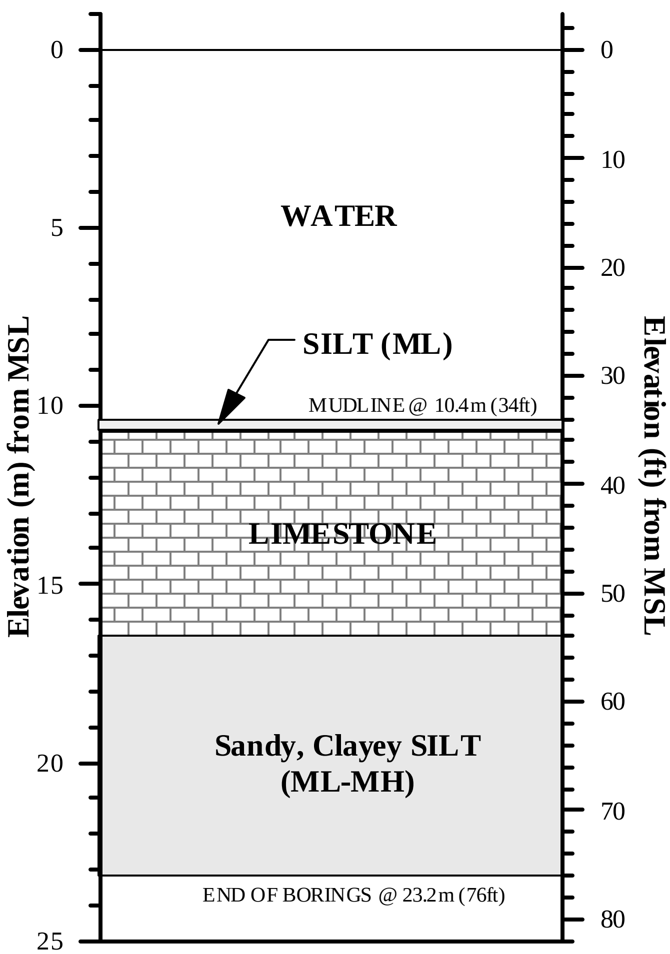

Fig. 2. Typical Soil Profile.

The following pile types were evaluated to determine if they could provide the support of the required structural loads and be driven with typical available impact hammers:

- Pre-stressed concrete piles with heavy H-pile "stingers" cast into the tip for penetrating the limestone.
- Steel H-piles.
- Steel pipe piles, both open and closed ended.

Corrosion concerns regarding the steel piles and steel pile components were addressed in the evaluation. Based on the required structural loads, steel open ended pipe (OEP) piles were selected for the project. The SC Ports Authority desired to expand the wharf length by 100-feet long, but chose to restrict the width of this extension to 40-feet. To resist the lateral forces upon this narrow extension, batter piles penetrating through the limestone and into the underlying sandy, silty clay were required. The use of batter piles would require uplift (i.e. tension) capacity within some of the piles. Open-ended pile piles with thick walls were selected for their high strength and small bearing area to allow the piles to be driven into the limestone. Previous experience of the authors showed that steel OEP piles could be installed within the limestone formation with little to no problems provided that a hammer with sufficient rated energy was used. To the author's knowledge, this is the first application of open ended steel pipe piles for a marine structure in the Georgetown, SC area.

Galvanized steel open ended pipe (OEP) piles, with an outside diameter of 50.8cm (20inches) and a wall thickness of 1.27cm (0.5inches), were selected as the deep foundation system. The pile material was ASTM 252 Grade 3 steel, with an elastic modulus of 30,000 kips per square inch (ksi) and a nominal yield stress (f_y) of 50 ksi. Tests conducted on selected samples of the pile steel by the pile manufacturer (i.e. Georgia Tublar Products) produced an average yield strength of 58 ksi with a range between 49.7 to 65.0 ksi. The maximum allowable compressive and tensile driving stresses for each pile is 45 kips per square inch (ksi), based on the nominal material yield strength and standard engineering practice (ASCE 20-96, FHWA HI-97-013).

The plan of the pier yielded three standard pile designs, designated as vertical, vertical at batter locations, and batter piles (see Figure 1). The structural loading (both axial compression and tension) and the subsequent required pile embedment varied at these three standard locations. The loading also varied for the vertical piles dependent on the pile location within the pier. We note that due to the pier design, no lateral loads were required from the OEP piles. Table 1 presents a summary of the ultimate structural loading and minimum tip elevations required for the three standard pile locations.

Table 1. Summary of Required Structural Loads and Pile Embedment.

Pile Location	**Structural Loading (kN)**		**Min. Tip Elevation (MSL) (m)**
	Comp.	**Tension**	
Vertical			
- Corner Location	445	0	-18.9
- Short End, Center	1370	0	-18.9
- Long End, Center	1575	0	-18.9
- Interior	2171	0	-18.9
Vertical @ Batter	2518	1108	-25.0
Batter (2H:1V)	1495	1495	-18.3

PILE INSTALLATION AND DYNAMIC TESTING

A Delmag D46-23 open ended diesel hammer, with a maximum rated energy of 145.3 kN-m (107.2 kip-ft), ram weight of 44.9 kN (10.1 kips), and an equivalent stroke of 3.23m (10.6 feet), was used to install and dynamically test the production piles. This hammer has a speed range between 38-55 blows per minute (bpm). No pile cushion was used during the installation and dynamic testing of the steel OEP piles.

A wave equation analysis (WEAP) of the hammer-pile-soil system using the computer program GRLWEAP™ indicated that this hammer would have sufficient energy to install the piles to the required capacity. However, due to the inconsistent nature of the underlying limestone deposits and the cemented lens within the sandy, clayey silt, final pile driving criteria would not be set until after the high strain dynamic testing of the initial piles.

Dynamic testing was conducted on a total of six (6) of the twenty eight (28) production piles, which is ~21% of the total. The testing was performed in accordance with ASTM D4945 using the Pile Driving Analyzer™, manufactured by Pile Dynamics Inc. The dynamic testing originally was scheduled for one pile within each of the standard pile designs (i.e. vertical, vertical at batter, and batter) piles. However, due to interference experienced between the dynamic gages and the pile driving template and unusual driving conditions experienced by two of the production piles, dynamic testing was conducted on a total of six (6) production piles. Table 2 provides a summary of the dynamic testing.

Table 2. Dynamic Testing Summary.

Pile	**L[1] (m)**	**Type**	**Test Time[2]**	**Comments**
1	21.3	Vertical	EOD	Gage interference with pile template prevented final EOD dynamic testing
20	21.3	Vertical	EOD	Test Pile for Vertical Piles
23	27.4	Vertical @ Batter	EOD	Test Pile for Vertical @ Batter Piles.
			3DR	3DR to confirm pile capacity.
26	22.9	Batter	EOD	Did not meet pile driving criteria @ EOD
			1DR	1DR to verify capacity.
24	27.4	Batter	EOD	Test Pile for Batter Piles
15	21.3	Batter	3DR	3DR to confirm pile capacity of pile that did not meet pile driving criteria @ EOD.

NOTES:
1. L = Pile Length.
2. EOD = End of Driving, 3DR = Three Day Restrike.

DYNAMIC TESTING RESULTS

Signal matching analysis using the software program CAPWAP™ was used to determine the axial capacities of each of the tested piles. By using signal matching procedures, the ultimate axial compressive and tensile capacities could be determined. In addition to CAPWAP analyses, ultimate axial compressive capacities were determined using the Case Method and the Energy Approach Methods (Paikowsky et al, 1994). A summary of the CAPWAP dynamic testing analyses is presented in Table 3. A summary of the three methods of determining pile capacity from dynamic measurements is presented in Table 4.

Table 3. Dynamic Testing CAPWAP Analysis Summary

Pile	Test Time[2]	Tip Elev.[3] (m)	CAPWAP Capacity (kN)			% Tip Cap.
			Side[4]	Tip	Total	
1[1]	EOD	-16.76	13	453	466	97
20	EOD	-19.20	459	2309	2768	83
	EOD	-19.51	313	1803	2116	85
23	EOD	-17.68	374	2269	2643	86
	EOD	-22.86	221	393	614	64
	3DR	-22.86	2248	593	2841	21
26	EOD	-18.90	461	264	725	36
	1DR	-18.90	898	470	1368	34
24	EOD	-18.90	381	1533	1915	80
	1HrR[5]	-18.90	1565	1067	2632	41
15	3DR	-18.90	2962	363	3325	11

NOTES:

1. Pile 1 was not tested at final embedment.
2. EOD = End of Driving, 3DR = Three Day Restrike.
3. Tip Elevation at time of testing.
4. Side Capacity is also Tensile Capacity.
5. 1HrR = One (1) Hour Restrike.

Table 4. Dynamic Testing Summary.

Pile	Test Time[1]	Tip Elev.[2] (m)	CAPWAP Capacity (kN)	Case[3] Capacity (kN)	EA[4] Capacity (kN)
1	EOD	-16.76	466	494	555
20	EOD	-19.20	2768	3296	3048
	EOD	-19.51	2116	2313	2392
23	EOD	-17.68	2643	2798	2921
	EOD	-22.86	614	525	832
	3DR	-22.86	2841	2931	3950
26	EOD	-18.90	725	694	929
	1DR	-18.90	1368	1584	1725
24	EOD	-18.90	1915	1904	1153
	1HrR[5]	-18.90	2632	2567	2719
15	3DR	-18.90	3325	3220	4205

NOTES:

1. EOD = End of Driving, 3DR = Three Day Restrike.
2. Tip Elevation at time of testing.
3. Based on RMX method and Case Damping Coefficient (J_c) of 0.6.
4. EA = Energy Approach.
5. 1HrR = One (1) Hour Restrike.

The dynamic test results verified that the individual piles had the required compressive and tensile axial capacities, thereby verifying the pile design. A refined GRLWEAP™ analysis was performed using the data from the CAPWAP™ analyses to set a pile driving criteria. The final pile driving criteria was based on the GRLWEAP™ results and the minimum required tip elevation from the original pile design. The remaining piles were driven to these criteria. Of the remaining production piles, only one (1) (i.e. pile 15) did not achieve the required penetration blow counts at End of Driving (EOD). Therefore, this pile was dynamically tested during a three day restrike (3DR). The results of the 3DR verified that the pile had sufficient axial capacities.

Further analysis of the dynamic test results also showed that the cemented seams within the sandy, clayey silt layer of the Pee Dee formation substantially affected pile capacity. This is evident in the End of Driving (EOD) results of pile 23. At a tip elevation of -17.68m (58.0ft), the pile had a total axial compressive capacity of 2643 kN. At a tip elevation of -22.86m (75.0ft), the total axial compressive capacity of the pile was 614 kN. Blow count records taken during the installation of pile 23 showed a marked decrease in penetration blow counts shortly after the pile tip past elevation -18m (59.0ft). The higher pile capacity and penetration blow counts at and near elevation -17.68m (58.0ft) corresponded to a cemented seam encountered during geotechnical investigation.

Time Dependent Pile Capacity Gain

As the piles were founded within a cohesive soil deposit (i.e. the sandy, clayey silt of the Pee Dee Formation), time dependent pile capacity gain (i.e. pile "setup" or "freeze") was expected for this project. Therefore, the CAPWAP capacity results were plotted with time to determine if a trend in capacity gain was evident for the project. The result of this analysis is presented in Fig. 3.

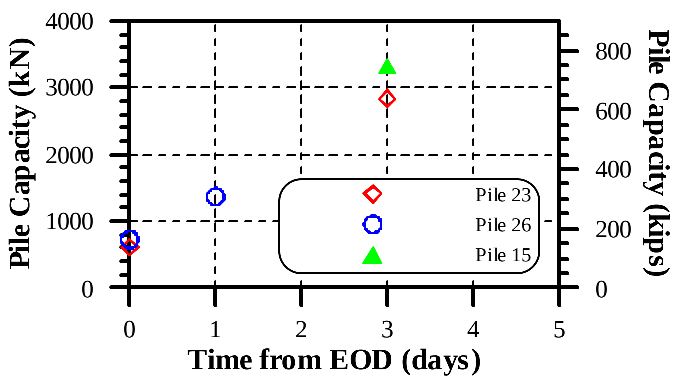

Fig. 3. Time Dependent Pile Capacity Gain Results.

A clear trend is evident in Fig. 3 showing capacity gain with time for piles 23 and 26. This is further reinforced by the results of pile 15, a batter pile similar to pile 26 with similar three day restrike (3DR) results. Due to lack of final capacity determinations for the tested piles, normalized rates of pile capacity gain (i.e. C_{gt}) as defined by Paikowsky et al. (1995) could not be determined. In addition, the lack of numerous restrikes on individual piles prevented analysis of the time dependent time capacity gain using the normalizing Q/Q_o vs. t/t_o method presented by Skov and Denver (1988). Although the lack of data prevented further analysis of time dependent pile capacity gain, the results showed that capacity gain over time occurred over the course of testing.

PILE PLUGGING MEASUREMENTS

Open ended pipe (OEP) piles act as low volume displacement piles when the interior of the pile fills with soil during installation. When the interior soil mass (i.e. "soil plug") within the pile develops sufficient frictional resistance to prevent an additional soil intrusion, the pile becomes "plugged" (Paikowsky et al., 1989). At this point, the pile acts as a displacement pile.

Plugging of a CEP pile can have significant effects on pile design and installation. These effects range from ultimate axial capacity to time dependent pile capacity gain. For example, Paikowsky and Whitman (1990) showed that within clays, plugged pipe piles have significant increases in the time required to dissipate excess pore water pressures generated by pile installation over non-plugged piles. This increase affects the time required to achieve capacity gain within time.

Pile plugging measurements are typically quantified by the plug length ratio (PLR) used by Kindel (1977). The PLR is defined as the length of the soil column inside the pile over the total pile penetration. Figure 3 presents the PLR in graphical form.

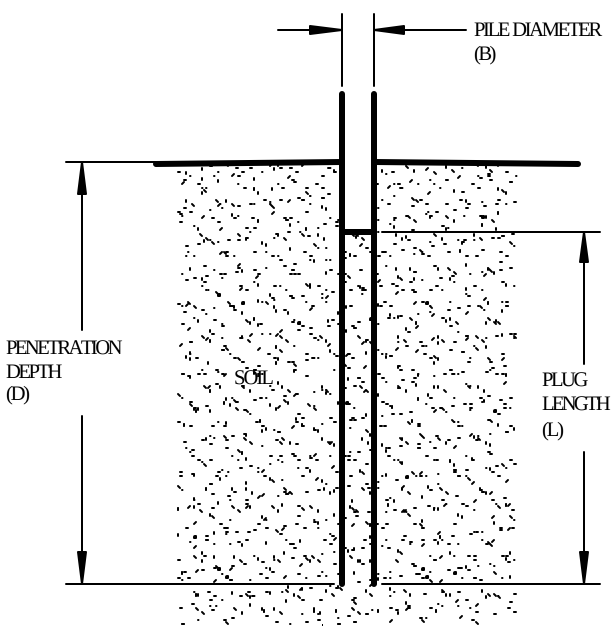

Fig. 3. Pile Plugging Schematic.

Pile plugging measurements were taken on twenty two (22) of the twenty eight (28) steel OEP piles installed for the project. Space and time restrictions during the project prevent pile plugging measurements within the remaining six (6) piles. A summary of the pile plugging measurements is presented in Table 4. Plug length ratio (PLR) vs. normalized maximum pile penetration (i.e. D_{max}/B) is presented in Figure 4.

Table 4. Pile Plugging Measurements.

Pile	D/B[1]	PLR[2]
1	27.6	1.00
2	22.2	0.99
3	21.7	0.96
4	19.4	1.01
7	17.8	1.00
8	22.0	1.03
9	19.4	0.97
10	17.9	1.03
11	20.0	0.97
12	24.0	1.09
13	18.2	1.00
14	20.4	1.03
15	20.0	0.93
16	16.7	0.98
17	19.9	1.02
19	18.2	1.02
20	19.6	1.02
21	18.1	1.03
22	18.6	0.97
23	29.2	1.01
25	18.1	1.00
27	18.1	1.07
	Average:	1.01 ± 0.04

NOTES:
1. D = Pile Embedment, B = Pile Diameter (see Fig. 3).
2. PLR = Pile Length Ratio .

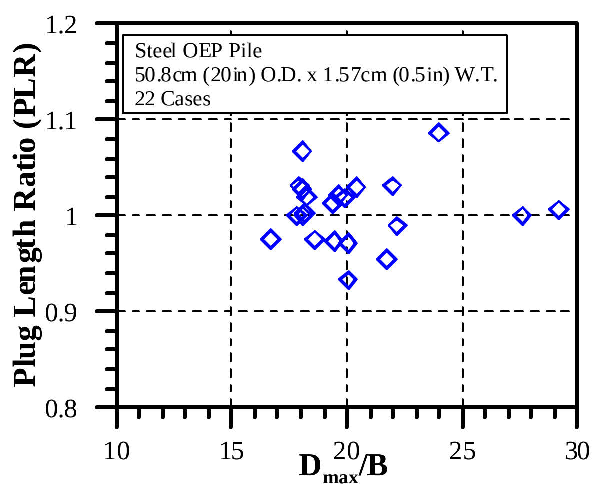

Fig. 4. PLR vs. Normalized Maximum Penetration.

As shown by the results presented in Table 4 and Fig. 4., the piles did not plug during installation. Comparison of the pile plugging measurements from this project to other data within cohesive soils showed an excellent correlation (see Table 5.)

Table 5. Comparison of Pile Plugging Measurements with Other Projects.

Reference	Pile Type	Soil Type	PLR		
			# Cases	Ave.	Std Dev.
Current Paper	50.8cm (20in) O.D. Steel OEP Pipe	Limestone & ML-MH Silt (Pee Dee Formation)	22	1.01	0.04
Paikowsky et al. (1989)	0.76m to 1.22m (30in to 48in) Steel OEP	Clay	60	0.92	NA[2]
NWS, North Charleston[1]	61cm (24in) Steel OEP Pipe	Cooper Marl Formation (ML-MH Silt)	3	1.00	NA

NOTES:
1. Naval Weapons Station mooring dolphin installed along the Cooper River in North Charleston, SC. Recent project of the Authors.
2. NA = Not Available.

SUMMARY AND CONCLUSIONS

Steel open ended pipe (OEP) piles were successfully designed, installed, and dynamically tested for a new pier extension in Georgetown, SC. The results of the dynamic test program showed that the piles had the required compressive and tensile capacities. Pile driving criteria set from the pile design, dynamic testing results, and wave equation analyses verified that the production piles had the required capacities. Verification dynamic testing of production piles that did not meet the pile driving criteria confirmed the required axial capacities were obtained.

Time dependent pile capacity gain (i.e. "setup" or "freeze") was observed within the tested piles. The limited time dependent pile capacity gain data did not allow for a detailed analysis of the phenomenon for this project. Future research into the phenomena of time dependent capacity gain within the Pee Dee Formation is recommended.

Pile plugging measurements taken from twenty two (22) of the twenty eight (28) production piles for the project showed that the piles did not plug during installation. Further analysis of the pile plugging measurements showed excellent correlation with similar soil types and pile sizes.

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