Quantifying the Influence of Construction Parameters on Hollow-Bar Micropiles Pullout Capacity in Sandy Soil

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ABSTRACT

The influences of construction parameters on the overall axial capacity of hollow-bar micropiles in sandy soils were quantified based on results from full-scale field testing. Eight micropiles were constructed at a site in the Outer Banks of North Carolina in two phases. The field testing program included installation of micropiles to a depth of 25 feet while varying the drilling or drill-bit insertion rate (I_R) and the grout flow rate (Q_R). In addition, the installation methods included micropiles that were continuously drilled and grouted with neat grout water-cement ratio (w/c) of 0.4, and others that were first drilled and grouted continuously with thinner grout (w/c-0.7) and then flushed from bottom to top with thicker grout (w/c-0.4). Eight micropile installations were designated as Fast/Fast (w/c-0.4), Fast/Slow (w/c-0.4), Slow/Fast (w/c-0.4), Slow/Slow (w/c-0.4), Fast/Fast (w/c-0.7/0.4), Fast/Slow (w/c-0.7/0.4), Slow/Fast (w/c-0.7/0.4), and Slow/Slow (w/c-0.7/0.4). The results of pullout field load testing are presented in terms of load-displacement curves and load transfer mechanism. The load-testing results showed an appreciable high ultimate pullout capacity by using slow drilling and slow grout pumping rate (w/c-0.4) as compared to the other three approaches. The result showed that an additional 50% pullout capacity could be achieved by using Slow/Slow construction approach with w/c-0.4 grout, as compared to commonly practiced (Fast/Fast) installation approach.

KEY WORDS: flow rate, hollow-bar micropiles, insertion rate, pullout capacity.

INTRODUCTION

North Carolina's Energy Efficiency Portfolio Standards, established in 2007, states that investor-owned electric utilities in North Carolina (NC) need to meet 12.5% of retail electricity sales through renewable energy resources by 2021 (American Wind Energy Association, www.awea.org). Offshore waves and currents are ocean-based renewable resources that have the potential to supplement the increasing energy needs with the deployment of Marine Hydrokinetics (MHK) arrays. These devices will require anchoring points to sustain loading from wind, waves, and current as well as accommodate the cyclic nature of some of these loading spectra, as described in API RP 2SK (2005). Micropiles offer an option for anchoring spread and single point mooring systems in marine applications (Meggitt et al. 2013). Sound & Sea Technology, Inc, Geomarine, Ltd. (now Art of UTEC) and Cellula Robotics, Ltd. developed automated grout injection system that can remotely be used in the marine environment to install micropiles. While micropiles have been utilized in practice, for example as earthquake foundation systems (e.g. Otani and Hoshiya, 2007), their utilization in the offshore oil and gas industry is not documented in literature. This is mainly due to their relatively lower holding capacity in comparison to the traditional anchoring systems of suctions caissons and driven piles. Given the lower magnitude of loading imposed by MHK devices, in comparison to an offshore oil and gas platform, micropiles have the potential of being cost-effective alternative as anchoring points and offer a redundancy factor (as these are installed in groups) that can be attractive from the perspective of managing failure modes.

Micropiles are foundation elements that are typically less than 12 inches in diameter and are constructed by placing a steel member into the subsurface profile (generally a pipe section or bar), and then using Portland cement grout to surround the steel element, and forming the pile. Cement grout is a lean material composed mainly of water and cement. According to FHWA (2005), the geotechnical capacity of a micropile in tension is equal to the capacity in compression because of higher pile circumference surface area compared to the bearing area. Therefore, it is important to assess the load capacity of micropiles especially when the main mode of loading is pullout as in the case of loading imposed by several of the MHK arrays.

A large scale field testing program was conducted to investigate the influence of construction approaches, in the form of insertion and grout flow rate, on the ultimate pullout capacity of micropiles. The field testing program included installation of micropiles to a depth of 25 ft. (7.62 m) while varying the drilling or drill-bit insertion rate (I_R) and the grout flow rate (Q_R). Eight micropiles installation approaches were designated as *Fast/Fast (w/c-0.4)*, *Fast/Slow (w/c-0.4), Slow/Fast (w/c-0.4), Slow/Slow (w/c-0.4), Fast/Fast (w/c-0.7/0.4)*, *Fast/Slow (w/c-0.7/0.4), Slow/Fast (w/c-0.7/0.4) and Slow/Slow (w/c-0.7/0.4)*. The results of pullout field load testing are presented and compared in view of the construction approach.

SITE CONDITION

The test site is located at the south-west corner of the UNC Coastal Studies Institute facility building, which is 70 feet away from Croatan Sound seashore. Soil parameters were obtained from the nearest boreholes (B-1 and B-2) of the Geotechnical Report No. 1-09-0563-EA by GeoTechnologies, Inc (2009). Boreholes, B-1 and B-2, show that the soil profile is composed of silty to fine medium sand (SM) with shell fragments. The top two feet of medium dense sand is underlain by a layer of dense sand that extends to a depth of seven feet. Alternating layers of medium dense and loose sand extend below the depth of seven feet. The Soil profile is summarized in Table 1. The effective friction angle (ϕ'), is interpreted from measured Standard Penetration (SPT) N-value using correlations between ϕ' and $N_{1,60}$ (Anderson et. al 2003), expressed as follows:

$$\varphi' = 53.881 - 27.6034 e^{-0.0147 * N_{1,60}} \tag{1}$$

$$N_{1,60} = N_{60} * \sqrt{\left(\frac{2000}{\sigma'_{z0}}\right)} \tag{2}$$

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60} \tag{3}$$

where,

σ_{z0}' = Vertical overburden stress in psf

N = the measured Standard Penetration Test number (blow/foot)

E_m = Energy Ratio; C_B, C_S and C_R are the correction factors for the borehole diameter, sampler and rod length.

Table 1. Summary of soil properties based on boreholes B-1 and B-2

Soil Profile, (ft.)	SPT-N_{field}	Dr (%)	γ_{unsat} (pcf)	$\gamma_t = \gamma_{sat}$ (pcf)	φ' (°)	OCR	Soil Type (USCS)
0 – 2.5	11	44	110	121	31	12.4	Medium dense sand (SP)
2.5-7.5	44	71	115	127	40	9.0	Dense sand (SP)
7.5-12.5	24	50	110	115	33	4.6	Medium dense sand (SP)
12.5-17.5	9	35	105	110	30	2.5	Loose sand (SP)
17.5-25	26	49	115	115	34	3.0	Medium dense sand (SP)

MICROPILES AND FIELD TESTING

Large-scale field testing was performed at the UNC CSI facility in two phases (1st phase: pile installation, and 2nd phase: pull-out test performance) from May 15 to June 5, 2017. The field-testing program included installing eight hollow-bar micropiles to a depth of 25 ft. (7.6 m) by using two different installation methods. Additionally, two pilot micropiles were installed to establish fastest and slowest rate of drill bit penetration and grout flow, respectively. The eight micropiles are categorized according to their installation as *Fast/Fast (w/c-0.4)*, *Fast/Slow (w/c-0.4), Slow/Fast (w/c-0.4), Slow/Slow (w/c-0.4), Fast/Fast (w/c-0.7/0.4)*, *Fast/Slow (w/c-0.7/0.4), Slow/Fast (w/c-0.7/0.4) and Slow/Slow (w/c-0.7/0.4)*. The installation methods included micropiles that were continuously drilled and grouted with neat grout water-cement ratio (w/c) of 0.4, and others that were first drilled and grouted continuously with thinner grout (w/c-0.7) and then flushed from bottom to top with thicker grout (w/c-0.4). Hollow-bar micropiles were only considered in this field study to advance the already Sound & Sea Technology, Inc, Geomarine, Ltd. (now Art of UTEC) and Cellula Robotics, Ltd. developed automated grout injection system that can remotely be used in the marine environment to install micropiles. The outer and inner diameter of the IBO-Titan hollow-bars were 2.88 and 1.77 inches, respectively. A sacrificial drill-bit of 8 inches in diameter was used for each micropiles. A 280 litres capacity CX10/10 colloidal grout mixer and a 8.1 metric tonnes of KR 802-1 hydraulic drill rig were utilized to install all hollow-bar micropiles.

Collected field data included pile-insertion and grout-flow rates per foot of pile length during installation and axial load-displacement data from pull-out tests. The pullout tests were performed according to ASTM D3689 seven days after micropile installation. A load cell was used to measure the pull-out force as a function of displacement at the top of each test micropile. The layout of the testing scheme is shown in Fig. 1 along with the configuration used for the pullout testing (Fig. 2). Pullout loading was applied until failure occurred or the peak load could not be maintained due to large increase in displacement rate. Table 2 summarizes the construction parameters including grout volume and insertion rates.

ANALYSES OF THE RESULTS

Several interpretations of the failure load can potentially be applied to evaluate ultimate capacity from load-displacement curves. However, not all of these criteria are applicable to micropiles. The three most commonly used criteria to estimate ultimate load capacity of micropiles are: (a) Davisson Offset Limit (Davisson, 1972), (b) Fuller and Hoy's Method (Fuller, & Hoy, 1970) and (c) Butler and Hoy Method (Butler, & Hoy, 1977).

Figure 1. Field deployment of ten micropiles

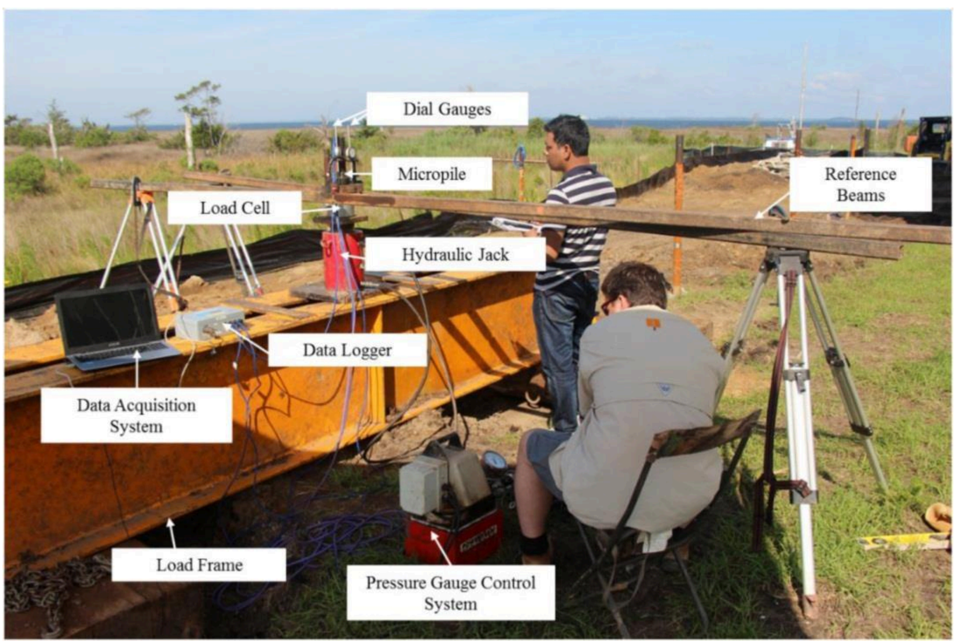

Figure 2. Set-up for pullout testing

As shown in Fig. 3, Fuller and Hoy's method provides the estimate of the failure point that is closest to the measured plunging load of the test data. However, the failure loads derived from Davisson method are the most conservative as the Davisson method underpredicts the measured failure load. Fig. 3 shows the interpretation procedure and results for MP-1.

Table 2: Pullout capacity values of eight Micropiles - different construction approaches

Micropile No.	Micropile Designation	Insertion Rate, I_R ft/min	Grout Flow Rate, Q_R ft^3/min	Drill Time min	Grout Vol. V ft^3	Avg. Pump Pressure, psi	Pullout Capacity P_{ult} (Kip)
MP1	Fast/Fast (I_R/Q_R)[1], w/c[2] = 0.4	8.0	4.3	3	13	110	89
MP5	Fast/Fast (I_R/Q_R), w/c = 0.7/0.4	13.0	4.5	2	26	100	94
MP2	Fast/Slow (I_R/Q_R), w/c = 0.4	3.5	2.38	7	62	115	155
MP6	Fast/Slow (I_R/Q_R), w/c = 0.7/0.4	4.0	2.73	6	16	100	101
MP3	Slow/Fast (I_R/Q_R), w/c = 0.4	1.6	3.9	16	30	115	119
MP7	Slow/Fast (I_R/Q_R), w/c = 0.7/0.4	1.9	3.3	13	25	100	107
MP4	Slow/Slow (I_R/Q_R), w/c=0.4	2.4	2.4	10	45	120	132
MP8	Slow/Slow (I_R/Q_R), w/c=0.7/0.4	2.3	2.7	11	44	110	120

[1]Insertion Rate/Flow Rate (I_R/Q_R).
[2]Water-Cement Ratio (w/c)

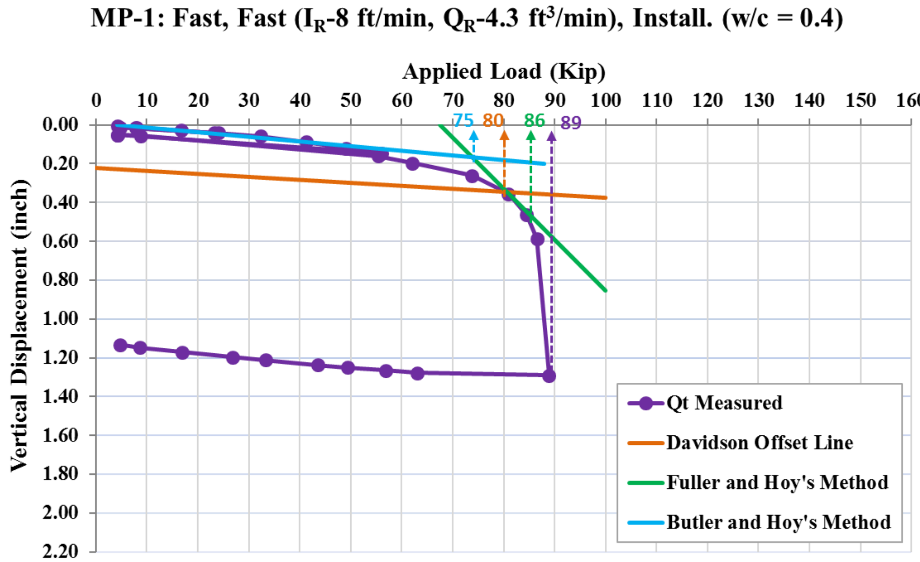

Figure 3. Ultimate uplift capacity evaluation from commonly using interpreted failure criteria: MP-1

The influence of construction approach in the form of insertion and grout flow rates was investigated in relation to the ultimate pullout capacity of micropiles. Based on water cement ratio (w/c) of 0.4 for the formation of the test piles, nearly 50% additional pullout capacity was achieved by using slow/slow (insertion rate/flow rate) method compare to fast/fast method (Fig. 4). It was determined that the total drilling time of micropile installation increased by a factor that ranged from 3 to 4 to achieve the additional 50% increase in capacity. However, the capacity can result in the reduction of the number of micropiles for a specific structure load, resulting in shorter total duration of construction. The pullout capacity of MP-2 (Fast/Slow, w/c = 0.4) is shown higher than all other micropiles due to the presence of an unexpected cavity inside the drilled hole. The grout filling the cavity induced larger diameter of the micropile (i.e. enlarged bulb) and provided additional pullout capacity.

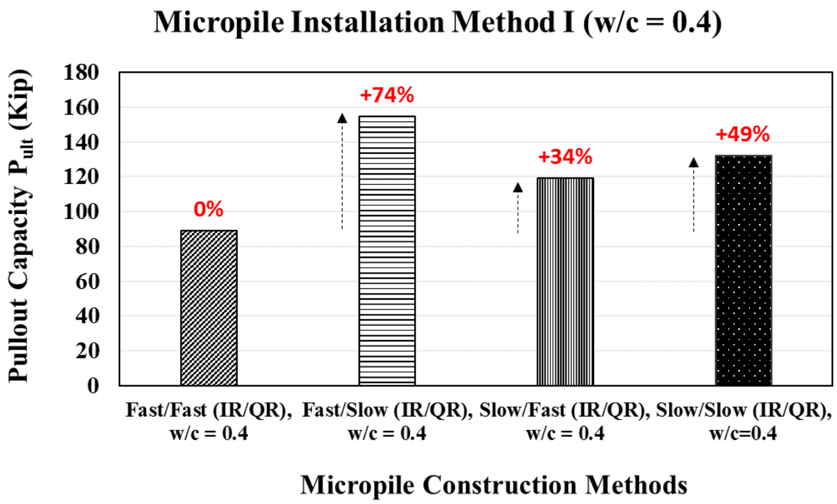

Figure 4. Influence of construction methods on pullout capacity for micropiles drilled and grouted at 0.4 w/c

Similar effect of construction methods (insertion and grout flow rates) was observed when micropiles were first drilled using lower density grout (w/c = 0.7), and then flushed with higher density or neat grout (w/c = 0.4) from the bottom up. The pullout capacity is found to increase as the insertion and grout flow rates are decreased (Fig. 5). One of the reasons could be the formation of a medium density grout (the mixing of w/c ratios of 0.4 and 0.7 leads to w/c ratio higher than 0.4).

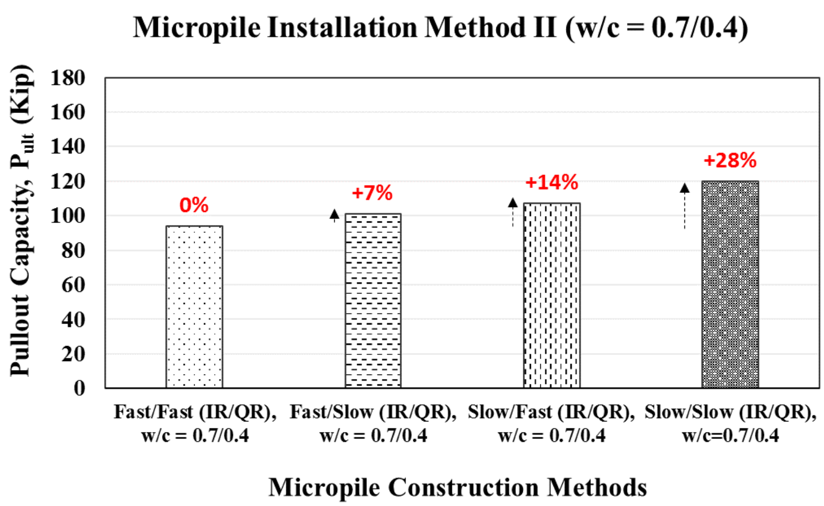

Figure 5. Influence of construction methods on pullout capacity for micropiles drilled and grouted at 0.7, and flushed at 0.4 w/c

In summary, it is found that MP-4 had the highest pullout capacity of all piles tested. In constructing MP-4, the slow/slow (I_R/Q_R) and w/c-0.4 construction approached were used. Flushing installation method seems to reduce the pullout capacity of micropiles as much as 30% due to slower insertion and grout flow rates. However, MP-5 or fast/fast (I_R/Q_R) and w/c-0.7/0.4 (as shown in Fig. 5) micropile construction showed 6% increase in the pullout capacity compared to MP-1 (fast/fast, w/c-0.4; shown in Fig. 4). Based on data observed from the field testing, it is recommended to use slow/slow (I_R/Q_R) and w/c-0.4 construction method to achieve the optimum pullout capacity of hollow core micropiles. The influence of installation method (w/c-0.4 and w/c-0.7/0.4) on ultimate pullout capacity is summarized in Fig. 6.

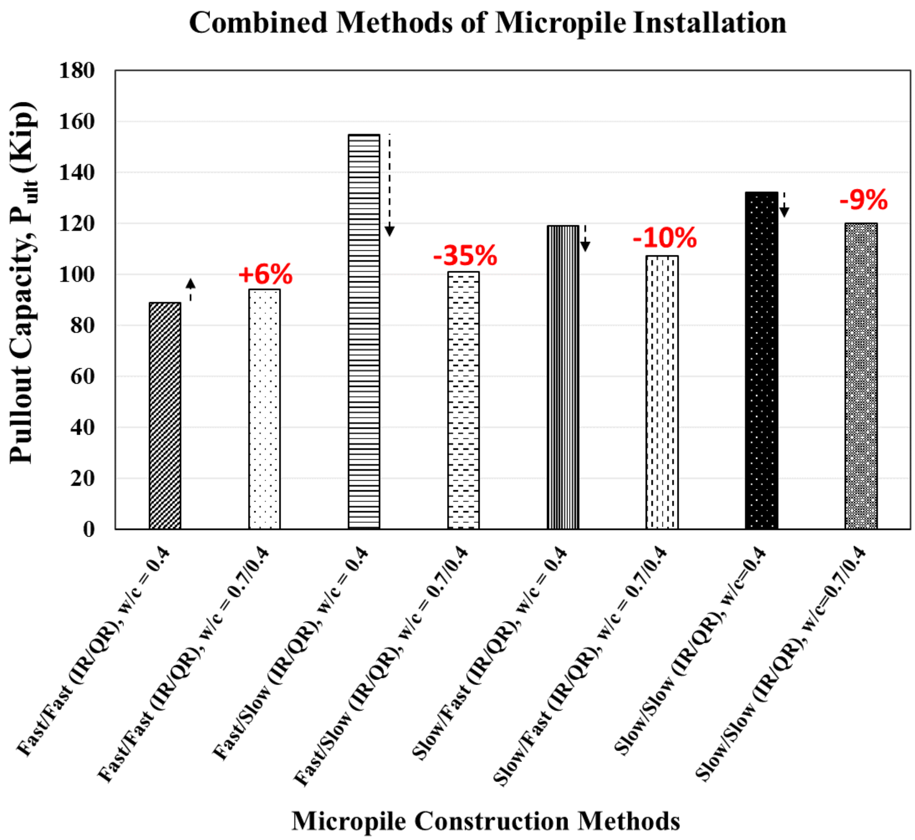

Figure 6. Influence of two installation methods, with and without flush, in developing higher strength hollow core bar micropiles

CONCLUSIONS

Based on the results presented in this study, it seems that the construction methods (I_R and Q_R) of hollow-bar micropiles play a significant role in developing side-shear bonding along the pile surface and surrounding soil. Cement grout of lower water-cement ratio improved the bonding strength of pile-soil interface and the greater erosion of the sand (verified from measured diameter of the retrieved micropiles) and as a result, the formation of annulus of micropiles. The pullout capacity of micropiles is dependent on the side-shear resistance of the surrounding soil. Therefore, the larger the annulus and the higher the side shear bonding, the higher the pullout capacity of the micropiles. The following conclusions are drawn from the data presented herein:

i. Micropiles constructed at slow insertion rate (2 ft/min) and slow pumping flow rate (2.4 ft^3/min) showed up to 50% additional uplift capacity compared with construction with fast insertion (8 ft/min) and fast flow rate (4.3 ft^3/min).

ii. Micropiles constructed using thicker grout (w/c-0.4) via single drilling and grouting system showed better performance than those drilled and grouted first with lighter grout (w/c-0.7) then flushed with w/c-0.4 grout from bottom.
iii. Fuller and Hoy's method of pile capacity interpretation from pile load test showed the closest match to measured plunging load.

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A Case History of Installation and Load Testing Challenges for Auger-Cast Piles in the Piedmont Geology

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ABSTRACT

This paper presents a case history of auger-cast pile installation and load testing for the expansion of a water reclamation facility near Atlanta, Georgia. Three load test piles were initially installed to bedrock, all of which met the refusal criteria of 0.3 meters (one foot) or less of auger penetration per minute; however, only one of the test piles achieved the test load, which is two times the design capacity of 1,100 kN (125 tons). Furthermore, extreme variability in rock depth was encountered between test piles and reaction piles near one corner of the facility. Additional investigations performed to assess the cause of test pile failure included additional borings and rock cores, pile integrity tests, and probe piles to explore the variability in rock depths and auger penetration rates across the site. These investigations revealed extreme variability in rock depth, steep rock slopes, and variable auger penetration rates in the area adjacent to the three failing load tests. Based on the results of a fourth and final load test located within the area of anomalous rock conditions, the pile design capacity for this area was reduced to 575 kN (65 tons), or 52 percent of the original design capacity. Thus, this case history demonstrates that auger-cast piles installed to rock may not always achieve the design capacity even if the standard auger refusal criteria are met. The importance of load testing and effective monitoring during pile installation is highlighted in this paper.

INTRODUCTION

A water reclamation facility located near Atlanta, Georgia had a planned expansion that included the addition of a two-story treatment building that would house a new influent pumping station near the center of the building footprint with membrane tanks located adjacent to the building. The project site is located within the Piedmont Physiographic Province of Georgia, and is underlain by Biotitic Hornblende Gneiss and Amphibolite. The site is located next to Little River and contained shallow fill underlain by alluvial soft clay and a liquefiable sand layer. A deep foundation system was recommended to avoid excessive settlement due to consolidation and/or liquefaction of the alluvial stratum. Although both driven steel H-piles and auger-cast piles were initially evaluated, driven piles were ultimately considered not suitable due to the proximity to existing structures and associated noise and vibration; thus, auger-cast piles installed to rock were recommended. Auger-cast piles with a diameter of 0.46 meters (18 inches) and an allowable capacity of 1,100 kN (125 tons) were recommended for the design of the foundations, and a refusal criterion of 0.3 meters (one foot) or less of auger penetration per minute was specified. Two of three initial load tests failed, and additional investigations consisting of SPT borings, rock coring, pile integrity tests, probe piles, and another load test were performed to assess the possible causes of pile failure and evaluate if a reduced pile capacity should be used in design. The additional investigations revealed extreme variability in subsurface conditions and ultimate pile capacities as low as 1,150 kN (130 tons) in the western portion of the site. Additional piles were added to the western portion of the site to accommodate a reduced design capacity of 575 kN (65 tons) while the eastern portion of the site remained at the original 1,100 kN (125 tons) capacity. The purpose of this paper is to present the load test results, describe the additional investigations performed to evaluate the cause of pile failure, and summarize the design solutions and lessons learned from this project.

SITE DESCRIPTION AND LOCATION

The water reclamation facility expansion footprint has a total area of about 1,580 square meters (17,000 square feet). The existing ground surface was relatively flat, and minimal grading of less than one meter (3 feet) was required at the site to achieve the first level finished floor elevation. The bottom of the influent pumping station (located near the middle of the footprint) was to be installed at approximately 7 meters (23 feet) below existing grade. A map showing the general location of the project is presented in Figure 1, and the building footprint is schematically shown in Figure 2.

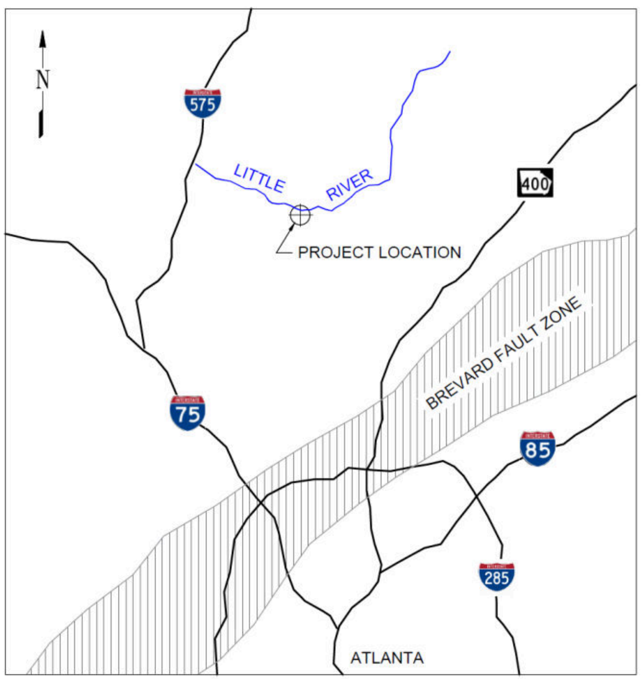

Figure 1. Project Location Map.

GEOTECHNICAL INFORMATION

Area Geology

The project site is located within the Central Uplands Group of the Piedmont Physiographic Province of Georgia, and is underlain by Biotitic Hornblende Gneiss and Amphibolite. The site is also located about 16 km (10 miles) north of the Brevard Fault Zone (see Figure 1) which is characterized by bedrock that is highly sheared and fractured. The exact structure of the Brevard Fault has been described as complex and enigmatic by geologists; however, the fault has been