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ROTARY DRIVEN PIPE PILES FOR A 14-STORY BUILDING IN NEW YORK CITY

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ABSTRACT

Rotary driven pipe piles are a unique solution for foundation construction in constrained urban areas. These piles consist of a closed-end, steel casing with sacrificial drill tip. The casing and drill tip are rotated into the ground using a fixed-mast drill rig. Three hundred sixty two 12.75-inch diameter, rotary driven pipe piles were installed to support a 14-story building in the upper east side of Manhattan. The soils consisted of uncontrolled fill, organic silts, and peat over stiff, saturated, varved silts and clays. A novel mathematical relationship between capacity, installation crowd, and torque was used to develop initial pile installation criteria. A simple discrete element model showed the piles would exhibited considerable freeze. This was verified by successive torque readings over time.

Four compression, one tension, and one lateral load test were performed. Torque measurements, load test results, and installation observations are presented. All piles performed exceptionally well during the test program in terms of total pile head deflection. Overall, field measurements matched predictions. Careful coordination and communication between all parties allowed pile installation to proceed rapidly; the foundation was completed on schedule and budget. Each pile was fitted with a geothermal conduit loop to create ‘energy piles’, which will be instrumented for future case study research..

INTRODUCTION

The Trevor Day School is located on the upper east side of Manhattan. The site is at 312-318 West 95th Street, mid-block between First and Second Avenues. The site is in a neighborhood of generally low to mid rise structures, which has seen a boom in construction including several high-rise residential towers. The property footprint is about 10,500 square feet, having an approximate 105-foot frontage along East 95th Street and about 100 feet deep. The site was formerly occupied by 5-story and 2-story brick buildings used as offices and storage by the school. These buildings were demolished in early 2012 to allow for construction of a 14-story, state-of-the-art grades 7 through 12 school. An artist rendering of the new school building is shown in Fig. 1.

The site is located in challenging geologic conditions including a buried marsh deposit, thick varved silts and clays, and depth to rock of over 200 feet. Mid and high rise structures in this neighborhood have traditionally been founded on timber piles and have experienced settlement problems causing both architectural and structural problems.

In addition to the challenging subsurface conditions, the school has an auditorium on a lower level such that most of the column loads are spread to the perimeter of the structure, resulting in very high loads. Multiple foundation options were considered for support of the structure, including deep caissons to rock (deemed cost prohibitive), ground improvement and a mat foundation (settlement performance still a concern), and micropile foundation system which was ultimately chosen as the most reasonable cost and tolerable settlement option by the owner's design team.

The foundation system was reviewed by the foundation contractor for value engineering alternatives, and an alternative rotary driven pipe pile was proposed to reduce both cost and schedule, while still achieving the same design goals as the micropile foundation. Rotary driven pipe piles are not common in the New York metropolitan area, and were an innovative solution. In addition, the piles allowed geothermal energy loops to be installed, helping the school achieve their green and state-of-the-art design goals. The design, construction, testing and performance of the piles are discussed herein.

Fig. 1. Artist Rendering of Trevor Day School

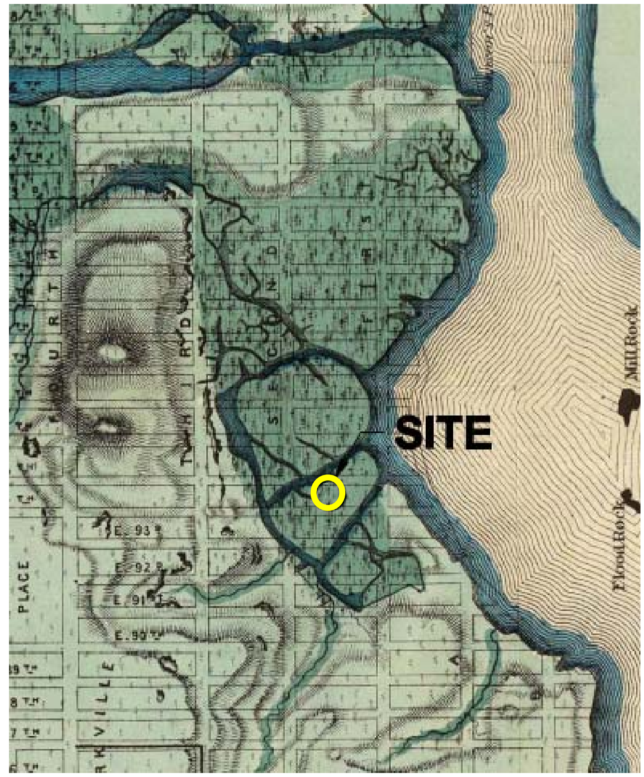

Fig. 2. Historical sanitary and topographic map showing former watercourses and marsh areas in Manhattan (from Viele, 1867)

SUBSURFACE CONDITIONS

Geotechnical engineering for this project was conducted by Langan Engineering of New York. The subsurface conditions at the site are influenced by two major geologic features: and upper marsh deposit, underlain by deeper glacial lake deposits. The upper soil conditions, to a depth of about 30 feet, consist of a former marsh deposit that was filled-in. Figure 2 is an excerpt from a historical Sanitary and Topographic Map (Viele, 1867), showing the site was part of a marsh and also was likely traversed by watercourses.

The upper soils are underlain by varved silts and clays deposited when this area of Manhattan was covered by a glacial lake. A thin layer of sand, about 5 feet thick, separates the upper marsh deposit from the lower glacial lake deposits. This sand is believed to be deposited following a breakthrough of the terminal moraine of the glacier forming a dam and the glacial lake. The upper part of the varved deposit, from about 30 to 100 feet deep, consists of varved low-plasticity silt and very fine sand, and is underlain by a deeper varved clay and silt deposit from about 100 to 170 feet deep. The varved deposits are underlain by a very dense glacial till, and finally bedrock at a depth of over 200 feet. The subsurface conditions and the uncorrected field Standard Penetration Test N-values for each layer are summarized in Fig. 3.

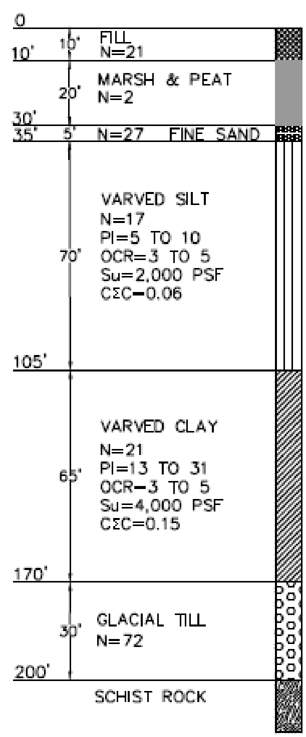

Fig. 3. Idealized Subsurface Conditions

FOUNDATION DESIGN

The foundation design for this project was prepared by Robert Silman Associates of New York based on recommendations of the geotechnical engineer. In addition to the challenging subsurface conditions described above, the foundation design was also impacted by structural and architectural demands for the school. An auditorium on the second floor required column loads be transferred to the perimeter of the site, resulting in heavily loaded columns up to about 1,300 kips. These columns were located within about 10 to 15 feet of lightly loaded columns, of about 300 kips. The foundation design required differential settlement between the heavy and lightly loaded columns be minimized.

Deep caissons to rock were considered to support the heavy column loads, but were deemed inefficient for light loads and cost and time prohibitive. Ground improvement was considered to strengthen the marsh deposit to allow a mat foundation to be used that would limit differential movement. However, the load transfer directly to the top of the varved glacial deposits was deemed to potentially result in unacceptable total settlement and therefore this option deemed not feasible by the design team.

To limit differential settlement between adjacent heavy and light columns to a tolerable amount, large pile caps were designed along the eastern and western sides of the site supported by a grid of micropiles. The pile caps were designed to be about 5 feet thick so that the columns loads could be spread relatively uniformly across the piles and reduce the impact of differing column loads over short distances. The final foundation pile layout plan is shown in Fig. 4.

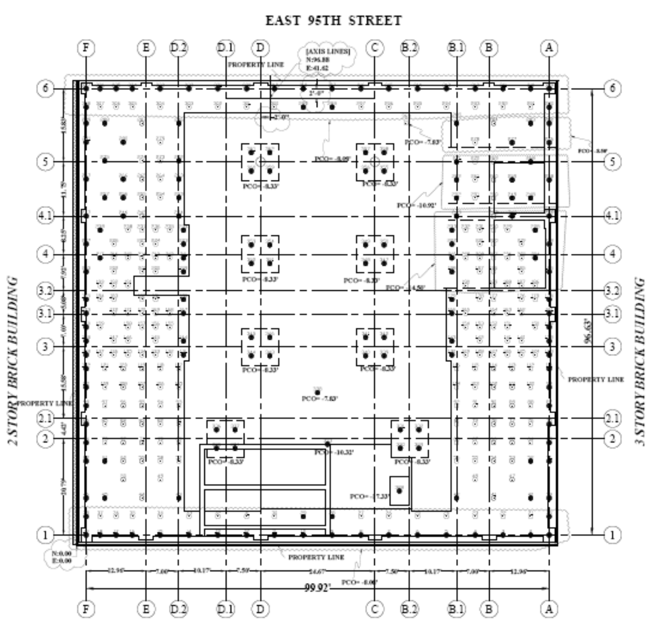

Fig. 4. Pile Location Plan

Target Pile Length

The target length of the piles was determined to reduce settlement by evaluating the load shed based on the estimated engineering parameters of the soil. Laboratory testing including consolidation tests and unconsolidated undrained triaxial tests were performed on undisturbed samples of the upper varved silt and lower varved clay. The laboratory testing indicated the upper varved silt was low plasticity, and had a higher overconsolidation ratio and lower compressibility than the underlying higher plasticity varved clays.

The length of the piles was iterated to take both strength and settlement conditions into consideration. The magnitude of loads was high enough to exceed the pre-consolidation pressures estimated from consolidation testing, resulting in virgin compression. Longer piles would result in higher capacity and fewer piles, but would transfer the load into the deeper, more compressible clays resulting in higher settlement. Shorter piles would be lower capacity requiring more piles, but transferred load primarily into the upper, less compressible, varved silts, reducing settlements.

The load spread evaluation also had to consider non-linear soil compression characteristics. If the load was shed at too shallow of a depth, the increase in stress compared to the estimated in-situ stress was in an early part of the log scale on the compression curve, resulting in higher settlement for a constant load compared to if the same load was applied to a soil with an in-situ stress on a later part of the log cycle of the compression curve. Therefore the piles were designed to find an optimal spot taking into consideration both the compression characteristics of the upper and lower varved layers, as well as the non-linear behavior of each layer.

The final pile design resulted in 50 ton allowable capacity piles, installed to a target depth of 70 feet below the marsh deposit into the bearing layer. This design resulted in a total of 362 piles for the project.

PILE DESIGN

Pile designs and installation criteria were prepared by the foundation contractors consultant, Magnum Geo-Solutions, LLC of Fort Collins, CO. Rotary driven pipe piles are a unique solution for foundation construction in constrained urban areas. The piles consist of a closed-end, steel casing with sacrificial drill tip as shown in Fig. 5. The piles are installed by rotation and crowd rather than pile driving. This avoids vibration problems associated with pile driving in close vicinity to existing structures. Rotary driven pipe piles on this project were beneficial compared to micropiles based on cost and speed of installation. This section describes the pile design.

The casing selected for this project was ASTM A500 Grade B structural steel tube with 12.75 inches diameter with 0.25 inch wall thickness. The piles were designed to penetrate through the soft soils and an additional 45 feet into a bearing stratum. The bearing stratum was assumed to be medium dense/stiff silt/sand and varved silts. The average SPT N-Value, N_{55}, of the bearing stratum was 15.6 bpf. The pile designer used a formula from Terzaghi and Peck (1967) to calculate an undrained shear strength, S_u, of 2,035 psf for the bearing stratum, as given by

$$S_u = 0.065 \text{ tsf/blow} \times N55 \qquad (1)$$

The bearing and pullout capacity of the pile was estimated using α, β, and λ methods from AASHTO 2008 with an adhesion factor, α, of 0.75, a shear ratio, β, of 0.5, and an empirical coefficient, λ, equal to 0.2. All three methods yield similar results and predict pile bearing capacity between 50 and 64 tons and pullout capacity between 43 and 49 tons. The λ method was the most conservative. Per NYC Building Code (2008), pile tip resistance was assumed negligible in all three methods.

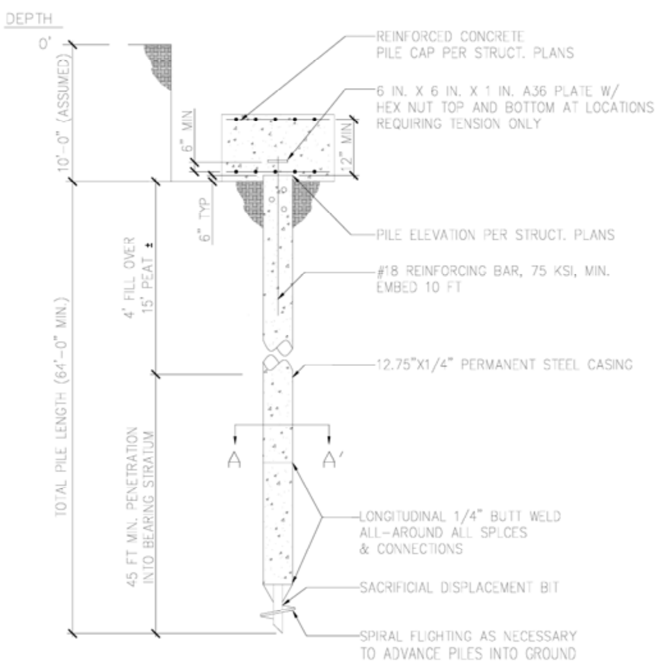

Fig. 5 Rotary Driven Pipe Pile Schematic

Pile lateral capacity was calculated using L-Pile™ Software by Ensoft, Inc. The upper soils were modeled as soft clay using pre-programmed p-y curves with buoyant unit weight of 65 pcf, cohesion of 400 psf, and strain at 50% peak strength of 0.02. The predicted lateral deflection of the pile head was 0.15 inches at the design load of 2 tons.

Much of the foregoing analysis is fairly typical of steel pipe pile design. One of the novel aspects of rotary driven pipe piles is that the torque and crowd on the casing during installation can be related to pile capacity much like hammer blows can be used for driven pile. The relationship between torque, crowd, and capacity is presented in Perko and Doner (2013), as given by

$$P_u = (2T/d_{eff}) + F \qquad (2)$$

where P_u is pile bearing capacity, T is installation torque, d_{eff} is pile casing diameter, and F is crowd. The Perko and Doner relationship was derived simply by computing the pile capacity as a function of an unknown soil-to-steel casing shear strength, determining the predicted torque during insertion using the same unknown shear strength, and then comparing the two equations. The unknown shear strength drops-out and the pile capacity is related to the installation torque.

For this project, the pile designer computed the torque that would result from the casing penetrating the upper soft soils and added this value to the torque required in the bearing stratum to yield the correct pile capacity. To be conservative in computing required torque, the crowd was set equal to zero. This resulted in a required torque equal to 120,000 ft-lbs.

The use of torque as a pile termination criteria was complicated on this project by transient pore pressures anticipated in the varved silts immediately after pile insertion. These pore water pressures would temporarily cause a decrease in effective stress, lower installation torque, and low pile capacity. Over time, these pore pressures would decrease resulting in pile freeze in terms of both higher torque and higher capacity. It was important to understand how much set-up time would be required for the piles and also the effect of inserting many closely spaced piles and the overall stress conditions in the soil at the site and under adjoining properties.

The pile designer constructed a simple discrete element model in plan view using a Microsoft Excel® spreadsheet to evaluate the effect of pile insertion on pore water pressures. Each cell in the spreadsheet was assigned a total head value and represented a 1-foot square column of soil with height equal to 70 feet (the approximate depth of the piles). A reference value of hydraulic conductivity corresponding to varved silt was assumed based on the pile designer's experience. A parametric analysis was run using different hydraulic conductivities to show that small changes within the range of common silt soils had small affect on the overall predictions based on the model.

Results of the pore water pressure model are shown graphically in Fig. 6. Initial hydrostatic boundary conditions were set at an arbitrary total head value of 40 feet as shown by the horizontal black line at the bottom of the graph at time step 2 minutes. At time step 10 minutes, the first pile on the left was "virtually" installed which caused a momentary spike in total head equal to an additional 40 feet due to displacement one pile casing volume of water. Thirty minutes later, the second pile is installed at a horizontal distance equal to 6 feet from the first pile causing another spike in total head. At this time step, the total head at the location of the first pile has already decreased by 10 feet. The model also predicts a very small increase in total head for the soil located between the two piles. Another 30 minutes later, a third pile is installed as depicted by the total head spike on the right side of the graph. Here again, the pore water pressure can be seen decreasing in the first two pile locations. After time step 240 minutes (4 hours), excess pore water pressure approximately equal to 10 feet of head remains at the three pile locations. The total head between the piles has increased by slightly less than 5 feet. After time step 1440 minutes (24 hrs), there exists excess pore water pressure equal to only a few feet of head spread across the three pile locations as represented by the pink line near the bottom of Fig. 6.

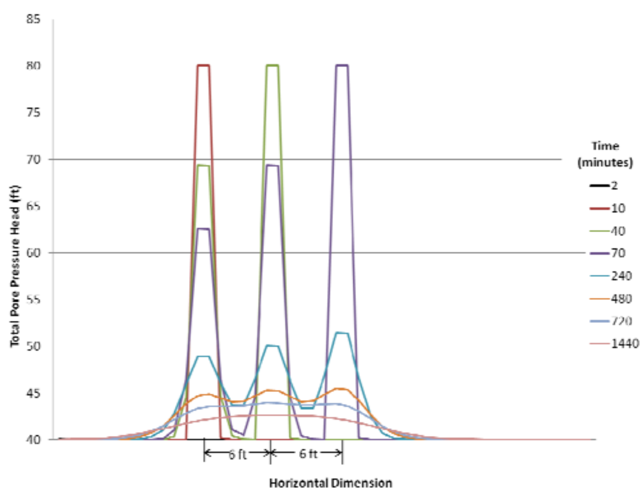

Fig. 6 Transient Pore-Pressure Analysis

The pore water pressure analysis showed that there would be some decrease in effective stress in the immediate vicinity of the piles. Complete dissipation of pore water pressures would require on the order of 1 to 2 days. However, the model also showed that the influence of excess pore water pressures would be limited to within two to three pile diameters from the pile cluster. This modeling helped alleviate concerns about pile installation on overall subgrade stability at the site and the effect of pile installation on nearby structures. It also showed that pore water pressures would affect installation torque readings and short-term pile capacity.

PILE INSTALLATION

The foundation contractor for this project was Intercoastal Foundations of New York. Rotary driven pipe piles at this site were rotated into the ground using an ABI drill rig with 90-foot tall mast and a rotary head with 140,000 ft-lb maximum torque. A photograph of the rig is shown in Fig. 7. The rig was capable of producing 40 tons of available crowd. Due to soft soils, the rig was supported on moveable timber mats. The small footprint of the site limited mobility and presented rigging challenges. Nonetheless, the foundation contractor was able to achieve high levels of production averaging 25 piles per day throughout most of the construction schedule. After pile installation, geothermal loops and reinforcing steel were inserted, and the pile casings were filled with concrete. The sacrificial drill tips prevented water and soil from infiltrating the pile casings prior to concrete placement.

Depth and torque were monitored and recorded during pile installation. Most of the piles reached a torque equal to 80% of the value predicted by the pile designer. The lower torque was attributed to excess pore water pressures. For quality assurance purposes, torque was checked periodically after installation. The procedure consisted of resetting on a representative selection of previously installed piles and turning the piles one revolution each while recording torque. In the majority of cases, the secondary torque readings achieved the pile designers target torque value. Where pile torque remained low after 24 hrs, the foundation contractor was instructed to continue "spot" checking piles. If the correct torque could not be achieved then the plan was to deepen the piles in the area in question. The requirement to increase pile depth was unnecessary.

Fig. 7 Rotary Driven Pipe Pile Installation

LOAD TEST RESULTS

Four compression, one tension, and one lateral load test were performed on the rotary driven pipe piles at this site. Tension and lateral loads tests performed well and were within project tolerances. Tension and lateral load tests will not be discussed further herein.

In general, pile load tests were conducted in accordance with New York City Building Code (2008). The results of the four compression tests are shown in Fig. 8. As can be seen in the figure, the average total pile head deflection at the design load was on the order of 1/8th inch. Pile head deflections at the required maximum text load of 200% the design load were on the order of 1/4 inch. Two of the compression load tests were conducted on sacrificial piles and the test load was continued in excess of 200% in order to evaluate pile ultimate capacity. In both cases, the load tests had to be halted due to exceeding the maximum capacity of the foundation contractors load test frame. One of the piles reached 140 tons without failure.

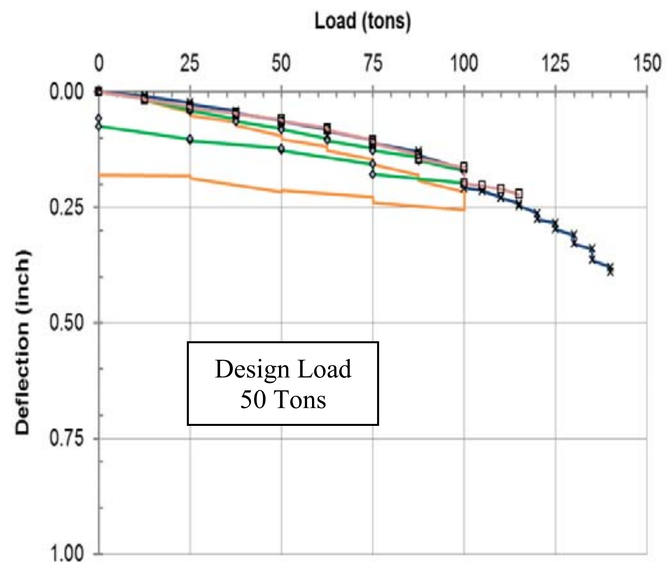

Fig. 8. Load Test Results for Four Compression Tests

Two of the compression test piles were instrumented with strain gages along the pile length. The two piles showed similar results. A graph showing the measured load transfer as a function of depth along the pile casing is shown in Fig. 9. The different color lines in this figure represent the load measured in four strain gages located at various depths at a particular load increment. As can be seen, the load transfer in the upper soft soils between 0 and 25 feet was small. Much of the pile resistance can be attributed to the upper 20 feet of the bearing stratum. The remainder of the pile resistance was shed in the remaining penetration into the bearing stratum. Less than 10% of the pile resistance at the design load of 50 tons was due to end bearing. Overall, the strain gage measurements were consistent with a friction pile in a soil profile with soft layer over more firm bearing stratum.

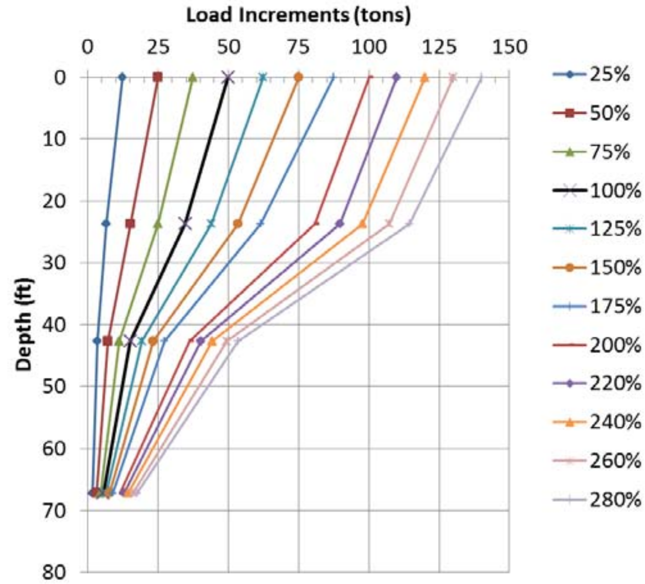

Fig. 9. Measured Load Transfer with Depth in Test Pile #1

The owner's geotechnical engineer used the strain gage data to construct a plot of mobilized skin friction within the soft soil zone (marsh layer) and at the top and bottom of the bearing stratum (bond zone). Results of these calculations are shown in Fig. 10. The skin friction in the marsh layer is represented by the dashed purple line in the figure. The break in the slope of this line followed by a flattening indicates that the ultimate skin friction in the marsh layer was exceeded at about the design load of the pile. Whereas, the skin friction in the bond zone appears to be still increasing with load even at the maximum load increment.

It is interesting to note that the calculated skin friction at the top of the bond zone shown in Fig. 10 is nearly identical to the value assumed by the pile designer using the Terzaghi and Peck relationship from 1967.

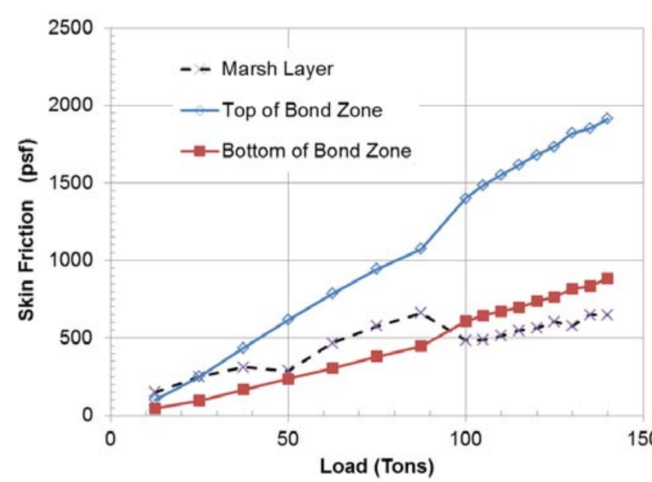

Fig. 10. Calculated Skin Friction in Test Pile #1

CONCLUSIONS AND LESSONS LEARNED

Rotary driven pipe piles are a unique solution for foundation construction in constrained urban areas. These piles are installed by rotation and crowd rather than conventional pile driving and, therefore, concerns over vibration and noise and the impact on adjacent structures are avoided.

Rotary driven pipe piles were shown in this case history to work well in a soil profile consisting of marsh deposits over varved silts. Historic relationships between undrained shear strength and SPT blow count in fine-grain soils by Terzaghi and Peck (1967) used in the pile design compared closely with load tests and strain gage measurements.

The mathematical relationship between capacity, installation crowd, and torque presented by Perko and Doner (2013) was used as a pile termination criteria and for quality assurance. The torque readings taken during pile installation compared well with this formulae.

A simple discrete element model showed the piles would exhibited considerable freeze. This was verified by successive torque readings over time. Excess pore water pressure modeling during simulated pile insertion helped to show 1 or 2 days of time would be required for pile freeze and also to show that the range of pore pressure build-up was limited to two to three pile diameters from an installed pile cluster.

The rotary driven pipe piles performed exceptionally well during the load test program in terms of total pile head deflection and capacity. Overall, field measurements matched predictions. Open cooperation between engineering and design consultants employed by the owner and specialty foundation contractor on the project portrayed in this case history allowed for a solution that maximized economy, constructability, performance, and environmental sustainability. Overall, the project was a successful demonstration of an iterative design approach with a challenging site.

FUTURE RESEARCH AND MONITORING

Trevor Day School has partnered with Virginia Tech, Langan Engineering and Geothermal International to implement a long-term research program to evaluate the geothermal energy piles. The research program is part of Virginia Tech's ongoing research into energy pile performance and design, and will provide Trevor Day School with real life data on the performance of the geothermal piles that can be used as part of the school's sustainability and green technology curriculum. Being one of only a few buildings with energy piles instrumented for long-term monitoring in the world, this research program will provide valuable data for all aspects of the project: the Owner, Consultant Engineer, Pile Designer and Geothermal System Designer.

Instrumentation was donated by Virginia Tech and GeoInstruments, and was installed in the piles by Langan and Intercoastal Foundations during pile construction. Two piles, one on each side of the site, were instrumented with sensors to measure changes in temperature and load induced in the piles by the geothermal energy loops and heating and cooling through the seasons. The instrumentation consists of nine strain gages and thermistors placed down the full length of each pile. The sensor cables were routed through the pile cap and into the basement of the building where a data logger and modem will transmit the data to the team remotely.

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