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DESIGN AND INSTALLATION OF AUGER, PRESSURE-GROUTED DISPLACEMENT PILES AT CONEY ISLAND HOSPITAL

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Abstract: Auger, pressure-grouted displacement piles are an emerging sector of the deep foundation industry in the United States. Installation of this system leads to increased shaft and toe resistance in soils where displacement leads to densification, and the resulting increase in ultimate capacity relative to other cast-in-place systems can be substantial. This paper documents the application of an auger, pressure-grouted displacement pile system at a hospital addition in Coney Island, which was the first full-scale use of such a system in Metropolitan New York City. The paper includes an overview of the site subsurface conditions and the original approach to foundation support, a description of the system used and the associated design methodology, and an examination of the results of two instrumented compression load tests. A summary discussion of the potential application of the system in the New York City area is also included.

I INTRODUCTION

The project site is within Coney Island Hospital complex located at the northeast corner of the intersection of the Belt Parkway and Ocean Parkway in the Coney Island section of Brooklyn, New York. The approximate site location is shown in Figure 1.

Figure 1: Site Location Map

Prior to construction, the majority of the proposed building footprint was occupied by various one-story buildings temporary structures and trailers that limited the areas where borings could be excavated for the purpose of subsurface investigation. The existing hospital buildings are reportedly supported on timber or pipe pile foundations with an allowable load of 30 tons.

The proposed addition consists of a 15,500 sq-ft building immediately adjacent to the main hospital buildings. The addition is to be a 7-story steel frame building with typical column loads ranging from 400 tons to 600 tons and column spacings of about 30 ft. The owner engaged Langan Engineering and Environmental Services, Inc. (Langan) to evaluate pile foundation options at the time where a schematic design had been developed assuming a 30-ton pile capacity. The structural slab was designed to span over the pile caps with no intermediate piles or grade beams. An important factor to be considered was that the surrounding hospital buildings had to remain operational during construction and therefore construction noise and vibrations had to be minimized.

II SUBSURFACE CONDITIONS

A two-phase subsurface investigation with a total of 16 borings was performed in accessible areas of the site. The subsurface conditions generally consisted of 4 to 17 ft of fill materials overlying a layer of loose to medium dense sand. The fill consisted primarily of sand with trace silt and gravel with occasional wood, brick and glass pieces. A thin layer of soft silty clay was found beneath the fill in two of the borings.

Geologically, the natural soils below the fill are Outwash Plain Sediments and are typically fine to coarse sands with traces of silt and gravel to a depth of 100 ft. In situ testing in the borings consisted of Standard Penetration Testing (SPT). In the upper 50 ft, the recorded SPT N-values in the sand ranged between 3 and 25 blows per foot (bpf) with an average of 14 bpf. Between depths of 50 ft and 100 ft, the sand gradually became denser with depth with some N-values exceeding 30 bpf. The bedrock surface in this area is estimated to be at about 1,000 ft or greater. Groundwater was encountered at a depth of 6 ft to 8 ft below grade and was slightly influenced by tidal variations. A typical subsurface profile used in design is presented in Figure 2.

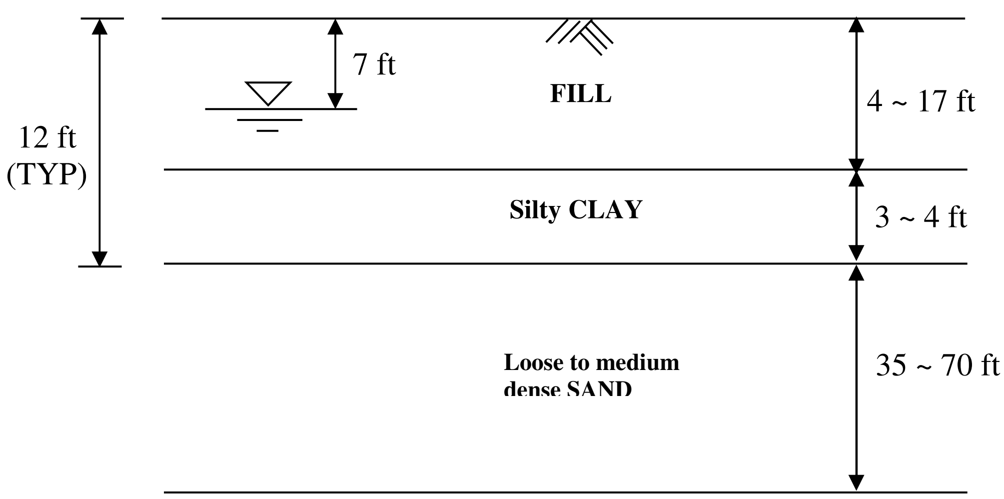

Figure 2: Typical Subsurface Profile

III FOUNDATION ALTERNATIVES

The following three deep foundation systems were originally evaluated:

- Timber piles with a working load of 30 tons (as originally specified in the schematic structural design)
- Steel Monotube piles with a working load of 80 tons
- Conventional, Augered, Pressure-Grouted (APG) piles also with a working load of 80 tons

All three systems considered were designed primarily as friction piles terminating in the natural sand stratum.

Design calculations indicated the following required depths with regard to the Monotube and APG foundations considered:

- Driven, steel 14 inch O.D. Monotube pile (J-taper) with a working load of 80 tons required a tip depth of approximately 60 ft below grade
- 16 inch diameter APG pile with working load of 80 tons required an installation depth of about 70 ft below grade (a minimum of 50 ft into the natural sand)

The use of APG piles in the New York City Metropolitan area has generally been limited to those applications where the limitation of noise and vibration were the over-riding factors. A cost analysis of the three systems was made based on the foundation layout and structural design details and also based on price quotes obtained from local foundation contractors. The analysis indicated that the Monotube system would be the most economical pile. However, the APG pile system was favored by the Owner due to the low levels of vibration and noise associated with APG pile installation. The subsurface information was reviewed by Berkel & Company Contractors, Inc. (Berkel) during their APG bid preparation as a sub-contractor for Underpinning and Foundations. Berkel felt that the conditions at the site were favorable for application of their pressure-grouted displacement (APGD) system and also submitted a displacement pile option. The APGD approach retained the desirable aspects of conventional APG piles (low noise and vibration), and could be installed at a lower cost.

IV VALUE ENGINEERING APPROACH

A. Supplemental Exploration

As discussed previously, several structures were present at the site during the original subsurface exploration. As part of their value engineering program, Berkel included post-demolition subsurface exploratory work. Cone Penetration Tests (CPT) were performed at 7 locations at the site. A composite plot of q_c versus depth from 4 representative CPTs is shown in Figure 3. The CPT data disclosed a soil profile that Berkel felt was nearly ideal for the application of the displacement system. Below fill that extends to as much as 17 feet below the ground surface, the profile in the depths explored is loose to medium-dense sand. The cone tip resistance in the upper reaches of the natural sand is about 55 tsf, and increases more or less linearly to about 40 feet, where the average q_c value is approximately 100 tsf. The q_c values tend to remain fairly constant below 45 feet.

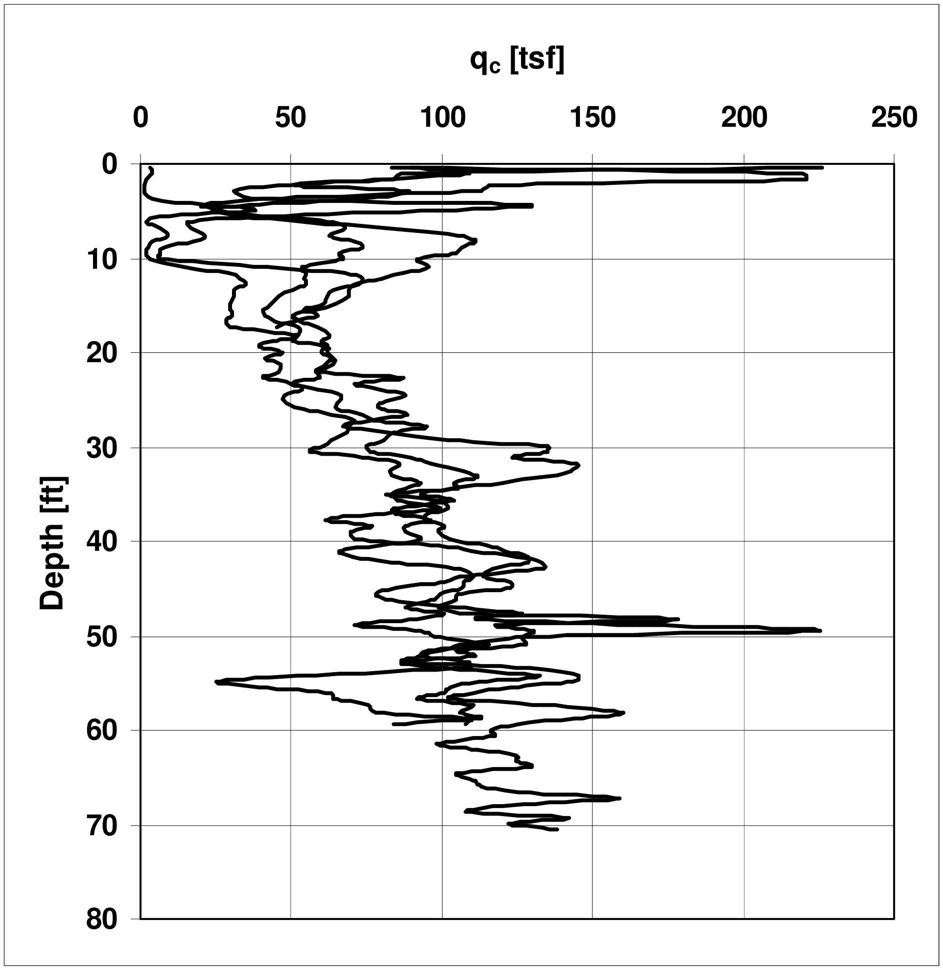

Figure 3: Composite Plot of CPT Data vs. Depth

B. <u>Displacement Pile Design</u>

During installation of APGD piles, the pile area is evacuated by a full horizontal displacement tool. The pile is then cast using fluid grout under pressure. In sands with q_c values in excess of 160 tsf, it becomes difficult to advance the tool and significant penetration into materials in excess of 200 tsf becomes extremely difficult. The profile at Coney Island Hospital allowed for a wide choice of pile diameter-depth combinations, with relatively easy installation.

The depth of fill at the site ranged from about 4 ft to 17 ft below the ground surface. Final design considered only the shaft component that developed in the natural sands below the fill along with the toe resistance component. The design methodology for capacity of APGD piles as installed by Berkel has been developed internally. A complete description of the development of the design process is described by NeSmith (2002a). Both shaft and toe resistances are calculated based on either SPT or CPT results, along with modifiers appropriate for the characteristics of the granular material penetrated.

Unit shaft resistances (f_s), based on either CPT tip resistance (q_c) or SPT blow count (N) corrected to N_{60} efficiency, are calculated as follows:

- $f_{s,cpt} = 0.01*q_c + w_s$ where $q_c \leq 200$ tsf (19 MPa)

- $f_{s,spt} = (0.05 \text{ tsf} * N) + w_s$ or
  $f_{s,spt} = (0.005 \text{ MPa} * N) + w_s$ where $N \leq 50$

For uniform, rounded materials with up to 40 percent fines, w_s is taken as zero, and a limit shaft resistance of 1.7 tsf (0.16 MPa) is applied. For well-graded, angular materials with less than 10 percent fines, w_s is taken as 0.5 tsf (0.05 MPa) with a limit shaft resistance of 2.2 tsf (0.21 MPa). Interpolation between these values is used for intermediate materials.

Unit toe resistances (q_t), based on either average CPT tip resistance (q_{cm}) or average corrected SPT blow count (N_m) is calculated as follows:

- $q_{t,cp\,t} = 0.4 * q_{cm} + w_t$ where $q_{cm} \leq 200$ tsf (19 MPa)

- $q_{t,spt} = (1.9 \text{ tsf} * N_m) + wt$ or
  $q_{t,spt} = (0.19 \text{ MPa} * N_m) + w_t$ where $N_m \leq 50$

Average CPT tip resistance or SPT blow count are calculated from four diameters above and below the pile toe. For uniform, rounded materials with up to 40 percent fines, w_t is taken as zero, and a limit toe resistance of 75 tsf (7.2 MPa) applied. For well-graded, angular materials with less than 10 percent fines, w_t is taken as 14 tsf (1.34 MPa) with a limit toe resistance of 89 tsf (8.62 MPa). Again, interpolation is used for intermediate materials. The relationships between ultimate capacity and depth for 2 representative cones from the Coney Island Hospital site are shown in Figure 4.

The design process described herein was developed by Berkel based on ASTM D 1143 load tests, many using the quick loading option. Ultimate load is taken as the lesser of the following:

(1) the load at which the gross pile head movement is equal to 6 percent of the pile diameter (0.96 inches for a 16-inch diameter pile)
(2) the load at which the displacement rate reaches 0.02 inches per ton (NeSmith, 2002b)

A factor of safety of 2 is typically applied to the sum of the shaft and toe components, since strain compatibility is implicit in the design process.

The New York City Building Code method for performing load tests and defining working load is quite different from that used elsewhere and there is not a rational process for correlating the results of these different processes. Because this was one of the early uses of the system under the New York City Building Code, a factor of safety of 2.5 was used to establish the target ultimate load. Given the working load of 80 tons, the target ultimate capacity to be generated in the natural sand was 200 tons. The resulting target test pile depth was 44 ft. At this toe level, the ultimate loads predicted by the Berkel methodology were 239 tons for Test Pile 1, and 251 tons for Test Pile 2.

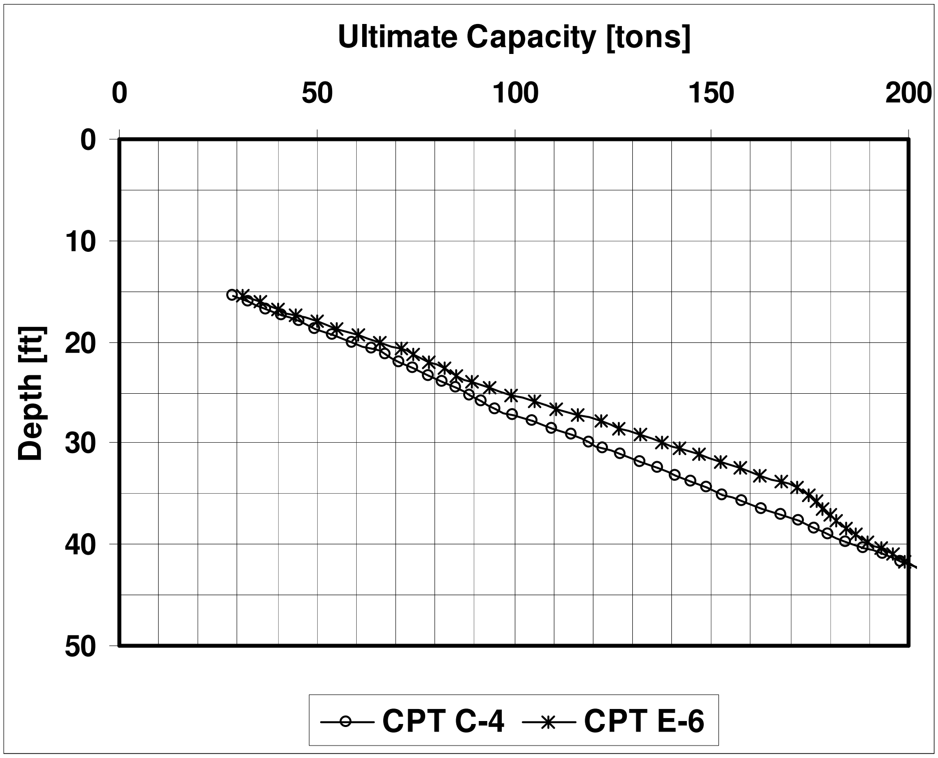

Figure 4: Ultimate Capacity vs. Depth

V LOAD TEST PROGRAM

Ten index piles were installed and two extended (96-hour) pile load tests were performed in accordance with the NYC Building Code prior to installation of production piles. Strain gages were attached to the steel reinforcement to monitor load transfer along the pile shaft. The test piles were installed to a depth of 44 ft below ground surface, which corresponded to about 30 ft to 35 ft into the bearing layer below the fill. The reinforcing steel in the test piles was the same as for the production piles and consisted of an 8.625-inch-diameter, 0.250-inch-thick steel pipe in the upper 30 ft and with 1 No.11 grade 75 bar extending the full length of the pile.

The load-displacement relationships and the associated load distribution graphs for both tests are shown on Figures 8 and 9. The load-displacement relationships include extrapolations of the test data using a hyperbolic model (Chin, 1970). The tests performed similarly, as would be expected from the subsurface conditions. Both tests “passed” relative to the requirements of the New York City Building Code. It is clear that toe resistance is just beginning to be mobilized at twice design. Key elements of the load test data are presented in Table 1.

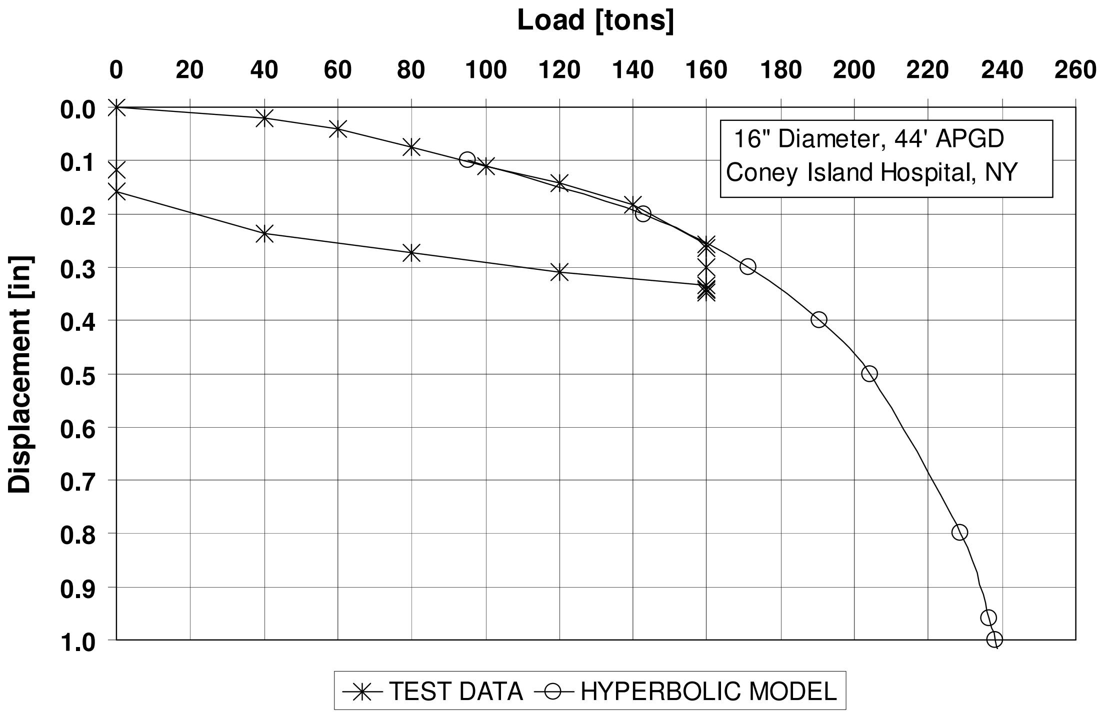

Figure 8a: Pile Load Test 1 – Load-Displacement Data

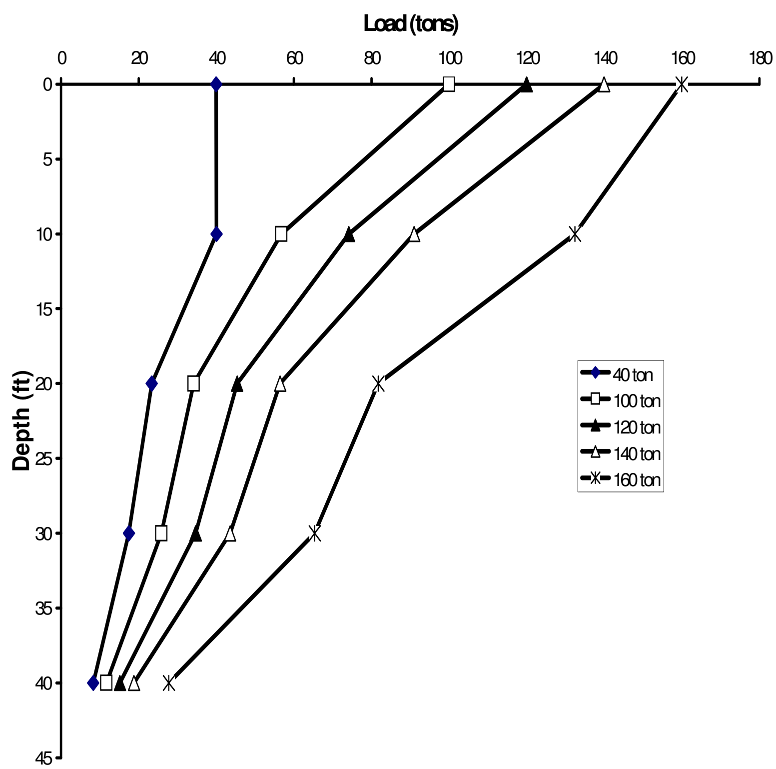

Figure 8b: Pile Load Test 1 – Load Distribution Data

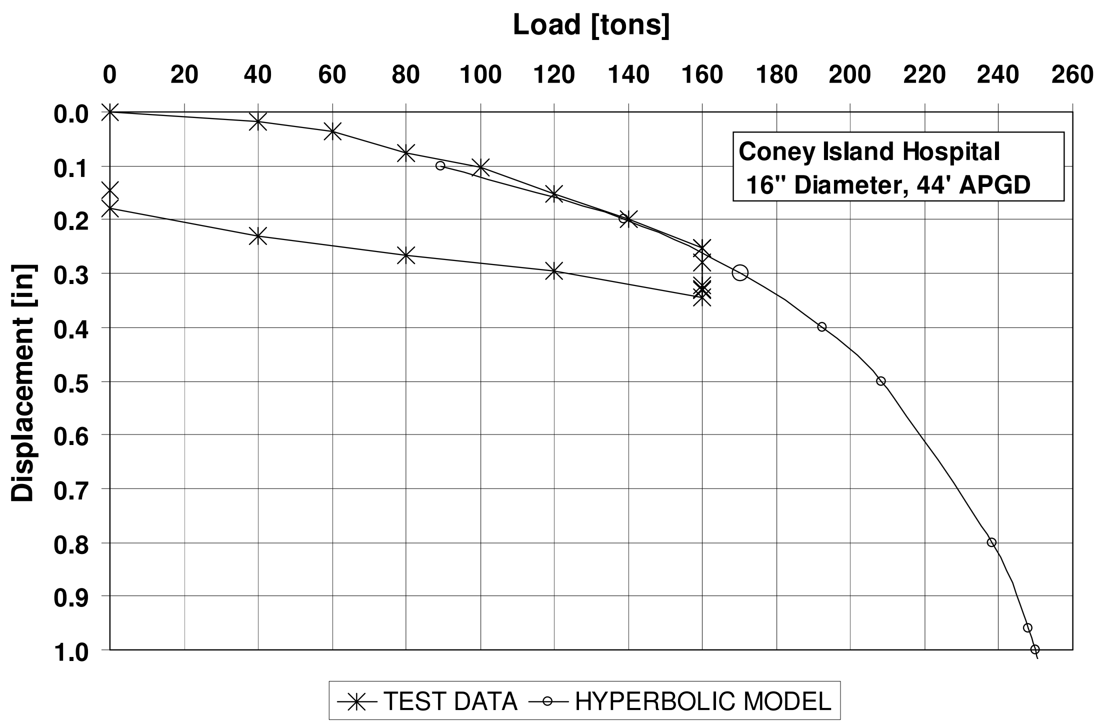

Figure 9a: Pile Load Test 2 – Load-Displacement Data

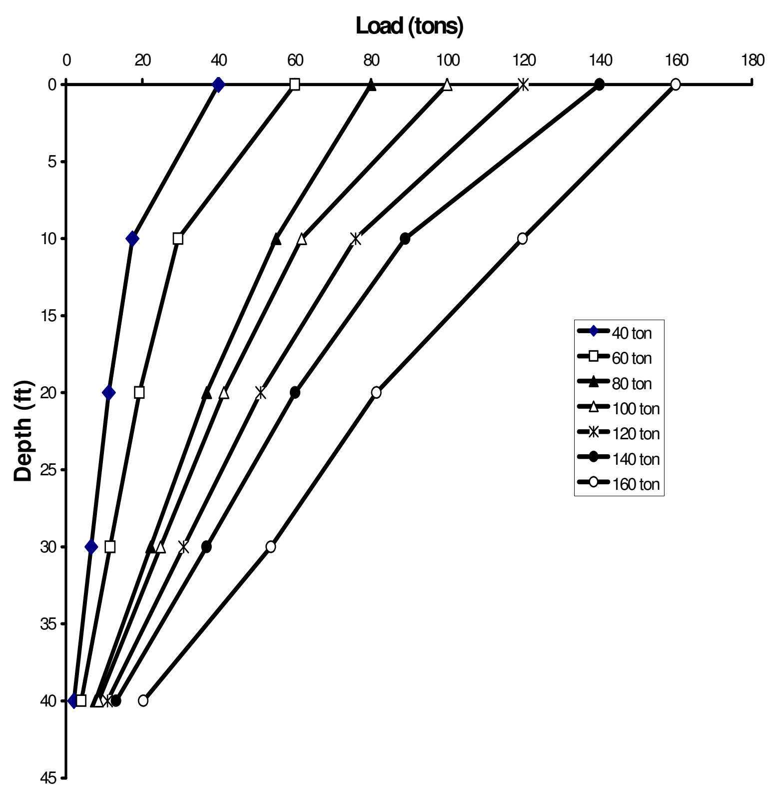

Figure 9b: Pile Load Test 2 - Load Distribution Data

Test Pile	Displacement at Design Load [in]	Displacement at 2 * Design (2hr) [in]	Displacement at 2 * Design (96hr) [in]	Net Displacement [in]	Estimated Ultimate Load [tons]
TP-1	**0.074**	**0.258**	**0.335**	**0.0118**	**236**
TP-2	**0.075**	**0.254**	**0.345**	**0.145**	**248**

Table 1: Key Elements of Load Test Data

VI PRODUCTION INSTALLATION AND QUALITY CONTROL

Figure 10 includes schematics of the Berkel Displacement Pile Tool and the pile installation platform. During installation of the APGD pile, the depth of the tip is displayed in the operator's compartment. When the desired tip level is reached, downward travel of the tool is stopped and pumping of grout is begun. Grout pressure is monitored at the top of the tool and displayed in the operator's compartment. When the target pressure has been reached, withdrawal of the tool is initiated. The withdrawal rate is varied to maintain grout pressure appropriate for the materials in which the pile is being cast. The target "lift off" pressure and a pressure range for shaft construction are set during index pile and test pile installation and a relationship between installation pressure and grout volume is established. Grout volume is checked to insure that the volume delivered is greater than the neat volume of the hole. However, the pile is cast based on pressure.

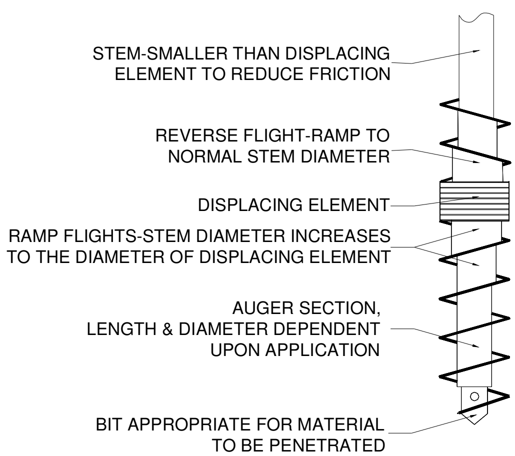

Figure 10a: Berkel Displacement Tool

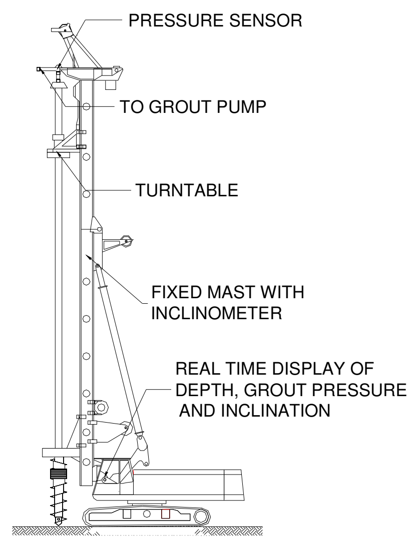

Figure 10b: Berkel Pile Installation Platform

The tool is rotated during withdrawal and any material that may have entered the annular space between the stem and the full diameter of the hole is captured by the reverse flighting and forced back into the sides of the hole. The soil-filled augers, and the displacing element, act as a packer and the grout is confined below the level of the displacing tool. Thus, grout return occurs only when the tool exits the ground, and there is not an "observed head".

For this project, grout volumes were determined manually by counting the strokes of a calibrated pump. The ratio of the pumped grout volume to the neat pile volume for the test piles was 1.20.

The installation platform used to install APGD piles includes a data acquisition system that records depth, torque (as indicated by the fluid pressure of the motors that drive the turntable) and grout pressure. Data is recorded at one-second intervals. These installation parameters are displayed in real time in the operator's compartment and on an external display for the inspector. Installation records were produced for each pile installation. The installation parameters for Test Pile 1 at this site are shown in Figure 11. Below the fill (about 15 feet at this location) the torque increases and the penetration rate decreases more or less linearly with depth, reflecting the general increase in cone penetration tip resistance.

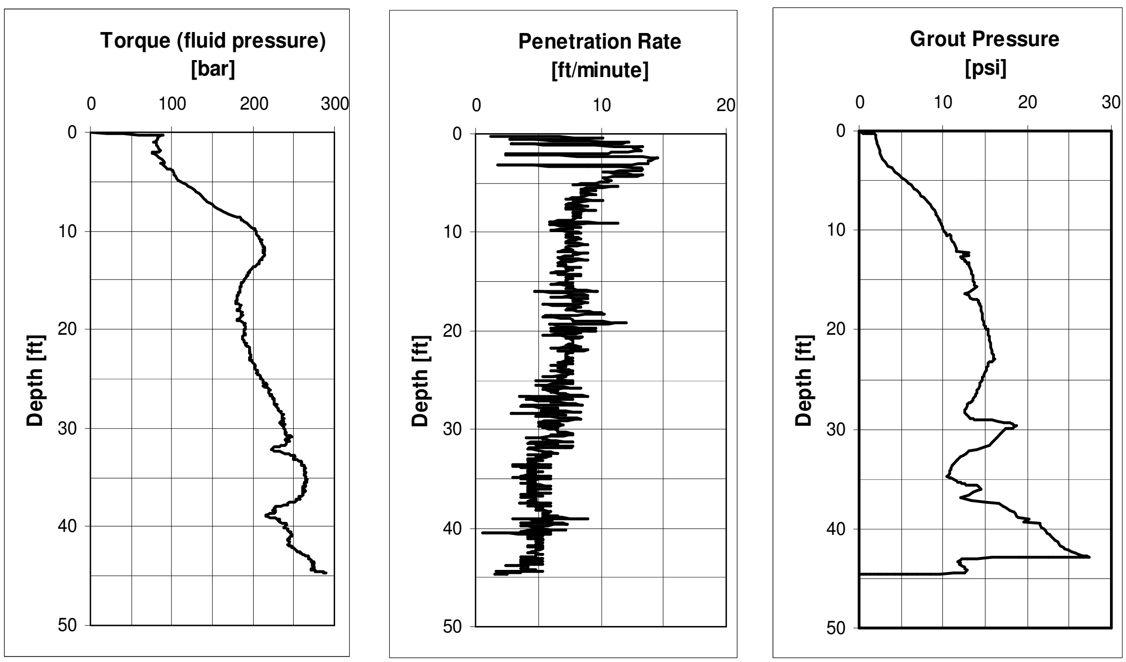

Figure 11: Recorded Pile Installation Parameters – Test Pile TP-1

VII CONCLUSIONS

A total of 226 cast-in-place displacement piles were installed at the site with no complaints about noise or vibrations and no interruption in hospital operation. The contract cost of the APGD piles was about 80 percent of that for conventional APG piles. Additionally, the spoils generated by the displacement system were less than 10 percent of those expected with conventional APG piles, which resulted in additional savings. The performance of the system with respect to generating the needed capacity was excellent, and was consistent with the design methodology.

The subsurface conditions at Coney Island Hospital are nearly ideal for the application of the APGD pile. Loose to medium dense sands such as those at this site provide for development of high shaft resistance with relatively easy installation. The presence of large scale granular deposits in Brooklyn, Long Island and parts of Staten Island indicate that application in these areas is feasible, and the system has in fact been used in Brooklyn and Long Island. The system has also been used in Manhattan and the Bronx where smaller scale granular deposits are present.

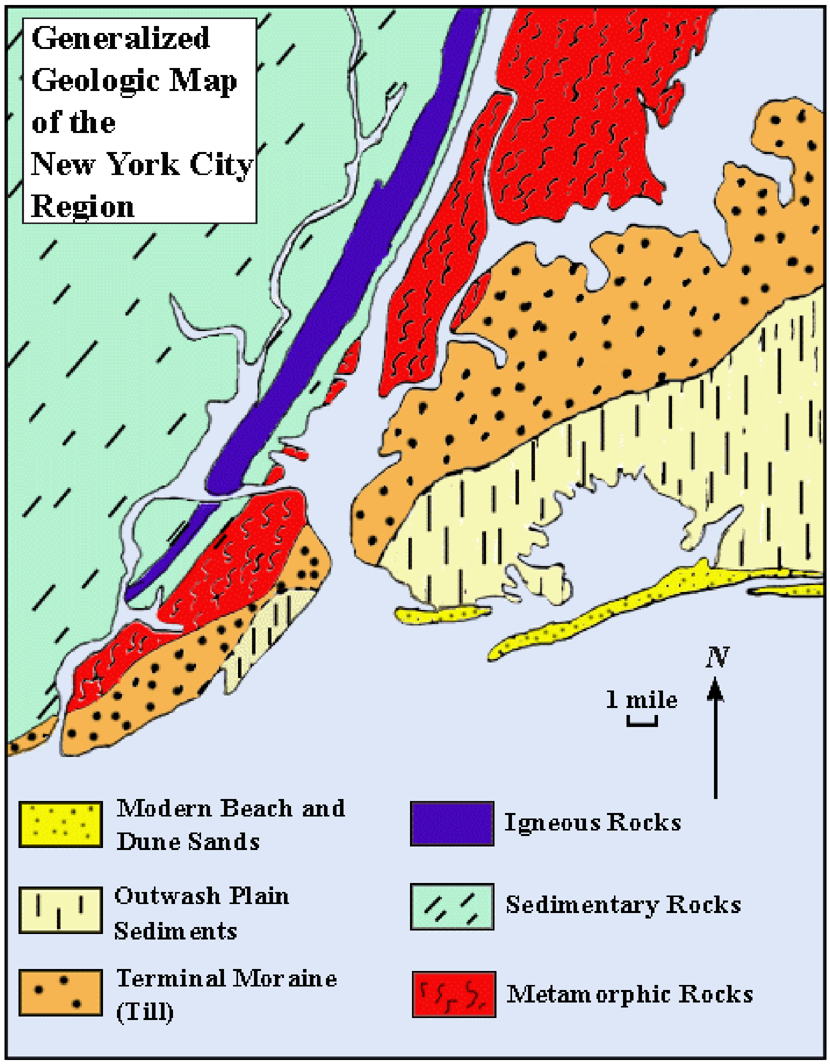

Figure 9: Generalized Geologic Map of the New York City Region

VII REFERENCES

NESMITH, W. M., 2002a. Static capacity analysis of augered, pressure-injected displacement piles. Proceedings of Deep Foundations 2003, Orlando, ASCE Geotechnical Special Publication No. 116, pp. 1174-1186.

NESMITH, W. M., 2002b. Design and installation of pressure-grouted displacement piles. Ninth International Conference on Piling and Deep Foundations, Nice, France, pp 561-567.

CHIN, F. K., 1970. “Estimation of ultimate load of piles not carried to failure”. Proceedings of the 2nd Southeast Asian Conference on Soil Engineering.