500 Walnut Street: High-Capacity Auger Pressure-Grouted Piles Used to Support a 26-Story, Multi-Family Tower Behind Independence Hall

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ABSTRACT

The purpose of this case history is to showcase the viability of high-capacity auger pressure-grouted (APG) piles as an alternative to the traditional deep foundation systems commonly used within the Philadelphia region (e.g. drilled shafts or caissons; driven piles; micro-piles). This case history demonstrates that relatively high capacities can be developed using APG piles when installed in similar geologic settings as the subject site and highlights the circumstances where APG piles may be a favorable alternative. Included is a brief summary of the 500 Walnut Street multi-family residential development, geologic setting, and process of how APG piles were selected and implemented for support of the proposed tower. Explanations are included demonstrating how the designers of the APG piles arrived at the theoretical pile capacities. Finally, this paper summarizes the results of a pile load test program used to confirm the recommended pile capacities, as well as experiences during the installation of production piles.

Project Background

The 500 Walnut Street multi-family residential tower is located southwest of the intersection of 5th Street and Walnut Street in Philadelphia, Pennsylvania, about a half-mile west of the Delaware River and historic Penn's Landing, and one block south of Independence Hall, as shown on the attached Figure 1 – Site Location Map. The long, narrow site is roughly 1,380 square meters (14,700 ft^2) in plan (about 21.6 meters (70 feet) east-west by about 64.0 meters (210 feet) north-south). City streets border the site on its north, south, and east sides. The existing 20-story Penn Mutual tower abuts the site to its west, with two basement levels extending about 7.6 meters (25 feet) below-grade.

The 500 Walnut Street tower structure is comprised of cast-in-place concrete walls, columns, beams, and slabs. The building includes two below-grade parking levels, with the deeper level extending about 7 meters (23 feet) below the surrounding street level. According to the Structural Engineer, the maximum vertical compressive column load was expected to be on the order of about 17.8 MN (4,000 kips). Similar magnitude uplift loads were anticipated, particularly at the shear walls near the core of the structure.

The total lateral resistance requirement was about 22.2 MN (5,000 kips), resulting in an allowable lateral capacity requirement of 89 kN (20 kips) per pile.

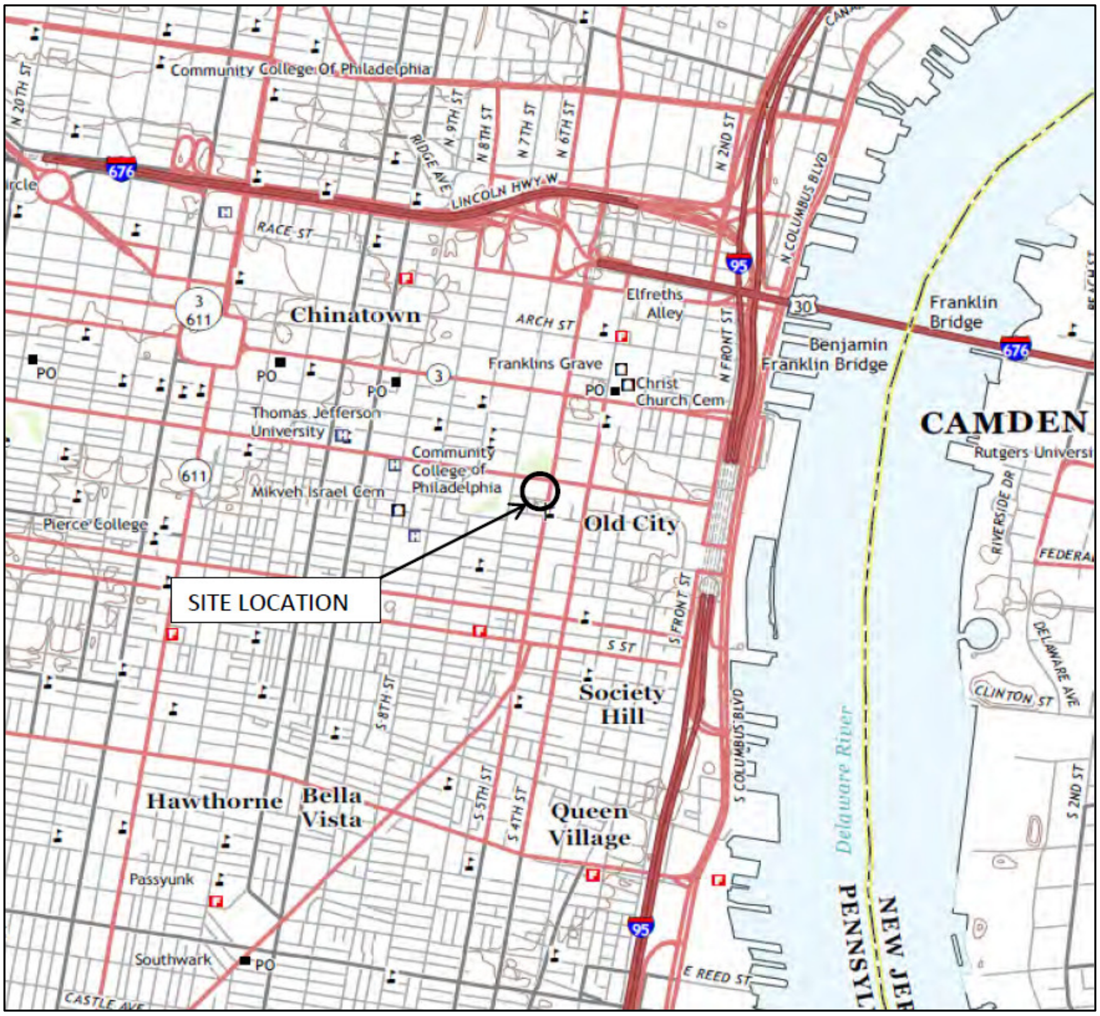

Figure 1. Project Site Location Map (USGS 2013).

Geologic Setting

The site for the 500 Walnut Street tower is located within the Lowland and Intermediate Upland Section of the Atlantic Coastal Plain Province. Locally, the site is underlain at depth by the alluvial deposits of the Trenton Gravel Formation, followed by a weathered profile (decomposed and weathered rock, grading to intact mica schist bedrock) of the Wissahickon Formation.

While researching the site geology and related history, the Geotechnical Engineer identified the site as being situated above an historic branch of the former Dock Creek. According to published documents obtained from the American Philosophical Society Museum (APSM) and the Philadelphia Water Department (Figure 2), this branch of Dock Creek underlying the site was buried by the year 1750. Dock Creek was evidently surrounded by marshes prior to this time. Historic structures at the site had been previously demolished and backfilled long before the current development.

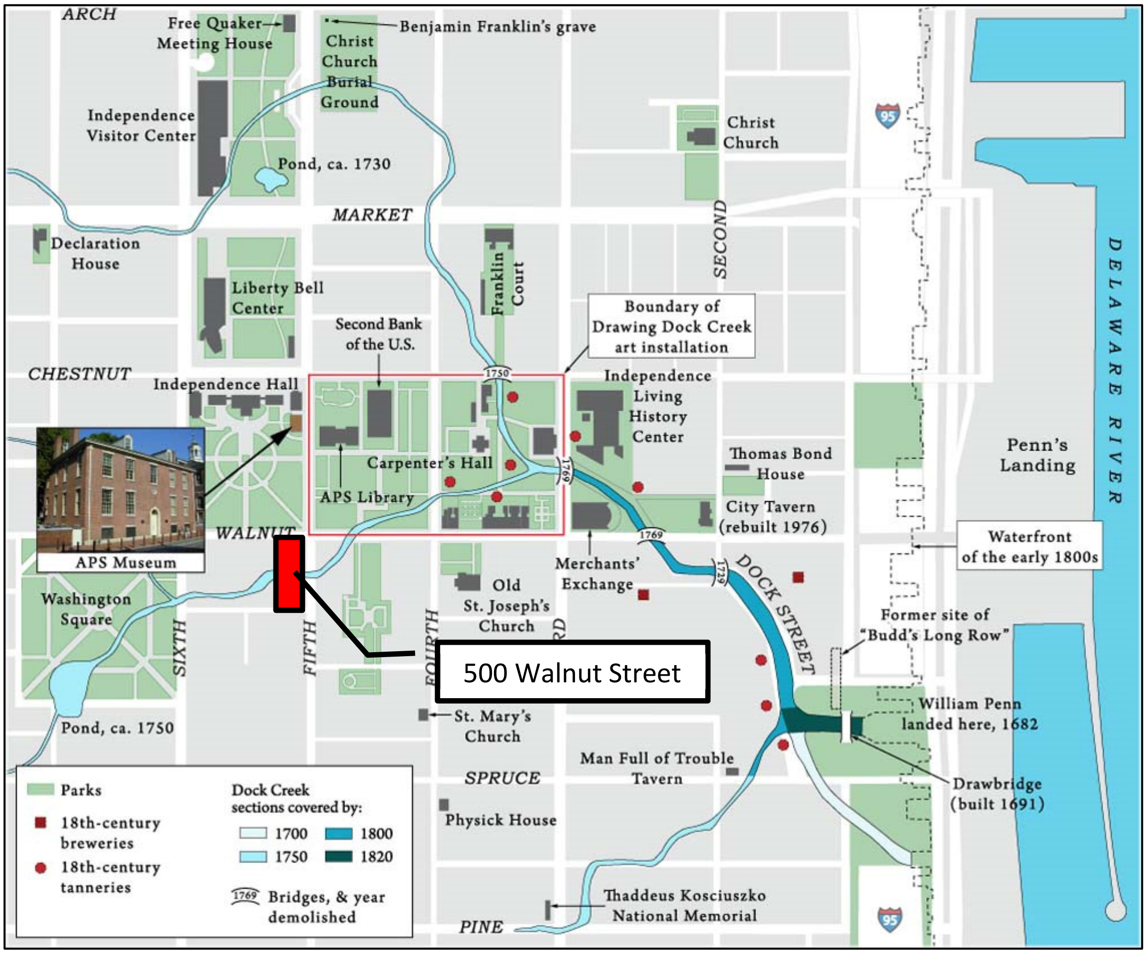

Figure 2. Illustration of 500 Walnut Street site location with respect to Dock Creek (APSM 2014).

Subsurface Conditions

Subsurface explorations conducted by the Geotechnical Engineer revealed a profile of urban fill / fill material [0 meters (0 feet) to ±7 meters (23 feet)], underlain sequentially by both fine- and coarse-grained fluvial deposits of a buried stream and marsh associated with the former Dock Creek [±7 meters (23 feet) to ±12.8 meters (42 feet)] and coarse-grained alluvial deposits of the Trenton Gravel Formation [±12.8 meters (42 feet) to ±23.8 meters (78 feet)], followed by a transitional zone of decomposed to weathered mica schist rock [±23.8 meters (78 feet) to ±27.4 meters (90 feet)]. Please refer to Figure 3 for an illustration of the generalized soil profile, including relevant standard penetration test (SPT) summary data.

Intact rock (defined by the Geotechnical Engineer as rock with a Rock Quality Designation greater than 40) was encountered at depths ranging from about 25 meters (82 feet) to 28.3 meters (93 feet) below ground surface, the surface of which generally sloped downward from south to north. Compressive strength test results on select rock core samples ranged from 13,720 kPa (1,990 psi) to 21,990 kPa (3,190 psi).

Groundwater at the site (according to two temporary groundwater piezometers) measured about ± 7 meters (23 feet) below surface grades (elevation -1.2 meters (-4 feet)), corresponding with the historic marsh deposits. Seasonal high groundwater was estimated at elevation -0.6 meters (-2 feet) for design considerations.

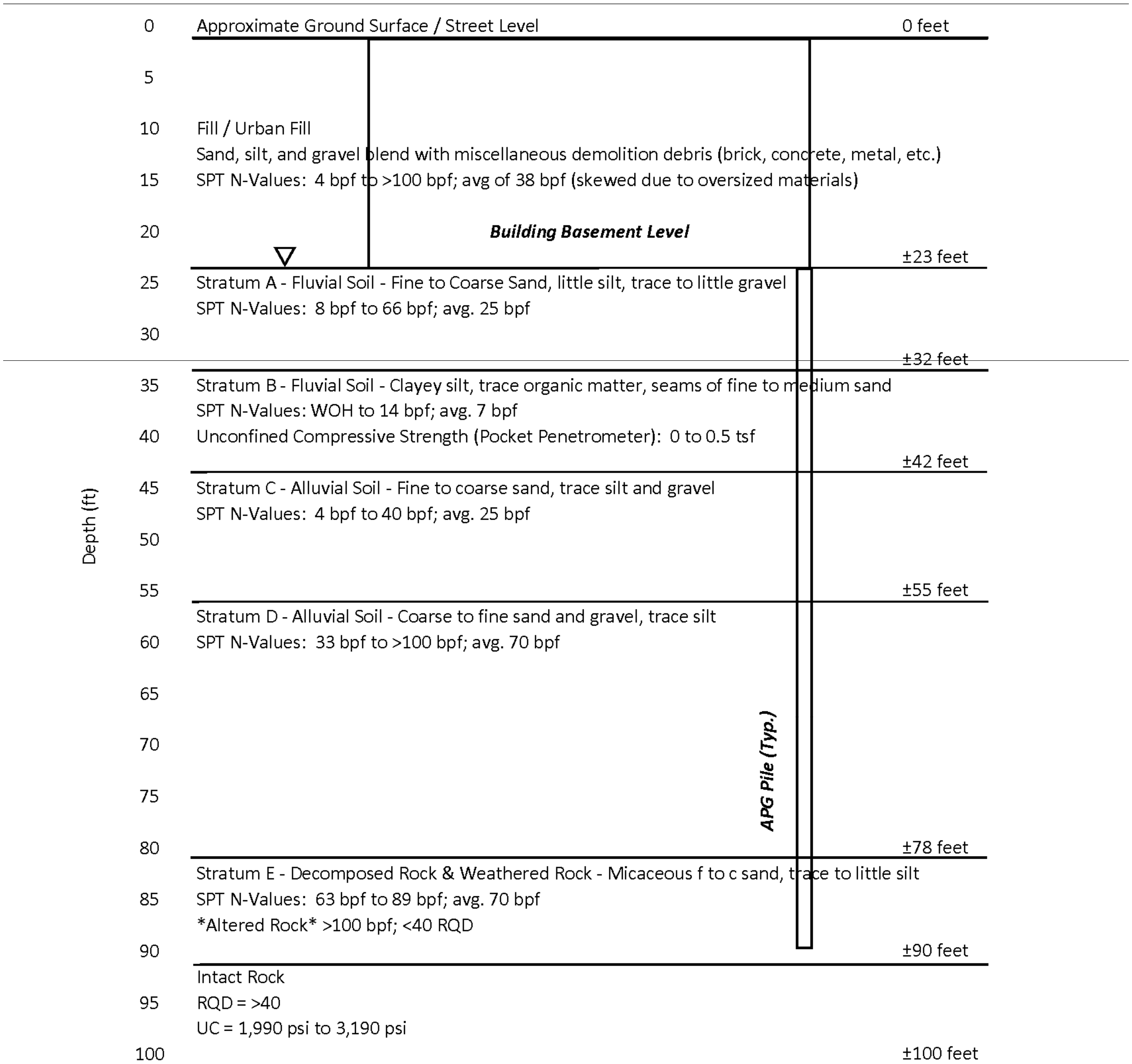

Figure 3. General Subsurface Profile (Gauffreau 2014).

Foundation Recommendations

Due to the high column and shear wall loads (compression, tension, and lateral), the presence of the existing Penn Mutual structure adjacent to the site, and the poor to marginal subsurface conditions encountered, the Geotechnical Engineer initially recommended drilled shafts (a.k.a. caissons) for support the proposed 500 Walnut Street tower. Specifically, the Geotechnical Engineer recommended straight drilled shafts, socketed into the underlying intact rock layer, designed and proportioned assuming an allowable end bearing capacity of 3,220 kPa (30 tsf) and an allowable side resistance of 535 kPa (5 tsf) in the socket.

Alternative foundation systems considered during the geotechnical design phase included traditional auger pressure-grouted (APG) piles, micro piles, driven piles, and a mat foundation. APG piles were considered initially but were ruled out due to assumed limitations with respect to the vertical and lateral capacities, as determined by conventional analyses. Micro piles were eliminated from consideration due to the anticipated expense, and the expected need to batter the piles to develop lateral capacity. Driven pile foundations were eliminated from consideration due to the presence of existing structures and vibration concerns, as well as the various obstructions that would be encountered during installation.

Value Engineering

The Structural Engineer proceeded with the design, incorporating drilled shafts for support of the 500 Walnut Street tower. The design documents were eventually released to prospective foundation contractors for bidding, most of whom specialized in drilled shaft foundation installation. During the bid process, the Construction Manager noted that the bid numbers were higher than anticipated. The authors were not privy to the actual contract bid values and unit rates, so this information is not available. However, the Construction Manager did disclose that the drilled shaft foundation contractors were incorporating a variety of contingencies due to various challenges associated with the drilled shaft foundation approach, including:

- Difficult site logistics, including very limited staging areas limited access to work areas;
- Overall depth to the intact rock bearing layer (18.3 to 21.3 meters (60 to 70 feet) deep, from bottom of basement excavation), which would require special drilling equipment;
- Need for temporary casing (and/or drilling fluids) to prevent collapse while excavating through the overburden soil layers, and corresponding challenges with delivering and staging the required casing;
- Presence of a cobble/boulder layer;
- Presence of a shallow groundwater table; and
- Expected volume of soil cuttings from the drilled shaft excavations, and related issues with spoils management at the site.

The Construction Manager subsequently reached out to a regional APG pile foundation contractor to solicit their opinion. During these discussions, the foundation contractor shared a recent local project experience where they were able to develop a relatively high capacity with APG piles in a similar geology. They noted how modern drilling equipment and techniques enable the installation of APG piles through soil strata that were previously considered impenetrable, including the cobble layers and weathered mica schist encountered at this site. This improved drilling capability expands the potential use and achievable capacities of APG piles.

To develop a better understanding of what capacities could be achieved from APG piles installed in various geologies, the APG pile foundation contractor began a practice of load testing piles beyond the typically required limit of two times the allowable design load. Instead, piles would be tested to loads up to three times the allowable design load. These additional test data were used to develop a database of empirical pile design parameters in various geologies.

Shortly thereafter, the APG pile foundation contractor supplied the design team with supporting load test data from the local site demonstrating their ability to achieve greater than a 1,780 kN (200-ton) allowable axial compressive capacity in a similar geologic setting. With the higher capacity APG piles now on the table as a viable option, the Structural Engineer pivoted and redesigned the foundation system for 1,780-kN (200-ton) allowable capacity piles. The revised APG pile foundation system was re-quoted, and ultimately determined to be the more cost-effective solution.

High-Capacity APG Pile Design

High-capacity APG piles were desired to achieve an economical foundation design due to the anticipated loads. However, experience has shown that conventional APG pile design methods do not account for recent improvements to APG pile installation techniques, modernized rotary heads with increased available torque, and redesigned cutting heads. As a result, they tend to underestimate side resistance contributions from both fine-grained and coarse-grained soils, resulting in moderate, under-predicted pile capacities. To avoid over-conservatism and to take advantage of their ability to penetrate dense gravelly sands (Trenton Gravel Formation) and intermediate geomaterials (weathered mica schist), the APG pile contractor used modified design parameters calibrated from their library of load test data in similar geologic conditions.

Both the Trenton Gravel Formation and weathered mica schist provide very high resistances with minimal pile-head deflection. The presence of these dense geologic features, as well as the local ability to obtain relatively high-strength grout on a reliable basis, provided the opportunity to maximize the allowable structural loads of APG piles in this setting, rather than being constrained by the traditionally estimated geotechnical capacity.

To achieve the desired allowable compression load, APG piles would have to be extended into the very dense decomposed rock / weathered rock at ±18.3 meters (60 feet) below working grade (basement level). Initial pricing efforts examined several different pile diameters with various allowable loads and considered both the cost of the piles and the foundation concrete. These efforts revealed that 45-cm (18-inch) diameter APG piles designed and proportioned to achieve an allowable compression load of 1,780 kN (200 tons) each would result in the most cost-effective deep foundation system. The 45-cm (18-inch) piles would also have an allowable tension load of 890 kN (100 tons) and lateral load of 89 kN (10 tons) each.

For economic reasons, the APG pile foundation contractor decided that two separate APG pile types would be installed. One pile type would be designed to resist only compression and lateral loads. The other pile type would be required to resist tension loads, as well as compression and lateral loads. Both pile types would have full length center bars installed, as well as a reinforcing steel cage installed in the upper 7 meters (23 feet) of the pile. However, the tension pile would have a much heavier #24 center bar that would be fully anchored into the concrete foundation pile cap above. Please refer to Figure 4 and Figure 5 for typical pile reinforcement details.

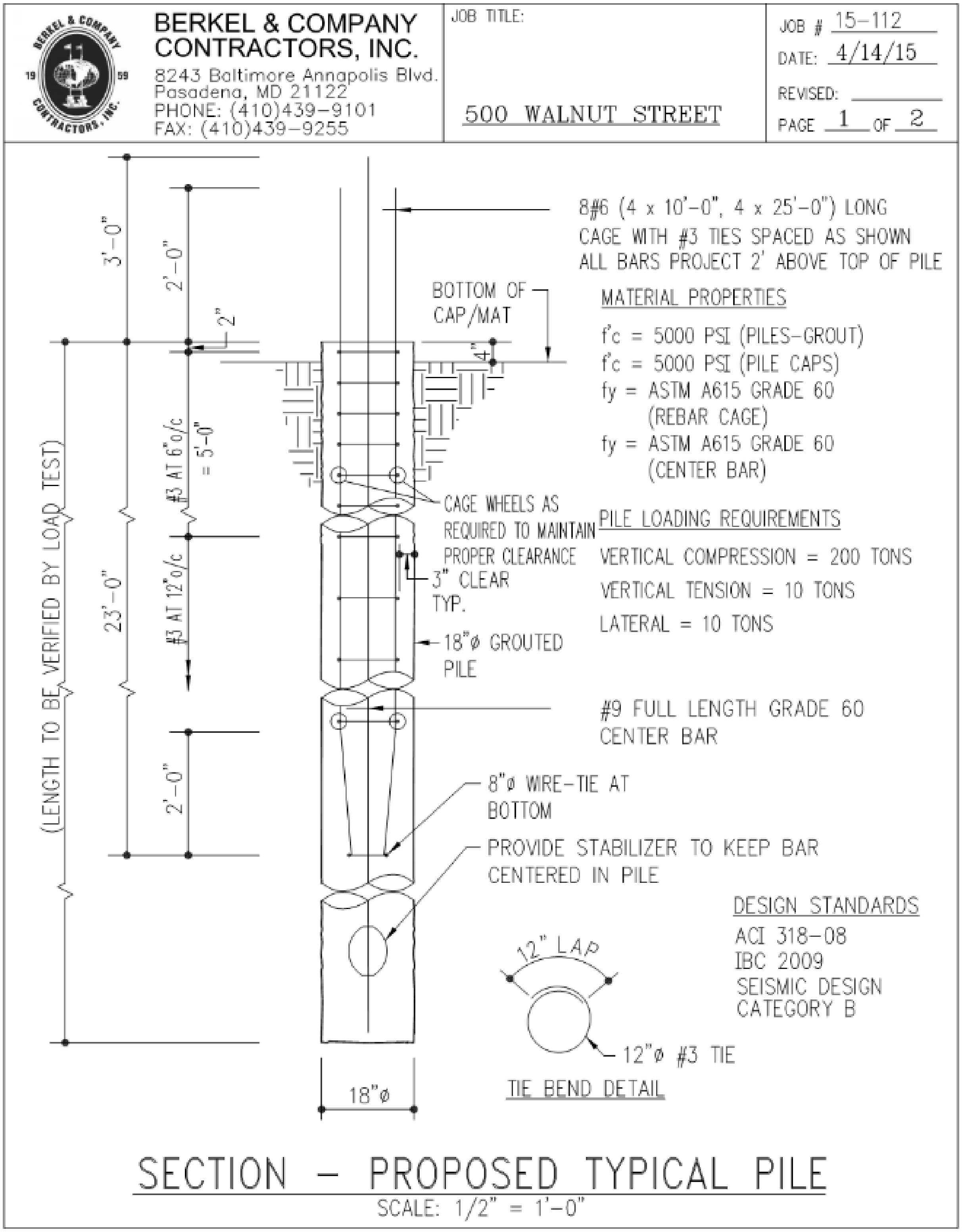

Figure 4. Typical APG Pile Detail (Chi 2015).

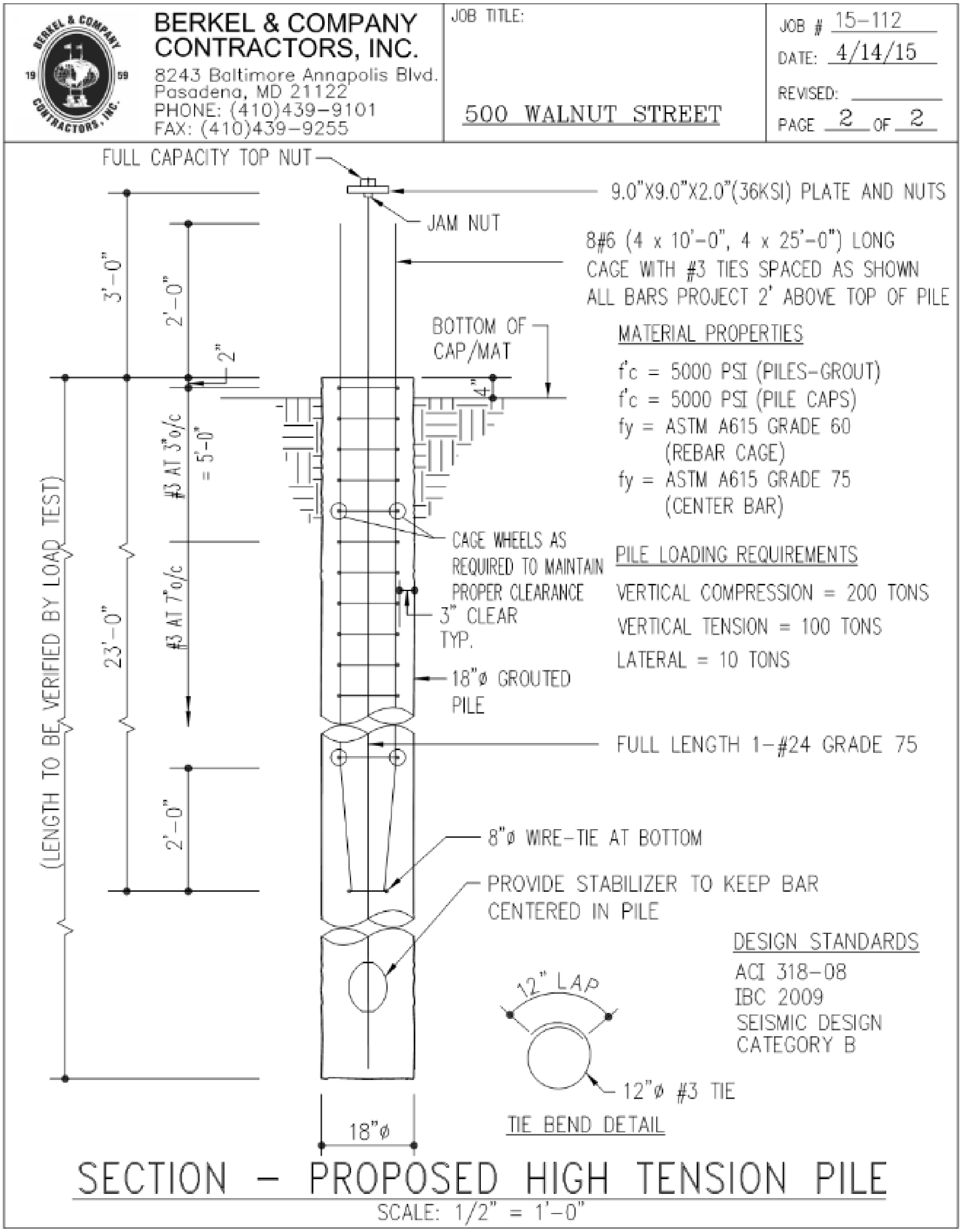

Figure 5. High Tension APG Pile Detail (Chi 2015).

Pile response under lateral load was evaluated using the commercially available software L-Pile by Ensoft. This analysis predicted the pile deflection and internal forces when subjected to the required lateral load. A "fixed head" condition was assumed for the lateral load analysis, based on the full embedment of the reinforcing steel cage into the pile cap. The fixed head assumption would produce the maximum theoretical stress at the top of the pile, and this resulting stress controlled the structural design of the piles. The subsequent sections provide additional discussion relative to the fixed head condition used for purposes of modeling lateral deflection, versus the actual free head condition during the lateral load test.

Load Test Program

To confirm design assumptions, test piles were installed for compression, tension, and lateral load tests. The test piles were sacrificial and were not used as production piles. A calibrated jack was used to apply load to the test piles. Reaction loads were transferred through a reaction frame and into the reaction piles. Refer to Figure 6 for a photograph of the pile compression test setup, and Figure 7 for a photograph of the lateral pile load test in progress.

Figure 6. Compression load test setup.

Figure 7. Lateral load test in progress.

The compression test pile was instrumented with four sister-bar mounted strain gauges so the soil resistance could be evaluated within each soil stratum. Strain gauges were located near the top and bottom of the pile, and at two intermediate locations that correlated with expected elevations of different soil strata. A test load of 4,450 kN (500 tons, or 250% of the allowable pile load) was applied per the method prescribed by ASTM D1143 Procedure A. At the 4,450 kN (500-ton) test load, pile head displacement was just under 15.2 mm (0.6 inches), with no indication of geotechnical failure. At the design load of 1,780 kN (200 tons), the pile head displacement was approximately 3.8 mm (0.15 inches). Refer to Figure 6 for the pile load versus deflection data. The generally linear nature of the pile-load versus deflection curve is typical for APG piles installed through very stiff intermediate geomaterials (such as the weathered rock at this site) to refusal on, or near, the underlying bedrock.

The tension test pile was tested to a load of 1,780 kN (200 tons, or 200% of the required allowable tension load) in general accordance with ASTM D3689. Because of the structural limitations of the center bar, higher test loads could not be safely applied.

At the 1,780 kN (200-ton) test load, pile-head displacement was approximately 20.3 mm (0.8 inches), with no indication of geotechnical failure. At the design load of 890 kN (100 tons), the pile head displacement was approximately 6.6 mm (0.26 inches). Refer to Figure 8 for the pile load versus deflection data.

The tension test pile was considered to have an ultimate tensile or uplift capacity in excess of the maximum applied test load of 1,780 kN (200 tons). Plot of the displacement vs. load data produced a graph with two distinct slopes. A relatively flat slope before the 530 kN (60-ton) load increment, and a steeper slope after the 530 kN (60-ton) increment. The theoretical tensile cracking stress of the grout corresponds to a 530 kN (60-ton) applied load. Thus, the change in slope is considered to be the point at which the grout cracked, resulting in a change in the elastic response of the pile.

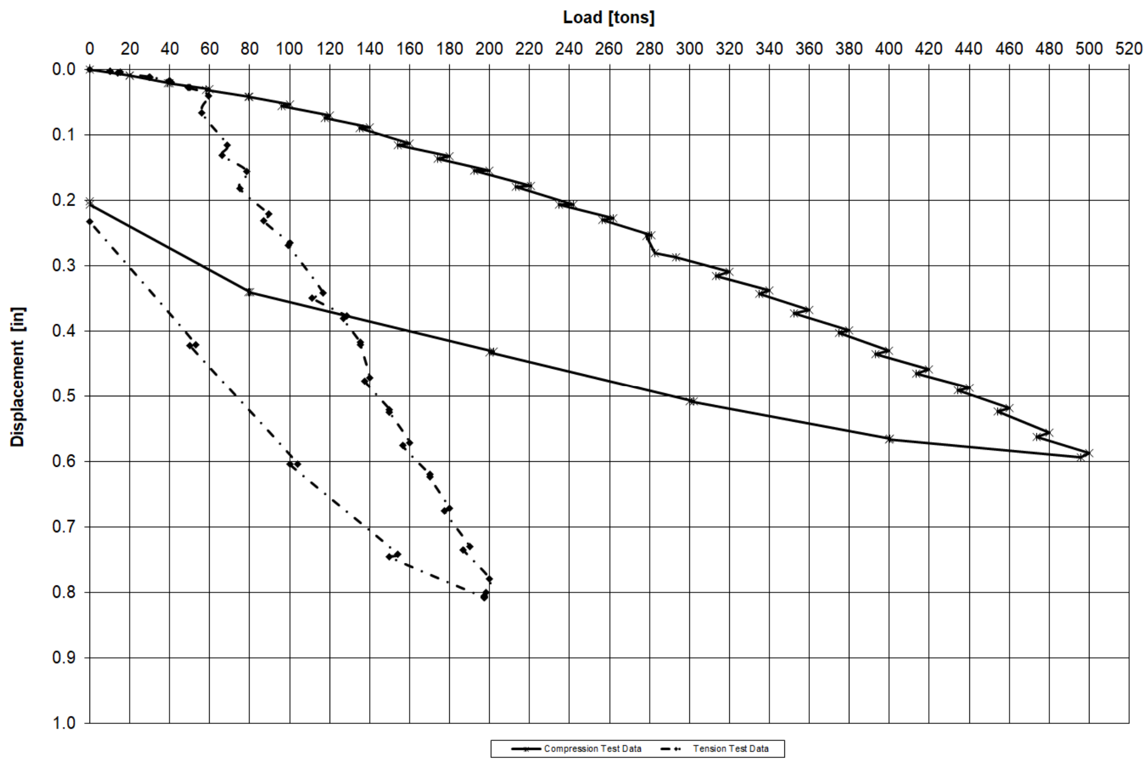

Figure 8. Applied Load vs. Pile-Head Deflection – Compression and Tension Pile Load Test (NeSmith 2015).

Test pile shaft resistance was estimated from the available strain gauge data. Data at the gauge depths, measured in micro-strain, was converted to load using a pile-modulus estimated from the data of the gauge near the pile-top, where the applied load is known. The strain gauge data confirmed the assumed strength and stiffness of the various soil strata. As expected, the Trenton sand and gravel layer and the weathered mica schist layer generated significant side resistance. The strain gauge data did not indicate a significant end bearing component, likely because the pile toe did not experience enough movement to develop the end bearing capacity. Refer to Figure 9 for an illustration of the load distribution versus pile depth.

Loads were estimated from the strain gage field data as follows:

- Data from the strain gage read in digits.
- Microstrain estimated from the gage data in digits and the gage factor on the supplied calibration sheets
- Pile modulus estimated from known load at the top of the pile and data from strain gage near the pile-top
- Loads at remaining gage depths estimated from respective gage data and the pile modulus estimated above (Load = modulus * pile area * strain)

Pile capacity was evaluated per the Modified Davisson Offset Limit method. Based on the results of the load test, the test pile was considered to have an ultimate compression capacity in excess of the maximum applied test load of 4,450 kN (500 tons).

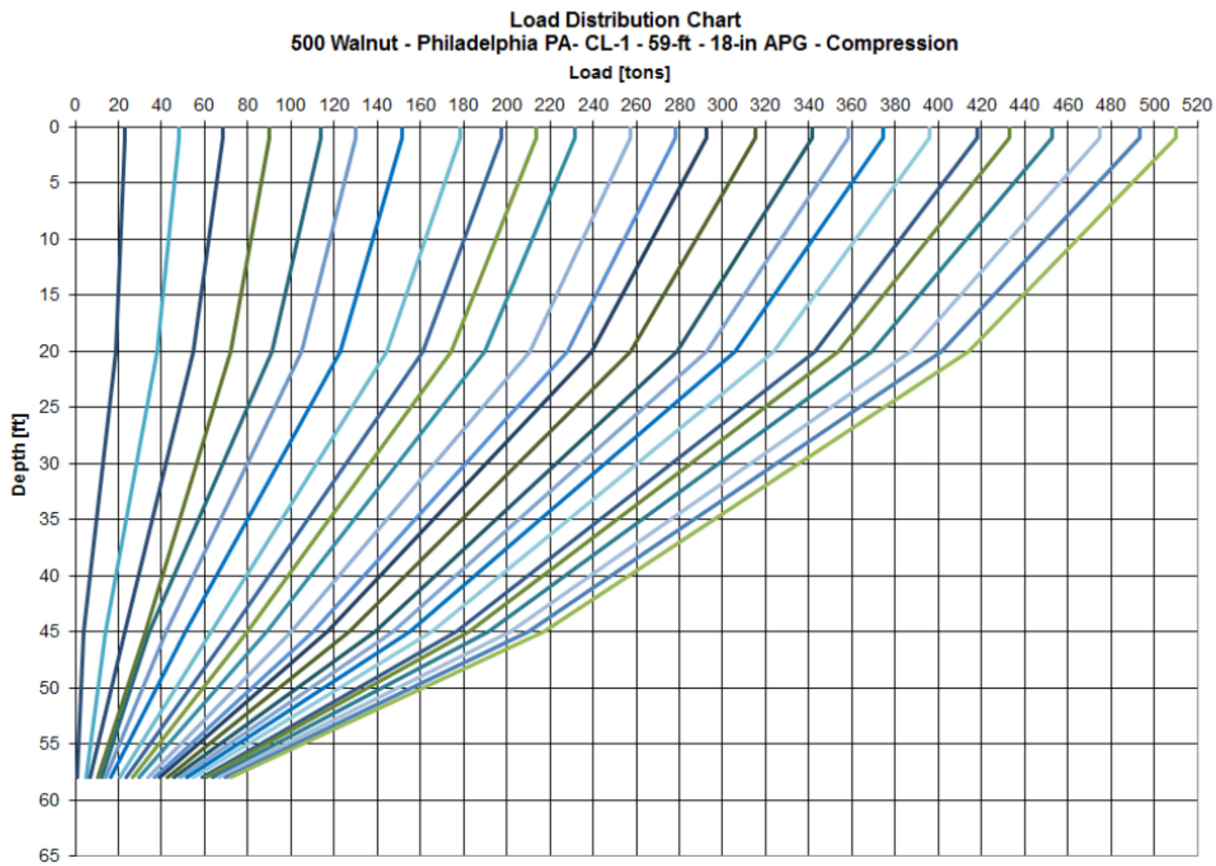

Figure 9. Load Distribution Chart – Compression Pile Load Test (NeSmith 2015).

The lateral test pile was tested to a load of 178 kN (20 tons, or 200% of the required allowable lateral load) in general accordance with ASTM D3966. At the 178 kN (20-ton) test load, pile-head displacement was approximately 45.7 mm (1.8 inches). At the design load of 89 kN (10 tons), the pile head displacement was approximately 14.0 mm (0.55 inches). Refer to Figure 10 for the pile load versus deflection data.

The lateral test pile condition did not match the same design conditions as production piles. The primary difference was that the test pile was tested in a free head condition, whereas the production piles are loaded in a fixed-head condition. Therefore, the lateral load test did not directly simulate the production pile condition. Instead, the lateral load test results were used to confirm the parameters used in the L-Pile analyses as a basis for the structural design of the production piles.

After completion of the lateral load test, an L-Pile model was constructed to model the pile under free-head conditions and mimic the lateral load test results, which resulted in realistic and verifiable design soil parameters. The L-Pile model was then changed to fixed-head to model the actual production piles. Group effects and top-of-pile level for production piles were also considered in the final design model. The lateral load test confirmed the validity of the original L-Pile analysis used for pile design.

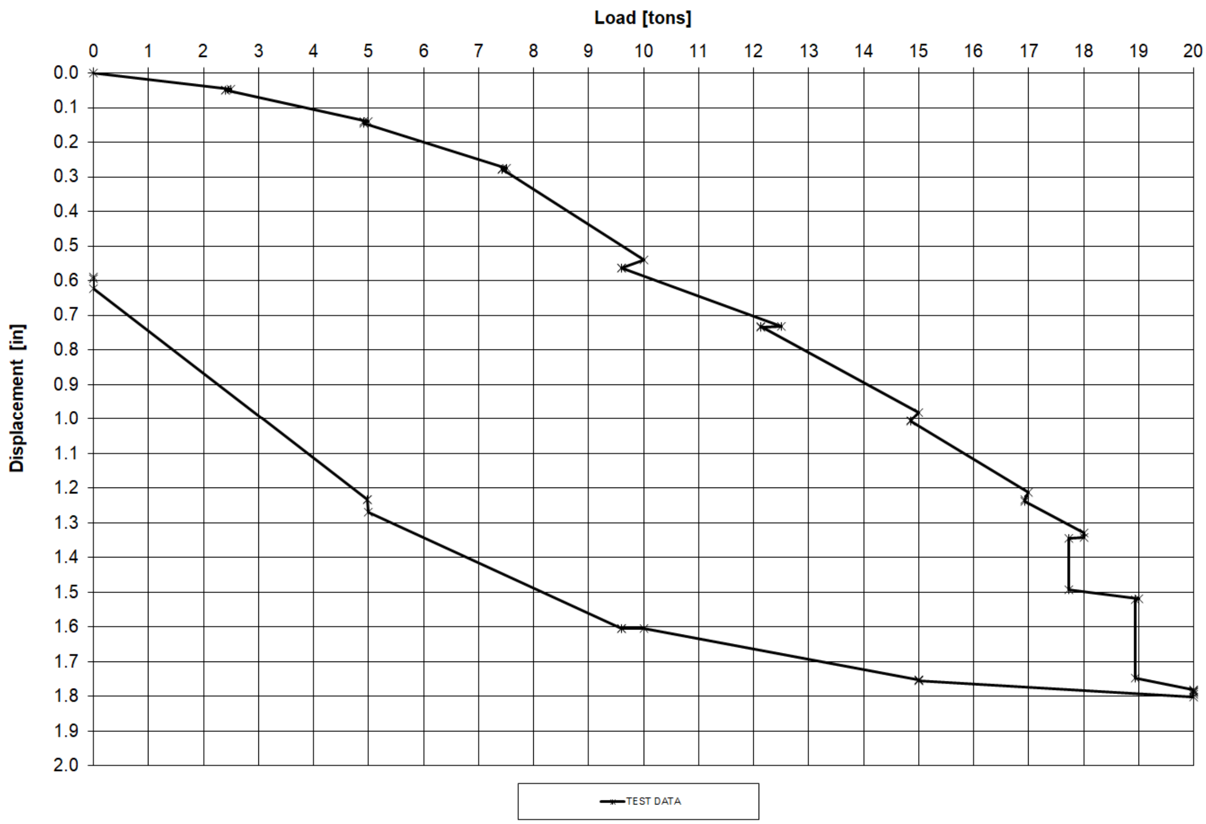

Figure 10. Applied Load vs. Pile-Head Deflection – Lateral Pile Load Test (NeSmith 2015).

Production Pile Installation

Production pile installation proceeded shortly after completion of the pile load test program and evaluation of the resulting data. Under normal operating conditions, the foundation contractor typically installed about 10 to 15 APG piles per day, with typical pile lengths ranging from about 19.8 meters (65 feet) to 24.4 meters (85 feet). In total, the foundation contractor installed 263 piles (249 piles for the building and an additional 14 piles for support of the tower crane) between July 1st and August 19th (approximate 7-week period). Figure 11 shows the layout of the APG piles for the building.

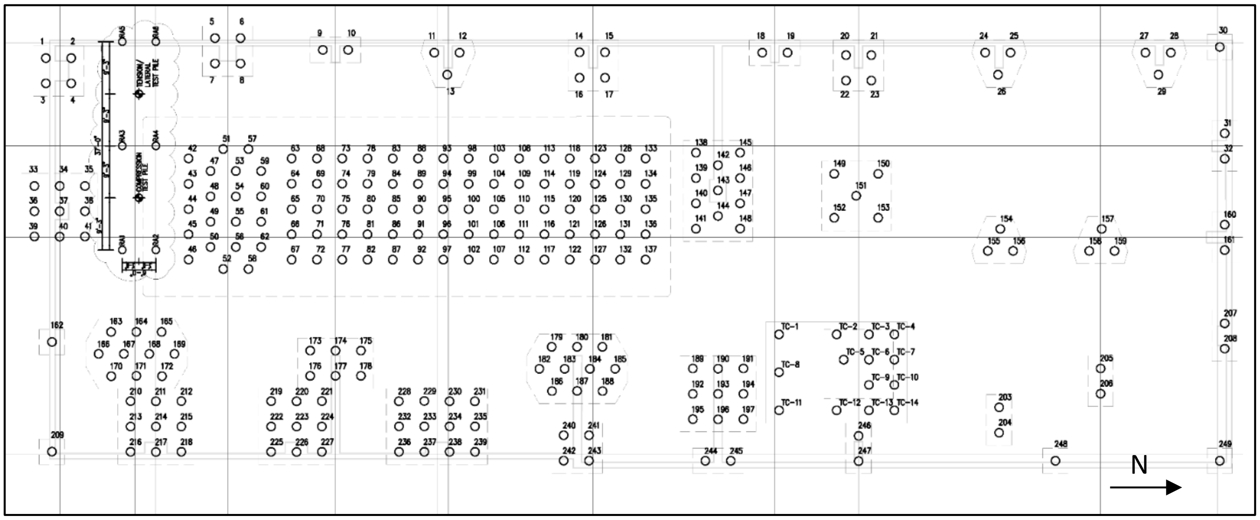

Figure 11. APG Pile Locations (Berkel 2015).

The Geotechnical Engineer provided full-time construction observation, materials testing, and Special Inspections services on behalf of the Owner throughout construction of the piles. The APG pile foundation contractor also performed their own independent QA/QC. The production pile installation proceeded relatively uninterrupted and with limited issues. Photographs during installation of the APG pile installation are provided in Figures 12 and 13. Minor issues that were encountered during installation of the APG piles included:

- Potential out-of-tolerance pile locations (due to auger flight deflection on shallow obstructions; special limitations preventing the rig from setting up on the proposed pile center; etc.), which were later surveyed and determined to be within the specified 7.6 cm (3-inch) tolerance or otherwise accepted, as constructed, by the Structural Engineer.
- One pile location was abandoned during installation due to broken tooling that was unrecoverable. As a result, the pile cap had to be reconfigured, including two new APG pile locations, to re-distribute the loads.
- Two piles were terminated prior to the specified refusal criteria due to limitations of the tooling. The foundation contractor's team evaluated the as-built data and determined that the pile capacity, as installed, exceeded the actual pile load demand. As a result, these piles were accepted by the Structural Engineer.

CONCLUSIONS

APG piles represent a proven, high-capacity deep foundation alternative in geologic settings similar to the 500 Walnut Street project site in Philadelphia. The industry has been somewhat slow to recognize the potential benefits to using APG piles over other traditional deep foundation alternatives, mostly due to perceived limitations with respect to the installation methods and the design side resistance. Through advancements in equipment, as well as proprietary drilling and APG pile installation techniques, specialty APG pile foundation contractors have shown their ability to penetrate relatively dense soil and moderately weathered rock (materials previously considered problematic or impenetrable as recent as 20 years ago), enabling them to maximize APG pile design capacities.

Figure 13. APG pile installation in progress.

Figure 12. APG pile rig; Test pile install.

The results of the load test data suggest that the contribution of the side resistance can be significantly underestimated throughout the pile length by conventional pile capacity analyses, particularly within dense layers of granular alluvial soil and/or weathered rock. Therefore, conventional capacity analyses of APG piles in dense, granular soil profiles should be revisited on a case-by-case basis to reduce conservatism and optimize the pile design.

Subsequent to the completion of the 500 Walnut project, APG piles with similar high capacities have been successfully used on several other recent projects in the Philadelphia metropolitan area, although load test data for these other projects are currently private. The successful performance of high capacity APG piles at these other projects validate our conclusion that APG piles installed in similar geology can achieve capacities higher than previously realized. When such additional capacities can be realized, and verified by load testing, APG piles become more competitive with other deep foundation systems, and in some cases, logistically favorable for installation.

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