AUGER PRESSURE GROUTED DISPLACEMENT PILES: An Acceptable Alternative

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ABSTRACT

There are many types of deep foundations currently being used in the San Francisco Bay Area, one of the most common being driven precast, prestressed concrete (PC/PS) piles. PC/PS piles are frequently selected for use due to of ease of installation, availability, and relatively low cost. Disadvantages of using PC/PS piles are noise, vibrations, pollution, front-end delays due to indicator pile programs, and reduced flexibility. An acceptable alternative to driven PC/PS piles was selected for construction of foundations for the heavy housing and administrative buildings at the San Francisco County Jail #3 replacement project in San Bruno, California. The pile type selected, the Auger Pressure Grouted Displacement (APGD) pile, was chosen to reduce noise, vibrations, installation time, and cost, and to allow for greater flexibility of installed pile lengths. This paper presents a case history of the use of APGD piles that were substituted for driven PC/PS concrete piles for the subject project. Standard and quick loading pile load tests were performed to confirm adherence with the design criteria originally used for the driven PC/PS concrete piles. The APGD piles were found to increase shear strength around the piles similarly to driven displacement piles. In addition, this paper presents the methodology used in monitoring APGD pile installation.

PROJECT DESCRIPTION

The San Francisco County Jail facility is located at One Moreland Drive in San Bruno, California, as shown in the Site Vicinity Map – Figure 1. The facility covers an approximate 150-acre parcel of land owned by the City and County of San Francisco in San Mateo County. The area is bounded by the San Francisco State Fish and Game Refuge, the Golden Gate National Recreation Area, Skyline College, and residences. Several jail structures currently exist at the facility including the existing Jail No. 3 housing complex, Jail No. 7, and the Learning Center. These three structures are located within a relatively level area, and are surrounded by moderately steep hills on the south, west, north, and northeast.

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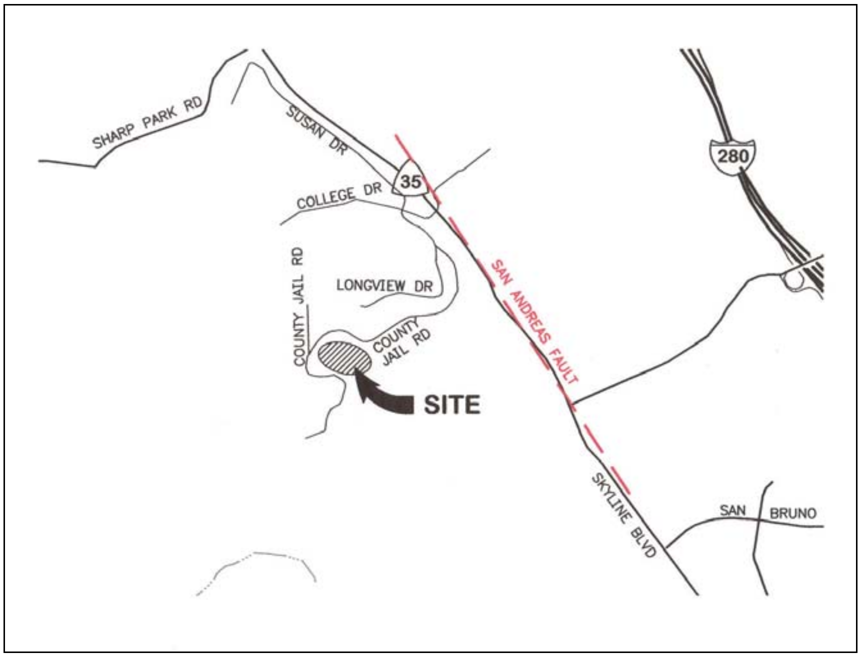

Figure 1 - Site Vicinty Map

The project consists of constructing the following:

- Six new roadway alignments
- New Staff and Public Parking Lots
- A four-story Jail No. 3 Housing Complex
- A two-story Administration Building
- A bridge connecting the new Jail No. 3 Housing Complex to the Administration Building
- A one-story Central Plant
- A 70-foot diameter, 800,000-gallon water storage tank
- Eight retaining walls

The layout of the three buildings are shown on the site plan – Figure 2.

This project is a design build project. The general contractor is AMEC, the architect is Kaplan, McLaughlin, Diaz, the structural engineers are The Crosby Group and SOHA Engineering, and the civil engineer is Telamon Engineering Consultant.

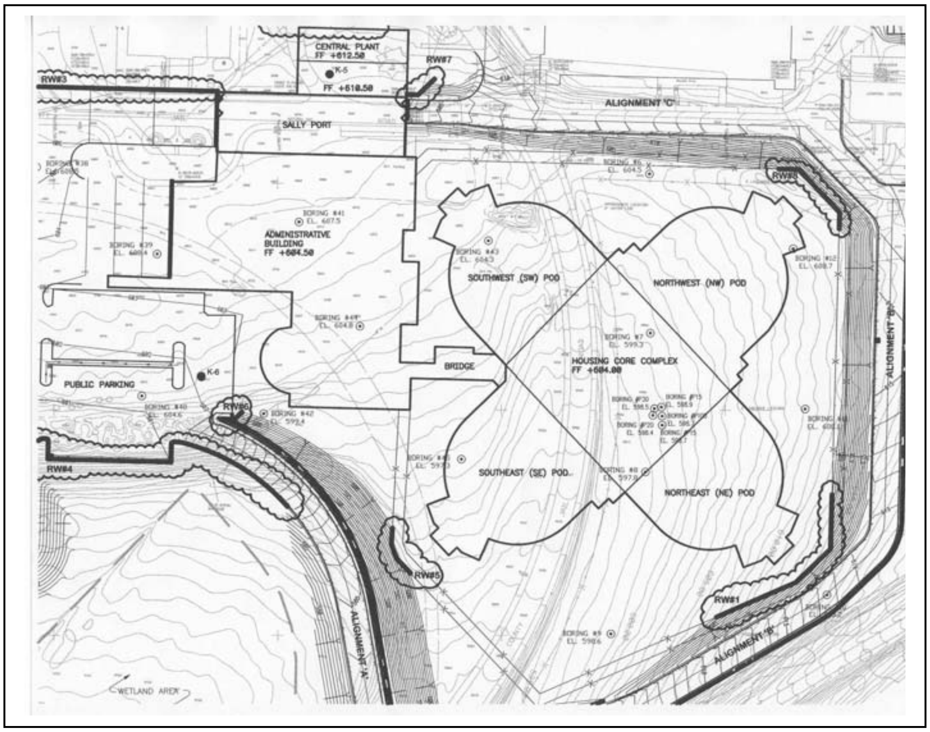

Figure 2 – Site Plan

GEOLOGIC SETTING AND SUBSURFACE CONDITIONS

The San Francisco County Jail site is located along the western foothills of the Coast Ranges Geomorphic Province in the County of San Mateo, California. The site is located approximately 3 miles east of the Pacific Ocean on the San Francisco Peninsula.

Geologic maps covering the site area have been prepared by Schlocker (1970), Brabb and Pampeyan (1972 and 1983), LaJoie and others (1974), Wagner and others (1991), Ellen and Wentworth (1995) and Wentworth and others (1998). These maps generally agree that the hills that surround the jail site are comprised of Franciscan Complex greenstone (altered fine-grained volcanic rock) of Cretaceous geologic age, while the low lying areas of the site are underlain by Quaternary surficial deposits such as alluvium and slope wash materials.

Wentworth and others (1985) described the surficial deposits present in the low lying areas in the site vicinity as slope debris and ravine fill. In addition, the jail facility has undergone extensive grading with fills reported to be up to about 30 feet thick. Small to moderate-size, old landslide deposits with subdued topographic expression were observed along the hillside areas of the site. A wetlands area is located at the southeast quadrant of the facility.

The site is located approximately 0.5 kilometer (km) west of the active San Andreas fault, about 9.5 km northeast of the San Gregorio fault, and about 30 km southwest of the active Hayward fault. No known active faults are present within the boundaries of the site. Based on the information provided in Hart and Bryant (1997), the site is not within an Alquist-Priolo Earthquake Fault Zone. The Working Group on California Earthquake Probabilities (1999) estimated probabilities of large earthquakes (magnitude 6.7 or greater) in the San Francisco Bay Region for segments of the San Andreas, Hayward, and Calaveras faults. The study estimated that there is a 70 percent probability of a large earthquake in the San Francisco Bay Region in the next 30 years (from 2000). With respect to individual fault segments considered in the study, estimates are provided of 21 percent for the San Francisco Peninsula segment of the San Andreas fault, 32 percent for the Hayward fault, and 18 percent for the Calaveras fault. A major seismic event on these faults could cause significant ground shaking at the site.

The Housing complex and Administration Buildings are underlain by up to 17 feet of fill consisting of very loose to loose organic silty sand. The fill was found to overlie interbedded layers of medium stiff to very stiff lean clay, sandy lean clay, and loose to medium dense clayey sand. These soils were found to be underlain by an approximately 3- to 6-foot thick layer of very soft to soft organic silt. The organic silts were found to be underlain by highly weathered and fractured Greenstone/Franciscan bedrock occurring at depths below the existing ground surface of about 32 to 62 feet at the Housing Building site, and about 32 to 47 feet at the Administration Building site. Groundwater was observed during drilling at depths of approximately 4 to 16 feet below the ground surface.

Unconfined compressive tests performed on the fill material indicated average shear strengths of 840 pounds per square foot (psf), while unconfined compressive strengths of the native soils were approximately 2,000 psf. A direct shear test on the native soil resulted in a cohesion of 860 psf with a friction angle of 26 degrees.

INITIAL FOUNDATION RECOMMENDATIONS

The fill soils at the site were judged incapable of directly supporting the building loads. Because of the soft consistency of the fill soils, and due to the presence of the soft zone of clayey soils overlying the Greenstone/Franciscan bedrock, it was decided to support the buildings on deep foundations extending into the weathered bedrock. A driven pile system was favored over a drilled pier system due to the shallow depth of groundwater, the presence of soils with moderate caving potential, and the unknown composition of the fill from an environmental standpoint. Pre-cast, pre-stressed 14-inch square concrete driven piles were originally recommended to support all of the compressional building loads and a portion of the lateral loads for the Administration Building and Housing Complex. To provide additional lateral resistance during a seismic event, short 24-inch diameter, 10-foot deep piers (lugs) were also to be recommended. The pile layout for the Housing Complex is shown on Figure 3. The piles were designed to derive support both through friction and end bearing in the alluvium and the underlying bedrock. Based on the results of our analyses, the piles needed to be socketed a minimum of 3 feet into bedrock, except where significant amounts of new fill were to be placed, which would lead to a potential downdrag condition, and would therefore necessitate deeper penetration into the bedrock. The piles were designed for an ultimate skin friction of 425 pounds per square foot (psf) for the upper 10 feet of the existing fill, 1,000 psf for the native sandy gravelly clay material above the bedrock, and 3,000 psf for the weathered rock. In addition, the piles were designed for an allowable end bearing value of 75,000 to 85,000 psf. A factor of safety of 2.0 was used for the skin friction portion and 3.0 for the end-bearing. If the end-bearing capacity is converted into friction and combined with the skin friction used, then the equivalent skin friction for the piles would be about 1,000 psf allowable or 2,000 psf ultimate. An average ultimate skin friction value for the piles used for the design was about 1000 psf. In areas where new fill was placed, the skin friction in the new fill was ignored. These values reflect higher unit resistances as compared to those used for design of conventional drilled pier foundations due to the beneficial effect of displacement of the soils surrounding the piles. For drilled piers, reduction factors typically ranging from 0.5 to 0.7 are applied to the earth pressure used in skin friction evaluations reflecting active soil pressures. For driven piles, this factor may be increased to reflect an at-rest or even a passive state. The factors associated with this condition can be between 1.45 and 1.8 (Bowles [1996]). Generally, these factors apply to granular soils and not necessarily applicable to cohesive soils.

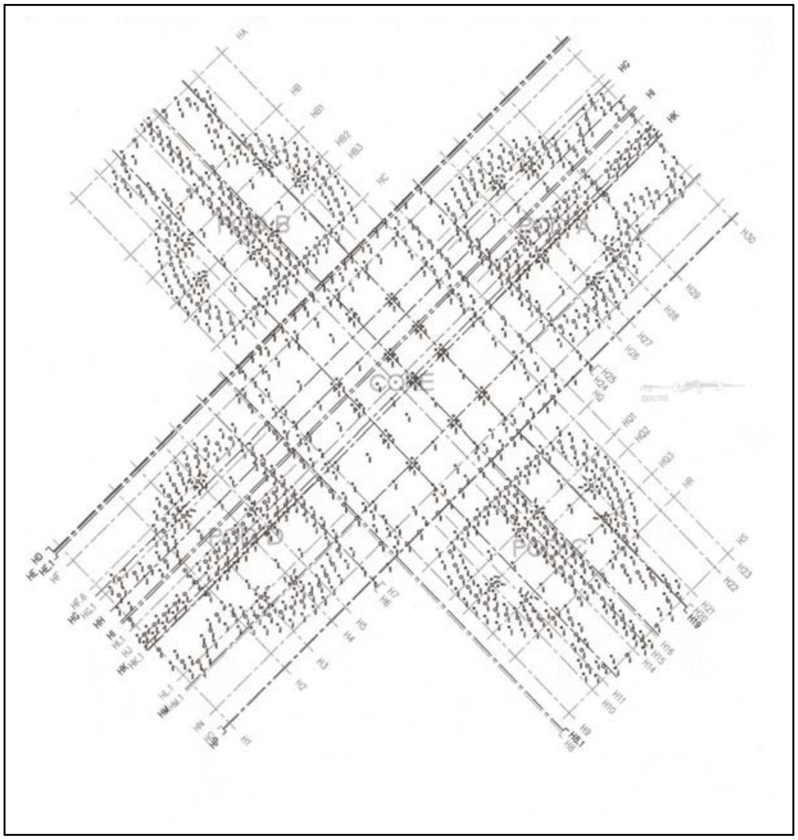

Figure 3 – Pile Locations at Housing Complex

The 14-inch driven piles were designed for allowable downward capacities of 200 kips, with a one-third increase for the seismic loads. For areas in the Housing Complex that would experience downdrag, the allowable capacities were reduced up to 60 kips to account for the downdrag loads.

The depth to bedrock varies at the sites of the planned structures from 32 to 60 feet. Because of this variation, to confirm the estimated depth of the bedrock, an indicator pile program of 20 piles was recommended.

AUGER PRESSURE GROUTED DISPLACEMENT (APGD) PILES

Because of the potential for complaints from the project neighbors during installation of the piles, an accelerated project schedule, and cost concerns, an alternative foundation system consisting of Auger Pressure Grouted Displacement (APGD) piles was considered to replace the precast, prestressed 14-inch square driven piles. This system was suggested by Berkel & Company Contractors, with the contention that the skin friction values would be equal to or greater than those used in our design of the driven piles. The APGD piles were to be 18 inches in diameter and reinforced by six #6 vertical rebars in the upper 24 feet, with a #8 rebar extending the entire depth of the pile. APGD piles are constructed by drilling a displacement auger into the subsurface soils and pumping grout under pressure while withdrawing the auger. The displacement auger includes a 16-inch diameter flight auger tip that is about 10 feet long. At about 4 feet above the auger tip, an approximately 2 ½-foot long, 18-inch diameter steel tube section placed over the auger acts as a displacement element. The displacement auger is fixed to a 12-inch drill stem. The drill rig used for APGD pile installation for this project was a Bauer drill rig with a capacity of approximately 335 bar of torque. During drilling, the soil is displaced laterally, thus creating an at-rest or passive condition. Because this installation process does not allow for observing the type of material encountered, the torque of the drill-rod is recorded throughout the process. The torque is initially calibrated by locating the first piles adjacent to test borings drilled during the field investigation and comparing the torque at various depths with the soils encountered in the borings. Although this system of monitoring seems unsophisticated, it worked well for this project due to the significant increase in torque going from the native soil to the bedrock (a minimum of 3 feet in the bedrock was required). During construction of the pile, the total volume of concrete used is compared to the theoretical volume to insure that caving has not occurred. The torque, drilling depth, drilling rate, and grout pressure were continuously electronically monitored at the drill rig cab. A sample print out of the electronically acquired installation monitoring data is shown in Figure 4. As indicated in this printout, the slope of the depth curve shows the drilling rate. The steeper the slope, the harder the material is to drill through. From the torque curve, an estimate can be made of the average maximum torque the machine is delivering as the displacement auger is drilled through hard material such as bedrock. A torque value of 300 bars was found to be indicative of a very hard material such as weathered bedrock (Greenstone/Franciscan formation) for this project.

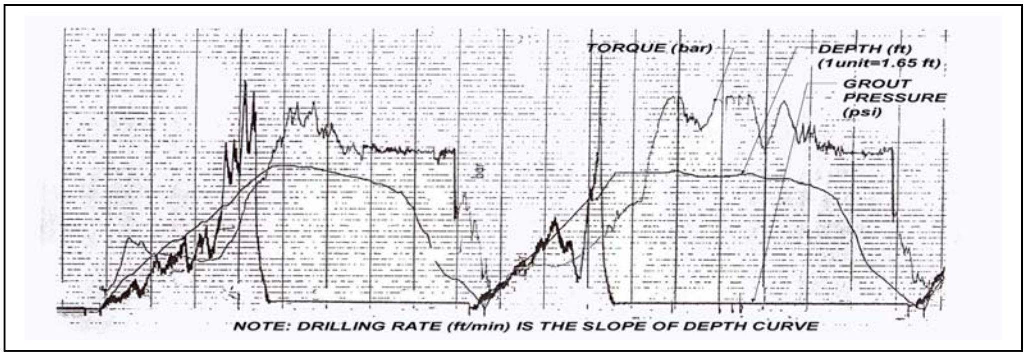

Figure 4 – Drill Rig Printout

LOAD TESTS - SETUP

Pile load tests were performed on test piles constructed at each of the building sites. The purpose of the load testing was to confirm that the APGD pile installation method resulted in equivalent or greater unit resistances than used in our design of the driven piles. The setup of one of the pile load tests is shown in Photograph 1, below. Both the standard loading and the quick load test procedures were used for the first test pile (located at the Housing Building site), while only a quick load test procedure was used on the test pile at the Administration Building site. Compression pile load tests were conducted in general conformance with ASTM D1143 using both the standard loading procedure and the quick load test method. Each compression load increment for the standard test method was maintained until the rate of pile displacement was about 0.01 inches per hour based on a time period of at least 15 minutes. The hold periods of load increments for the quick load test method were about 5 minutes. Loading increments of about 50 to 55 kips were used on all tests. Nominal load decrements were about 25 percent of the maximum load attained during the subject loading cycle. As shown in Photograph 2, a load cell was used to measure applied compression loads during each test. Load and displacement measurements for each test pile were obtained using a data acquisition system connected to an on-site laptop computer. Pile load test data was processed and the results viewed in real time on a laptop display using a program developed by GEODAQ, Inc. Primary and secondary displacement readings were obtained during all pile load tests. Primary readings were made using four DC powered displacement transducers (DCDT) mounted close to the pile top, and backup measurements were made by reading a scale through a level.

Photograph 1 – Load Test Set-Up

Photograph 2 – Load Cell

LOAD TESTS – RESULTS

A maximum load of 450 kips was applied to the test pile at the Housing Building location. Axial displacement at 200 kips (approximate design load) varied from 0.065 to 0.071 inches. The maximum axial displacement was about 0.21 inches at the end of the 450 kips hold period of 12 hours. After unloading the pile to zero load, the axial displacement was about 0.03 inches, as shown graphically in Figure 5.

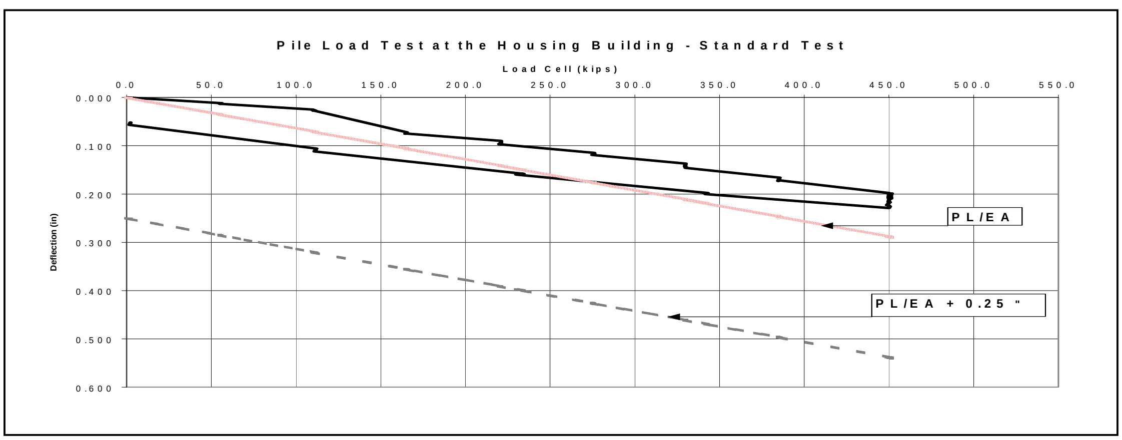

Figure 5 – Pile Load Test at the Housing Building – Standard Test

A maximum load of 520 kips was applied to the same pile during a subsequent quick load test. Axial displacement at the design load of 220 kips varied from 0.071 to 0.073 inches during the quick load test. The maximum axial displacement was about 0.21 inches at the maximum test load of 520 kips. After unloading the pile to zero load, the axial displacement was less than 0.01 inches as shown graphically in Figure 6. The load-displacement plots for the standard test method and the quick load test method were nearly identical, which indicates that secondary loading effects such as grain crushing, consolidation, and soil creep were minimal. The load test results indicate that the ultimate load capacity for the test pile is significantly greater than the required ultimate capacity of 450 kips. Results from both tests indicate that the pile-soil system remains essentially elastic within the range of applied compression load.

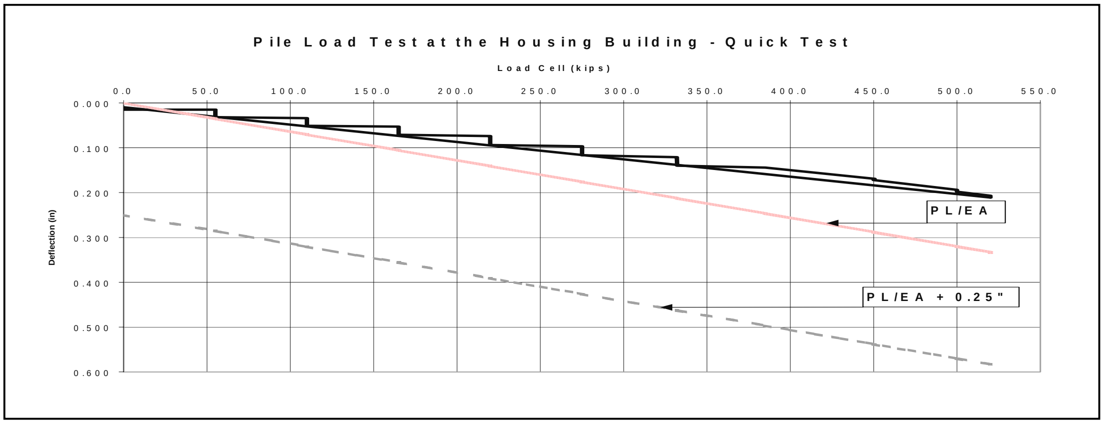

Figure 6 – Pile Load Test at the Housing Building – Quick Test

The second test pile, located in the Administration Building area, was tested to a maximum load of about 490 kips. Axial displacement at 200 kips (approximate design load) was about 0.07 inches. The maximum axial displacement was about 0.34 inches at 490 kips, the maximum test load. After unloading the pile to zero, the axial displacement was about 0.10 inches as shown in Figure 7. The load test results indicate that the ultimate load capacity for the test pile is significantly greater than the required ultimate capacity of 400 kips. Results of this test indicate that the pile-soil system was slightly beyond the elastic range of behavior at the maximum test load.

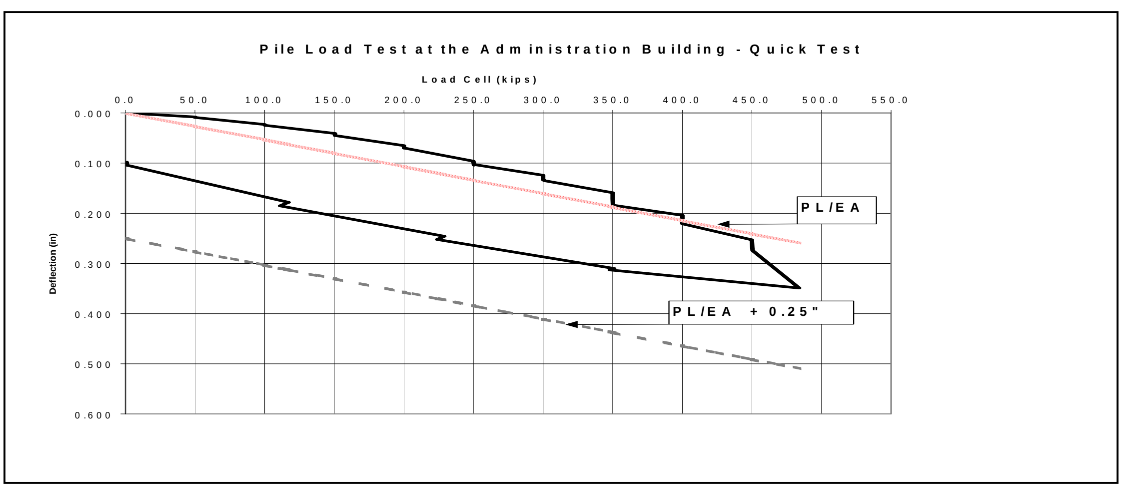

Figure 7 – Pile Load Test at the Administration Building – Quick Test

Both test pile load test results were interpreted based on Tomlinson (1969) using a value of ¼ inch settlement plus the elastic compression of pile (PL/EA) curve to develop a failure line as drawn in Figures 5, 6, and 7.

LOAD TESTS – CONCLUSIONS

The pile load test results indicate that the APGD piles tested exceed the required pile capacities used in our analysis. The load test at the Administration Building shows that it took about twice the design load to fully mobilize resistance over the entire length of the pile, as indicated by the intersection of the PL/EA curve with the load-displacement curve. These results also indicate that the piles have achieved twice the design capacity utilizing skin friction only without mobilizing end-bearing resistance. The standard load test at the Housing Building site shows that at twice the design load, the full length of the pile had not been mobilized, and, therefore, end bearing had not been engaged. When a quick load test was repeated on the same pile at the Housing Building site (after completing the standard load test), the pile deflection was similar to the standard test. The interpreted ultimate skin friction values from the load tests were in the range (averaged over the length of the pile) of 1,800 to 2,400 psf. An average ultimate skin friction value of about 2,000 psf was used for our analyses.

INSTALLATION

The piles were installed between September 20, 2001 and March 6, 2002 using a Bauer hydraulic drill rig. During installation, torque measurements were continuously made and recorded versus depth of penetration, and the drilling rate was also monitored. As shown in Photograph 3, monitoring was performed by both the drill rig operator and our field representative. A torque of about 280 to 300 bars with a drill rate of about one to three feet per minute was established as an indication of penetration into bedrock. As soon as the required embedment was achieved, the hole was filled through the stem of the auger with grout using a grout pump while the auger was being withdrawn. The grout pressure was recorded, and the grout volume calculated. After filling the hole with grout, the steel cage and center bar were inserted into the hole. After the auger was pulled out of the hole, the material retained on the auger tip was observed as a secondary means of confirmation that bedrock material was penetrated. Photograph 4 shows the tip of the auger. Pile production averaged about 20 to 25 piles per day. Berkel & Company has completed installing about 800 APGD piles at the housing complex and about 160 piles at the administration building. The 24-inch diameter short piers (lugs) for additional lateral load resistance were also installed by Berkel & Company by the auger pressure grouted (APG) piling method. Pile lengths ranged from 25 to 68 feet in the housing complex and between 25 to 48 feet in the administration building.

Photograph 3 – APDG Pile Installation

Photograph 4 – Displacement Auger

CONCLUSIONS

Auger Pressure Grouted Displacement (APGD) piles installed and tested for this project demonstrated unit capacities equal to or greater than those that would be expected for a driven PC/PS concrete pile foundation. In addition, the selection of the APGD piling method allowed for a quick start, reduced noise and vibration, and was less costly relative to driven PC/PS concrete piles. The pile load tests indicate that the shear strengths of the soils at the site were improved during the drilling process which resulted in higher soil resistances than indicated by our laboratory tests. It is essential that when the APGD piling method is used, an installation monitoring procedure be established similar to that followed for this project.

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