CASE HISTORY: EFFECT OF PLASTIC CLAYS ON THE BEHAVIOR OF APGD PILES IN CENTRAL FLORIDA

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In recent years, with the increase of the number of high-rise buildings that are proposed and being built in Central Florida especially in Downtown Orlando, Augered Pressure Grouted Displacement (APGD) piles became a pronounced foundation option. The APGD pile is a bored displacement pile that does not result in drilling spoil. APGD piles are mostly suited for loose to medium granular soils and slightly silty/clayey sandy materials, such as those encountered in Central Florida. The case history presented in this paper involves the APGD piles recommended for a 35-story mixed-use condominium in Downtown Orlando. The subsoil condition was typical for the region; that is loose to medium fine sands underlain by thin layer of plastic sandy clays and plastic clays followed by mixed clayey sands and sandy clays with phosphates and traces of shells transitioned to the weathered limestone. Four APGD test piles were installed and were, statically, tested. Three of the tested piles failed to support the design load. Analysis of load test results showed that the bottom half of the test piles, below the plastic clay deposits, was not effective, which was attributed to structural deficiency resulted from possible pile necking due, in our opinion, to the increase of the porewater pressure within the plastic clays. Three additional test piles were constructed with modified reinforcement cage; that is to extend beyond the plastic clays with tighter stirrups. The additional test piles were, statnamically, tested and were able to sustain the design loads. The construction of the APGD piles through subsoil conditions similar to those of Downtown Orlando, where plastic clayey deposits are encountered, should account for the high porewater pressures that mostly will be developed within the plastic clay, hence, may cause pile necking. Extending the cage to penetrate through and pass the plastic clays reduced the necking potential and improved the test piles performance.

INTRODUCTION

During the last decade, a record-high number for high-rise buildings may have been set in Central Florida since it continues to attract an increasing number of residents and businesses. Many of the old low-rise buildings in Downtown Orlando are being demolished and replaced by high-rise mixed retail, business, and residential towers. Due to high column loads associated with those towers, deep foundations are usually utilized.

Popular deep foundation systems in Central Florida include: i) driven piles such as closed and open-ended pipe piles, H-piles, and pre-cast concrete piles, ii) bored and cast in place piles such as Augered Cast In Place (ACIP) piles and Augered Pressure Grouted Displacement (APGD) piles, and iii) drilled shafts, which usually utilized for bridge foundations and where a competent bearing stratum is encountered at reasonable depth. Solid, ribbed, and compensated rafts were also utilized for lighter column loads. Piled-raft system may be utilized to control settlement when mat foundations provide satisfactory bearing capacity; however, although piled-raft system has been used in Europe for decades, it has never been utilized in Central Florida. Among the above systems, deep foundations in form of APGD piles is currently gaining popularity especially within urban areas.

The APGD pile is similar to the conventional ACIP pile being a bored pile, however, the APGD pile is a displacement type that does not result in drilling spoil, which adds to the advantages of the conventional ACIP piles that is being vibrations-less, noiseless, fast installation, and, economically sound.

The APGD pile is constructed utilizing a special drilling tool that is capable of displacing the penetrated soil horizontally instead of excavating the soil and bringing the drilling spoil upwards. This is of critical importance in downtown, high traffic, and urban areas since it eliminates the need for fast spoil removal process. The method is superior, where the subsoil is contaminated, which is common in urban areas.

The lateral displacement of the penetrated material improves the friction resistance of the resulted pile. Drilling tools of different diameters and lengths, depending on the application, are currently used. The horizontal displacement of the penetrated soils could be at its immediate horizontal position as in loose to medium soils or after being transported upward to the displacement element as in medium to dense soils, NeSmith (2003). Most construction platforms are equipped with display unit that provides the drilling technician with the drilling tool depth, the grout pressure, the volume of the pumped grout, and the resisting torque. Once the displacement tool's tip reaches the desired depth, the downward travel of the tool is stopped and concrete pumping begins. Withdrawal of the tool starts when a target grout pressure is established. For a typical application in loose to medium granular materials, target installation pressures will generally be in the range of 69 to 103 kPa for lift off and shaft construction, (NeSmith, 2004). The withdrawal rate depends on the grout pressure and is usually varied to maintain the target grout pressure. The grout pressure is usually correlated to the grout ratio during the test piles installation. Although the grout volume is continuously checked to ensure that the delivered grout volume is greater than the theoretical pile volume, it should be emphasized that the pile is mainly cast based on the target grout pressure.

Recently, APGD piles were recommended as the most suitable and economic foundation system to support a high-rise building (35-story mixed use condominium) in Downtown Orlando. The initial pile load test program called for testing three piles in compression, of which, two test piles failed to support the pile design load. The test pile failure was referred to structural deficiency. The pile test program extended by recommending additional two test piles. Again, one of the additional test piles failed, where the other test pile was not tested.

At this stage, we were asked to evaluate the pile load tests and recommend future precautions. After evaluating the pile load tests and the subsoil conditions, four additional modified test piles were recommended. The modified test piles had longer cage with tighter stirrups that extended beyond the plastic clays. Three of the four test piles were, statnamically, tested in compression and were able to support the design load. The modification was recommended for the production piles.

This paper presents the details of the project, the encountered subsoil conditions, the initial and additional pile load test programs, pile load test data, analysis, and evaluation, possible reasoning behind the unexpected failures, and conclusions and recommendations for future studies needed to provide better understanding for the behavior of the APGD piles constructed through plastic clayey deposits.

LITERATURE REVIEW

A common feature of the APGD pile is the displacement tool that has the capability to provide somewhat horizontal displacement of the material penetrated (NeSmith, 2002). After the pile reaches the proposed tip elevation, grout/concrete is injected through a hollow stem while the auger is being withdrawn maintaining specific grout pressure.

If constructed in materials that has the ability to densify in response to lateral displacement, the APGD pile may result in increasing shaft and toe resistances higher than those developed during the installation of the conventional ACIP piles (NeSmith, 2002).

The ideal soil profile for the APGD piles is clean, granular, well-graded sand, loose near the surface, with a gradual uniform increase in density with depth. NeSmith (2002) stated that the existence of saturated fine-grained materials can impact pile constructability and quality in two ways: i) the spacing between piles must be increased to preclude interaction between piles, and ii) the generation of pore water pressure during construction in the vicinity of the pile, which can have negative effect on pile integrity.

Bustamante and Gianeselli (1998) provided specific guidelines for the design process of the APGD piles that included series of design curves for estimating shaft friction resistance based on Cone Penetration Test (CPT) and Pressuremeter Test (PMT) results. Toe resistance was estimated from an average CPT value near the pile toe. Rizkallah and Burns (1998) presented a shaft resistance that depends on the cone tip resistance and subsoil conditions. Dutch Standard MEN 6743 (1993) included guidelines for both the shaft and toe capacity of screw piles in granular materials that utilize the CPT results. NeSmith (2002) presented the results of 40 load tests performed on APGD piles in seven different geologic settings in the United States. The data collected during these tests was used to develop empirical relations correlating the tip and shaft resistance to both the CPT and SPT results.

NeSmith, W. and NeSmith, M. (2006) utilized the data acquired during the installation of the APGD piles to provide an adjunct to the geotechnical site characterization and as a verification tool for the design pile capacity. They introduced the terms "Installation Effort" and "Cumulative Installation Effort" which may be used to emulate the subsoil resistance and achieved pile capacity, respectively.

GENERAL CENTRAL FLORIDA GEOLOGY

The general geology of the Central Florida is usually characterized by sedimentary strata formed during three distinct geologic periods. The surficial stratum is composed of undifferentiated Holocene/Pleistocene/Pliocene age sands containing varying amount of silts and clays, which typically extend to depths in the order of 15 meters below ground surface. This upper mostly sandy zone contains the shallow surficial aquifer. A Miocene age deposit, the Hawthorn Group, underlies the surficial sands and is typically composed of clays, clayey sands, sandy clays, and sandy limestone containing appreciable amount of phosphates. This relatively impermeable stratum extends to depths of 45 to 60 meters beneath ground surface and serves as the confining layer (seal) for the underlying Floridan Aquifer. The Florida Aquifer, composed of Eocene age Ocala, Avalon Park, and Ocala City Limestones, is one of the most productive aquifers in the world.

THE REPORTED CASE HISTORY

The project site is located by Lake Eola in Downtown Orlando, Florida. The mixed-use retail, business, and residential tower consisted of 35-story reinforced concrete with attached 6-level parking garage topped with amenities deck. Ground surface elevation was near elevation +30 meters MSL (mean sea level) along the north portion of the site that is gently sloping towards the south portion to about elevation +29 meters NGVD. Column loads were provided by the structural engineer to be in the range of 9 to 22 MN.

Subsoil and Groundwater Conditions

The subsoil conditions within the project site were explored utilizing 4 Standard Penetration Test (SPT) borings to depths of 30.5 to 38 meters and 12 Cone Penetration Test (CPT) soundings to depths of 18 to 27.5 meters below grade, as recommended by the geotechnical consultant. It should be noted that the authors were not the geotechnical consultant for the project, however, the authors got involves at subsequent stage as explained hereafter. The average recorded SPT "N-Values" and cone tip resistance are plotted against depth as shown on Figure 1 along with a generalized description of the subsoil conditions. The soil column was mostly fine sand and slightly silty fine sands to about 13 meters underlain by plastic sandy clays to plastic clays to a depth of about 17 meters. The following layer was the Hawthorn Group, which consisted of a mix of clayey sands, sandy clays with occasional phosphates and traces of shells to about 25 meters transitioning to medium dense to dense sands with some limestone fragments and shells to the boring termination depth of about 38 meters. Groundwater (shallow not artesian) was encountered at depths of about 2.1 to 2.7 meters below grade, which corresponds to elevations of +26.8 to +27.7 meters MSL.

The Recommended Deep Foundation System

The original geotechnical report recommended Augered Pressure Grouted Displacement (APGD) piles as the most suitable and economic foundation system. An axial capacity of 1160 to 1340 KN was recommended for a 0.4-meter diameter 24.4-meter long pile with a cut-off of 3 to 4.6 meters below grade for the Tower portion and 1.5 to 3 meters for the parking structure. For 0.46-meter diameter pile of the same length, capacities in the order of 1340 to 1515 KN were recommended.

Conventional Augered Cast In Place (ACIP) piles were recommended as an alternative deep foundation system. Axial capacities of 800 and 1070 KN were recommended for 24.4-meter long pile with 0.4 and 0.46-meter diameter, respectively. For 30.5-meter long pile, axial capacities of 1070 to 1340 KN were recommended for 0.4 and 0.46-meter diameter, respectively.

The APGD piles were selected to support the building because of its ability to achieve higher capacities with the same pile geometry and to minimize the drilling spoil. The length of the steel cage was recommended to be about 7.6 meter.

Pile Loading Test Program

The original geotechnical report called for testing three piles in compression; two of 0.46-meter diameter (TP-1 and TP-2) and one of 0.4-meter diameter (TP-3). Table 1 summarizes the installation records for test piles TP-1, TP-2, and TP-3. The grout factor is defined as the ratio of the volume of the pumped grout/concrete to the theoretical volume of the pile. Test pile TP-1 showed very low grout factor of 1.03 compared to the test piles TP-2 and TP-3, which had grout factors of 1.14 and 1.20, respectively. It should be noted that Test Pile TP-1 was reported to be contaminated near the pile head. Grout cylinders were collected during pile installation and were tested to estimate the grout strength. The results showed that the grout's 7-day compressive strength exceeded the minimum required value of 75% of the 28-day strengths.

Loading Test Results

The tested piles were loaded according to the "Quick Load Test Method for Individual Piles" in general accordance with the "Standard Test Method for Piles under Static Axial Compressive Load" (ASTM D1143). The test plan was to load the test piles to 2.5 times the design load and

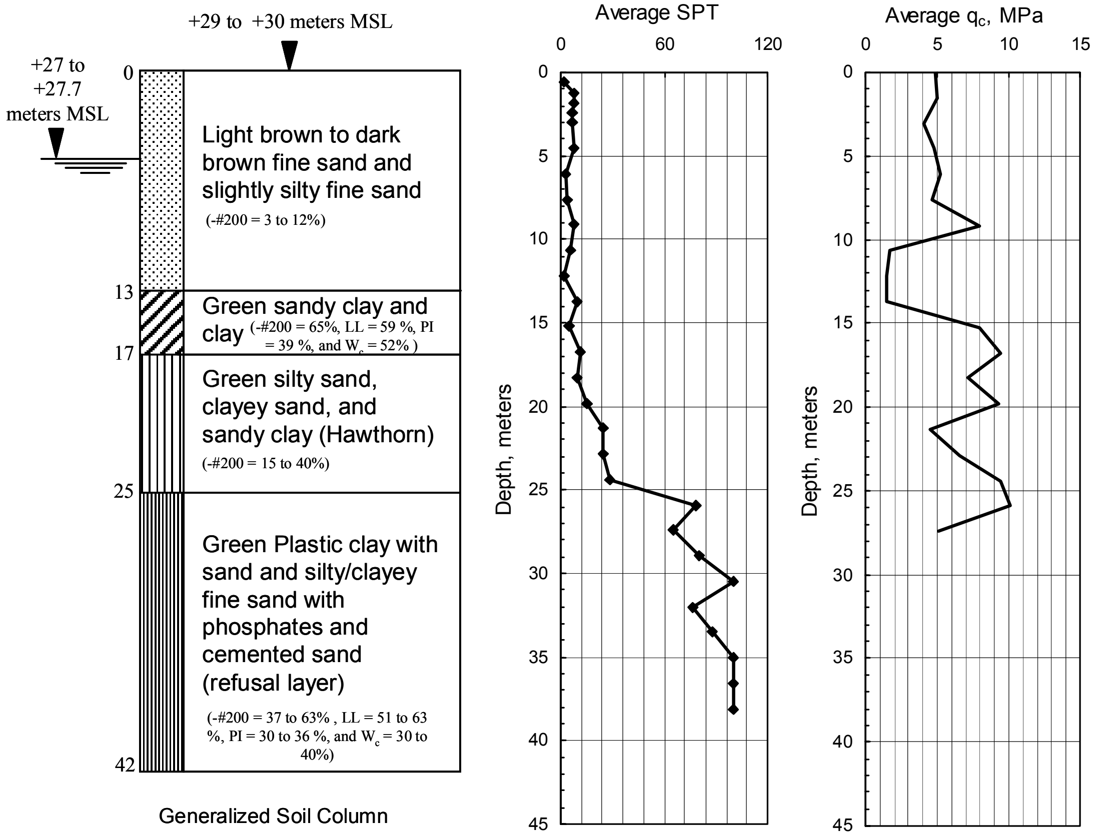

Figure 1. Generalized subsoil conditions, Average SPT "N-Values", and Average cone tip resistance versus depth.

hold the load for 30 minutes then to unload the test piles in equal increments. Strain gauges were placed at depths 2.7, 8.5, and 21.6 meters from the pile top as well as at the pile tip to estimate the load transfer profile. The resulted load-displacement curves as shown on Figure 2.

The results showed that test piles TP-1 and TP-2 failed prior to the proposed test load (2850 KN). Test pile TP-3 was, somewhat, able to withhold the design load with a pile head settlement of about 5.1 mm, however, the permanent settlement after unloading the pile was more than 19.1 mm (the limit recommended by the Florida Building Code).

Pile Integrity Testing (PIT) was conducted after the load tests were performed and no defects in the top half of the pile were detected.

The original geotechnical engineer and the piling contractor agreed that the failure is mostly due to pile structural deficiency and more specifically due to the contamination occurred in the top few feet of the piles.

Table 1. Installation records for test piles TP-1, TP-2, and TP-3

Test Pile	Pile diameter (meter)	Pile Length (meter)	Drilling Time (minute)	Approximate Torque at Tip (bars)	Grout Time (minute)	Grout Factor
TP-1	0.46	23.16	6	141	5	1.03
TP-2	0.46	21.95	5	144	5	1.14
TP-3	0.46	23.16	4	165	5	1.20

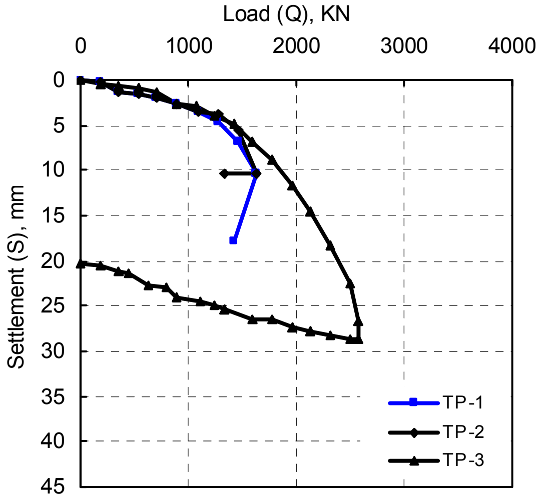

Figure 2. Load-Settlement curve for TP-1, TP-2, and TP-3

Additional Test Piles

After evaluating the failed pile load tests, the original geotechnical engineer recommended constructing two additional test piles (TP-4 and TP-5) close to the locations of TP-1 and TP-2 and to be tested statically in compression. Table 2 summarizes the additional test piles installation records.

The load-displacement curve resulted from the pile load test performed on TP-4 along with those for test piles TP-1, TP-2, and TP-3 are shown on Figure 3 for comparison purposes. The results indicated that TP-4 failed prior to achieving the desired test load (2850 KN) with a failure load of about 2140 KN. Again, the failure was attributed to structural deficiency in the pile.

Pile Integrity Testing (PIT) was conducted on test pile TP-4 after the load test was completed. The report stated that a relative reduction in pile impedance and/or relatively low soil resistance was encountered at approximately 11.6 meters below the pile top.

Because of the unexpected failure of test piles TP-1, TP-2, and TP-4 and the inadequacy of test pile TP-3, the proposed load test on the installed test pile TP-5 was put on hold. The production piles construction was postponed until an overall evaluation is performed.

POSSIBLE FAILURE REASONS

After the additional test pile failed to support the design load, we were asked to review and evaluate the load test results of test piles TP1, TP-2, TP-3, and TP-4 and to explore possible reasoning of the unexpected test piles' failure. Different reasons for test pile failure were explored and evaluated as follows:

i) Material deficiency: since grout cylinders had been collected and tested for compressive strength and the results showed that sufficient strength were achieved prior to testing the test piles, this reason was eliminated.

ii) Construction inadequacy or Structural Deficiency: it is known that the construction of the ACIP piles and also the APGD piles is very sensitive and operator dependent. Drilling time, grouting time, grout pressure during both lift off and shaft construction, and grout factor have appreciable impact on the quality and hence, the developed capacity of the final pile. As stated by NeSmith (2002) and as the construction technique of the APGD pile suggests, lateral displacements, hence, increase of lateral pressure is expected within the penetrated materials. If drilling is too fast, which is typical in the APGD piles construction, undrained loading conditions will be developed, which, in contractive soils, will result in immediate and sharp increase in porewater pressures. For sandy soils with high hydraulic conductivity, porewater pressures will immediately dissipate, which will reduce the risk of producing necking in the pile section. For plastic clays, such as those encountered at the project site between depths 12 to 15 meters, the hydraulic conductivity is very low, hence, the developed porewater pressure during penetrating the plastic clays will need long time to dissipate. If a high grout pressure of more than the developed porewater pressure during grout pumping is not maintained at all time, pile necking would be likely to occur at the locations of the high porewater pressure. This scenario, in our opinion, seems reasonable and is in general agreement with the structural deficiencies suggested by the original geotechnical engineer and the piling contractor. This scenario may also be confirmed noting that pile capacities interpreted from the failed load test results correlate well with a pile with half of the test piles' lengths, which in part shows that only the top half of the pile is effective.

Table 2. Installation records for test piles TP-4 and TP-5

Test Pile	Pile diameter (meter)	Pile Length (meter)	Drilling Time (minute)	Approximate Torque at Tip (bars)	Grout Time (minute)	Grout Factor
TP-4	0.46	23.16	5	192	7	1.42
TP-5	0.46	21.95	5	174	5	1.33

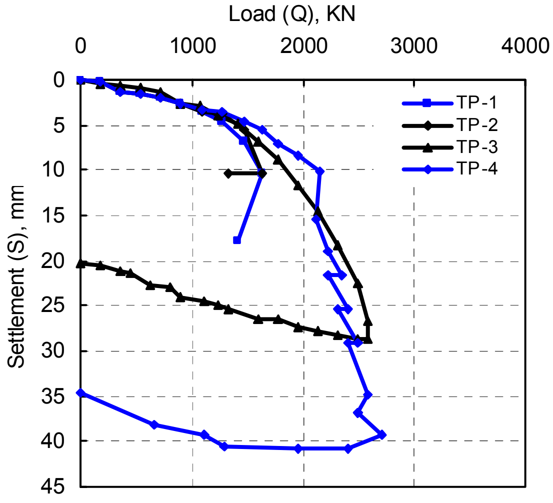

Figure 3. Load-Settlement curve for TP-1, TP-2, TP-3, and TP-4

iii) Eccentric Loading: slight differences between deflections recorded by the dial gauges used to monitor the pile load tests were reported, which indicated that small eccentricities were existed. Eccentricities will trigger the structural failure especially if necking was occurred at sections that was not reinforced (depths of 12 to 15 feet).

MODIFIED PILES AND ADDITIONAL TESTING

We recommended constructing four additional test piles (TP-6, TP-7, TP-8, and TP-9) to be tested in compression. The installation records for the additional modified test piles are shown in Table 3.

Longer reinforcement cages with tighter-spaced stirrups within the plastic clay deposits were proposed for test piles TP-8 and TP-9 to provide additional lateral support in order to counteract the porewater pressures that expected to be developed within the plastic clays. Test piles TP-6 and TP-7 were not modified.

Test piles TP-7, TP-8, and TP-9 were tested using the Statnamic Testing Procedure (ASTM WK7120 "Standard Test Method for A Pile Under an Axial Compressive Force Pulse") because time was in essence. The load-displacement curves for the three load tests performed are plotted as shown on Figure 4 and the curves were re-plotted along with previous load tests as shown on Figure 5.

Results of pile load tests TP-7, TP-8, and TP-9 were analyzed utilizing: i) Davisson Offset Limit Load, ii) De Beer Yield Load, iii) Hansen 80% Criterion, iv) Chin-Kondner Extrapolation, v) Mazurkiewicz Method, vi) Decourt Extrapolation Methods, and vii) Florida building Code (2004) criterion.

The results showed that test piles TP-8 and TP-9 were able to support the design load providing a minimum safety factor of two. Test pile TP-7, with the short cage length, barely supported a design load of 1245 KN, which clearly confirmed that the longer cage with tighter-spaced stirrups through the plastic clays improved the pile performance. Recommendations were made to construct the production piles of 0.46-meter diameter and a minimum length of 23 meters unless refusal encountered to provide a 1425 KN compression capacity. The reinforcement cage was recommended to extend not less than 15.25 meters, which will ensure penetrating the plastic clay deposits to provide, along with tighter-spaced stirrups, sufficient resistance to the lateral pressure that may result from the high porewater pressures developed during the constructing the APGD piles through the plastic clays.

CONCLUSIONS

A case history in which APGD piles were used to support a high-rise building in Downtown Orlando, Florida was presented. Test piles, originally recommended, failed to carry the design compression load. The reason behind the test piles failure was not clear, however, the assumption that structural deficiency occurred due to pile necking at the zone of plastic clays was generally accepted. Plastic clays are contractive soils that exhibit large increase in porewater pressures under undrained loading conditions, which is usually present during the APGD piles installation, especially if drilling through the plastic clays was fast.

Table 3. Installation records for test piles TP-6, TP-7, TP-8, and TP-9

Test Pile	Pile diameter (meter)	Pile Length (meter)	Cage Length (meter)	Drilling Time (minute)	Approximate Torque at Tip (bars)	Grout Time (minute)	Grout Factor
TP-6	0.46	23.47	9.14	12	240	8.5	1.12
TP-7	0.46	20.73	9.14	5.75	170	8	1.43
TP-8	0.46	23.16	15.24	8.67	165	7.33	1.35
TP-9	0.46	23.16	18.29	45	165	7.5	1.21

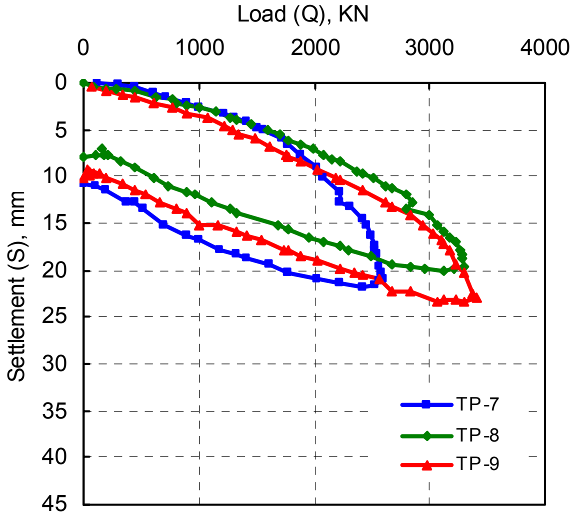

Figure 4. Load-Settlement curve for TP-7, TP-8, and TP-9

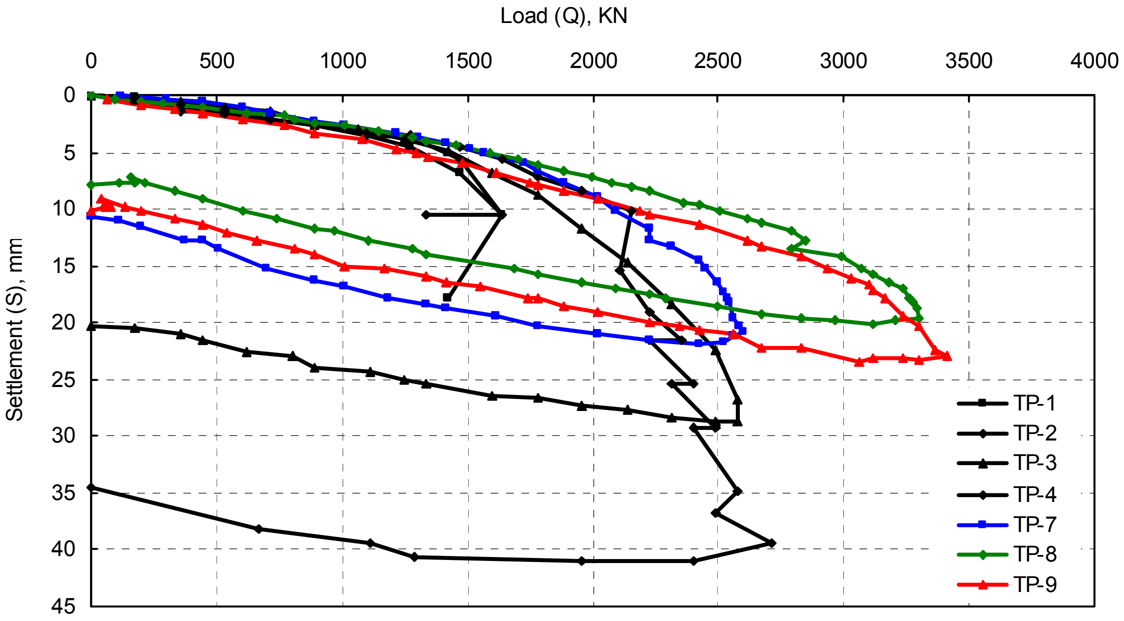

Figure 5. Load-Settlement curve for test piles TP-1, TP-2, TP-3, TP-4, TP-7, TP-8, and TP-9

Extending the reinforcement cage to penetrate through and pass the plastic clays zone with tighter-spaced stirrups through the clays, resulted in better pile performance due to the provided lateral support at the critical zone of the plastic clay, which, reduced the risk of pile necking.

The presented case history showed that the subsoil column is a vital factor and should be of great concerns during the design and the construction of APGD piles especially if saturated fine-grained soils are encountered.

There is a need for a thorough experimental and/or theoretical study for the effect of soft and plastic clays on the overall behavior of the APGD piles. Experimental laboratory and/or large scale models and studies may provide reasonable qualitative results. Three dimensional finite element modeling with transient pore pressure simulation and dynamic module capabilities should provide means to simulate the construction procedure of the APGD piles and hence, a parametric study may then be carried out to evaluate the effect of soft soils, construction procedure, operator dependency, and other factors on the pile performance.

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