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Static Capacity Analysis of Augered, Pressure-Injected Displacement Piles

Willie M. NeSmith, * P.E., Member, ASCE

*Chief Geotechnical Engineer, Berkel & Company Contractors, Inc., P.O Box 335, Bonner Springs, Kansas 66012

Abstract

Augered, pressure-injected displacement (APID) piles develop significantly higher capacities than conventional augercast (APG) piles in loose to medium dense, granular materials. During APID installation, a Ko to Kp environment is created in the materials surrounding the pile immediately prior to casting, and this condition results in increased shaft resistance in soils where displacement leads to densification. Current augercast design methodologies assume a Ko to Ka condition in the vicinity of the pile, and thus the body of knowledge associated with conventional APG piles would be expected to be a conservative base line from which to begin assessing the capacity APID pile systems.

This paper offers an empirical design methodology for APID piles, based on evaluation of 28 load tests, primarily in granular materials and in a variety of geologic settings. A design process that includes modification for soil types within the coarse-grained range is presented for CPT and SPT exploratory information.

Introduction

The new ASCE Standard Guidelines for the Design and Installation of Pile Foundations will include a definition of and installation guidelines for "Drilled Displacement Piles." The use of drilled (or augered) displacement piles is a relatively new concept in the United States, and the technology that led to the processes currently being used emerged in the 1960s as the Atlas and Fundex systems. Refinements to the processes, and advances in equipment, led to the current generation of drilled displacement systems that have been used in Europe since the late 1980s.

There are significant differences in tooling among the systems now in use. A common feature however, is a displacing element that provides for the horizontal movement of the material penetrated. After the pile area has been evacuated, the piles may be cast using concrete or grout, which may be tremied or pressure injected through the displacement tooling. In those materials that densify in response to displacement, the combination of lateral displacement, and concrete or grout cast against the soil, results in shaft resistances higher than those developed with traditional cast-in-place systems.

A great deal of literature has been devoted to the evaluation of static capacity of drilled shafts and APG systems. In the construction of either of these systems, the soil in the vicinity of the foundation member is brought (to some degree) toward an active state. Thus, conventional cast-in-place design methodologies should provide a lower bound for the assessment of drilled displacement systems.

Although the use of drilled displacement systems is a fairly mature technology in Europe, relatively little has been published with respect to capacity analysis. Work by Bustamante and Gianeselli (1993) included analysis of 24 load tests on Atlas and “cased screw” piles; however, these systems differ significantly from those currently available in the United States, and in only 8 cases were the soil conditions desirable for drilled displacement systems. A load test program conducted on a drilled displacement system at Auburn University’s NGES test site (Brown and Drew, 2000) provided an indication of the efficacy of the approach in Piedmont residuum.

This paper exhibits a summary of 28 load tests on an augered displacement system in 7 geologic settings. The data from these tests has been used to develop empirical correlations between load transfer components and the results of Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT).

Installation of drilled displacement piles

The system addressed in this paper is an augered, pressure-grouted displacement pile. The displacement tool for this system is shown in Figure 1, and the installation platform is shown in Figure 2. Currently, tools ranging from 0.31 meter (12 inches) to 0.46 meter (18 inches) in diameter are available. The auger section is typically about 0.9 meter (3 feet) in length, but may vary depending upon application. The installation platform includes a vertical mast with an attached turntable capable of producing 25 meter-tons (180,000 ft-lbs) of torque, and a system of cabling that allows a downward force (crowd) of 356 kN (40 tons) to be placed on the tools. The current equipment allows for installation to a maximum depth of 24 meters (79 feet).

As the tool is advanced, the material penetrated is displaced horizontally, either at its original horizontal position (in loose to medium soils), or after being transported upward by the auger to the displacing element (in medium to dense soils). In either case, material in the auger flighting is compressed by being forced to the ramp area and displacing element. Because the auger flighting is packed with material, there is an outward force in the vicinity of the auger section. Some densification can occur around the auger section and there is, at worst, neutral displacement around the auger.

The depth of the tip is displayed in the operator’s compartment, and when the desired tip level is reached, downward travel of the tool is stopped and pumping of grout is begun. A pressure monitoring mechanism consisting of a piston-type sensor and transducer are mounted at the top of the tools, and real-time grout pressure is 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.

A target “lift off” pressure and a pressure range for shaft construction are set during probe pile and test pile installation, and a relationship between installation pressure and grout volume is established. For a typical application in loose to medium granular materials, target installation pressures will generally be in the range of 138 to 207 kPa (20 to 30 psi) for lift-off and 69 to 103 kPa (10 to 15 psi) for shaft construction.

Grout volume is checked in the same fashion as it is for APG construction; however, the piles are constructed based on grout pressure. Ratios of pumped grout volume to the calculated volume of the area evacuated are typically in the range of 1.10 to 1.15. Turntable torque, as indicated by hydraulic pressure of the drive system, is also displayed and recorded.

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 auger and the displacing element act as a packer, and the grout is confined below the level of the displacing tool, Figure 3. Thus, grout return typically occurs only when the tool exits the ground, and there is not an "observed head", common in specifications for conventional APG applications. As the tool nears the surface, the grout pressure is allowed to decrease, and pumping of grout may be stopped at a point prior to the time the tip if the tool exits from the ground. While lateral fracturing of the near-surface soils is possible if installation grout pressures are maintained near the surface, most often, the grout bypasses the tool, and issues from the pile location. This is wasteful, and can be dangerous, since grout has been seen to erupt from the pile location to a height in excess of 3 meters (10 feet).

Near-surface grout pressures, and the level at which pumping is stopped are determined during the test pile program, and modified as appropriate during production installation. The goal is to have some grout flow from the pile as the tool exits, but not to have a geyser of grout.

The process of determining lift off and installation pressures, and procedures used as the tool nears the surface have been developed based on observation of field operations. The basic operating position is that the pile could be cast by tremie, and that the overpressure (i.e. the pressure sensed by the transducer at the top of the tools) is a mechanism for assuring that there is adequate grout flow through every segment of the pile, regardless of the material penetrated. It was observed that, with a constant rate of withdrawal, grout pressures were high in dense sands, and dropped as the tool passed through loose sand or soft fine-grained materials. The technique of varying the withdrawal rate to maintain a target pressure, and the use of the target lift off and shaft installation pressures noted, have evolved from correlation of installation observations with the results of load tests.

In addition to the real-time display of depth, grout pressure and torque, an on-board printer produces a permanent record of each installation. An annotated reproduction of a printout (with torque omitted for clarity) is shown on Figure 4. In the case illustrated, lifting of the tool occurred when the measured grout pressure was approximately 228KPa (33 psi), and was accompanied by a rapid drop in grout pressure. The withdrawal rate was then adjusted to maintain the grout pressure in the target range.

The spacing between fresh piles required to preclude interaction is a function of the subsurface materials being penetrated. Piles as close as 3 pile diameters center to center have been installed in loose to medium clean sands with no detectable interaction. Larger spacing is needed where saturated, dirty granular materials or saturated fine-grained materials are present in the profile, and spacing as large as 12 pile diameters center to center have been required. A spacing of 6 pile diameters center to center is typical. Initial production installation spacing is based on observations during test pile installation, and is modified as appropriate.

From a materials standpoint, the grout mixes for the displacement system are the same as those used for augercast piling, and similar quality assurance processes are appropriate. Reinforcing steel configurations and insertion processes are the same as those used in APG work.

Application

The ideal soil profile for the system under consideration would be clean, angular, well-graded sand, loose near the surface, with a gradual, uniform increase in density with depth. While such profiles may exist, in practice, there is almost always some complicating feature. With APID piling, one should first examine the depth to which the system can be reasonably installed, not the depth that is needed in order to achieve a predetermined capacity. While it may be possible to penetrate deeply into a very dense sand strata, doing so may slow production, cause excessive tool wear, and overstress the installation platform. Additionally, such features may make it difficult to withdraw the tools, which can impact the quality of the piles.

The practical drilling limit depends upon the nature of the installation platform but in general, drilling and withdrawal become difficult at cone tip resistances (q_c) of about 14 MPa (150 tsf), and very difficult at 19 MPa (200 tsf). Very dense layers less than about 2 meters (6.6 feet) thick can usually be readily penetrated. Very dense layers in excess of 3 meters (10 feet) thick however, normally are a problem during evacuation of the pile area and casting, and extending the tooling through such features should be avoided.

Another issue that must be considered is the occurrence of saturated, fine-grained materials, which can impact production and pile quality in two ways. As discussed previously, where piles are extended through saturated, fine-grained materials, the spacing between fresh piles must be increased to preclude interaction. The parameters involved in this issue have been only generally quantified, in that the thicker and softer the fine-grained zone, the greater the spacing must be to preclude interaction. The extreme case observed to date was a spacing of 12 pile diameters center to center where a zone of soft silty clay was present from approximately 3 meters (10 feet) to 6 meters (20 feet) below the ground surface.

The generation of pore pressure in the vicinity of the pile is another result of penetrating saturated dirty granular materials and saturated fine-grained materials. Excessive bleeding of fresh piles manifests the build up of excessive pore pressure. This is troublesome from a construction standpoint, and can have a negative impact on pile integrity. This issue can be overcome by staging the installation in such a way as to preclude the build up of excess pore pressure;

however, pile locations must be available over a relatively large area. Where there is a high density of piles in a single element (tank foundations for example), the number of piles that can be installed in any given time period may be governed by pore pressure issues rather than the actual time required to go through the installation process. More data is needed in order to begin to quantify the relationship between the build up of pore pressure during pile installation and the thickness and nature of saturated fine-grained material being penetrated.

Berkel Database

This evaluation has been based on the results of 28 load tests (22 compression and 6 tension) at 19 different sites. All piles were installed by Berkel & Company Contractors, Inc. The sites represent a wide area geographically and a variety of geologic settings, but in all cases the major load-carrying features were granular. The database distribution by geologic setting is exhibited in Table 1.

Table 1. Sites by Geologic Setting

Geologic Setting	Sites	Major Features
Alluvium, Major River (AR, CA, FL, IA, WA)	5	Loose to dense sand, some gravel, well-graded (primarily), clean to some silt and clay
Post Miocene (FL)	4	Loose (primarily) to medium silty, clayey sand
Barrier Island (FL, AL, MD)	4	Medium to very dense sand, uniform, clean
Piedmont (GA)	3	Loose (primarily), silty sand/sandy silt, micaceous—Toe in partially weathered rock.
Glacial Outwash (MN)	1	Loose to medium sand with fine gravel, clean, well-graded
Gulf Coastal Plain (FL)	1	Loose to medium silty clayey sand
Colma (CA)	1	Medium to very dense silty and clayey sand

Pile diameters range from 0.36 to 0.46 meter (14 to 18 inches), however about 80% were 0.41 meter (16 inches). Pile depths ranged from 6 to 21 meters, and averaged approximately 13 meters (43 feet). Twelve of the compression load tests had either strain gauges or tell tales. Fourteen of the tests were performed using the Quick loading option of ASTM D 1143. The remaining tests were performed using the Standard loading procedure, some with one or more cycles.

Previous Work

Drilled displacement piles have evolved as proprietary processes both in Europe and the United States, and little information relative to design has been published. Work by Bustamante and Gianeselli, undertaken on behalf of Laboratoire Central des Ponts et Chaussees (LCPC), provides the most information on methodologies with characteristics similar to the system addressed in this paper. However, of the 24 tests referenced, none were apparently in an environment where granular material provided the primary shaft and toe resistance. The referenced information does indicate that for 13 of the tests, silt was the relevant shaft component material.

A series of curves (reproduced in part on Figure 5) was produced for calculation of shaft resistance with curve selection based on a range of cone penetration test (CPT) and pressuremeter test (PMT) values. The shaft component is determined by relating CPT values (and SPT values) to unit shaft resistance along the appropriate curve. The CPT and PMT data provided the basis for the work, and SPT correlations were based on a q_c/N of about 4 for sands and gravels. Toe resistance is calculated by multiplying a factor ranging from 0.5 to 0.75 times the toe CPT value (or 1.8 to 2.1 times the toe SPT value).

Dutch standard NEN 6743 contains guidelines for shaft and toe capacity of screw piles with grout in granular materials. Shaft resistance is calculated as the product of the CPT value and a factor of 0.009. Toe resistance is calculated in a manner similar to the Bustamante and Gianeselli process except shape factors are included and a value of 0.80 times the toe CPT value is used. The reader is referred to the referenced literature for details for calculating the toe CPT value.

Results of a study at Auburn University's National Geotechnical Experimentation Site (NGES) at Spring Villa (Brown and Drew) on a drilled displacement system yielded unit shaft resistance values of $0.013q_c$ and $0.026q_c$ for an isolated pile and the center pile of a five-pile group with piles spaced at three pile diameters, respectively. An average q_c value of 3.5 MPa was used in establishing this relationship. The toe component was assumed to be absent in the referenced study.

Delineation of Ultimate Load and Load Components

From the early load test results on the augered, pressure-grouted, displacement pile system considered in this work, it was clear that the piles consistently produced broad, sweeping load-displacement relationships with no clear "failure". This is not surprising given the nature of the system and the fact that the major load development elements for shaft and toe resistance were primarily granular. This behavior is also consistent with the performance of cast-in-place systems in granular materials generally. The other recurring feature was that a displacement rate of 0.057 mm/kN (0.02 in/ton) often marked the beginning of a rapid increase in displacement rate. It was found that if the interpreted failure load (IFL) is defined as the lesser of 1) the load at which 25.4 mm (1 in) of gross pile butt movement occurs, or 2) the load at which the displacement rate reaches 0.057 mm/kN (0.02in/ton), movements at IFL/2 were typically less than 6.4 mm (0.25 in.) or less. These criteria were applied to the load test data to determine the IFL. The limiting displacement rate and movement occasionally occur at about the same load, but the displacement rate did not govern in any case.

Where the load tests were not taken to the IFL the load displacement relationship was extrapolated using the method described by Chin. This process, in the author's experience, is a reliable tool for examining the behavior of cast-in-place systems in granular materials, if the limitations of the process are recognized, and, in particular, with adjustment for elastic compression of the pile (Fleming). This process was used to estimate the shaft and toe components where no instrumentation was available.

Shaft Resistance

The results of the shaft resistance evaluation are given on Figs. 5 and 6. The Bustamante and Gianeselli curves for screw piles fit the data reasonably well, and the trend of the NEN 6743 relationship is consistent with that of the CPT data. The scatter in the data is not random however, and there is a consistent trend for clean, well-graded, angular materials to plot at the upper end of the data range. Based on the available data, a relationship that includes consideration of fines content, grain size distribution, and particle angularity, is appropriate for the system under consideration. The following relationships are proposed for estimating unit shaft resistance ($\boldsymbol{f}_s$) based on CPT cone resistance ($\boldsymbol{q}_c$) and uncorrected SPT blow count ($\boldsymbol{N}$):

$$\boldsymbol{f}_{scpt} = 0.01\boldsymbol{q}_c + \boldsymbol{w}_s \qquad \boldsymbol{q}_c < 19 \text{ MPa (200 tsf)} \qquad (1)$$

$$\boldsymbol{f}_{spt} = 0.005\boldsymbol{N} \text{ MPa } (0.05\boldsymbol{N} \text{ tsf}) + \boldsymbol{w}_s \qquad \boldsymbol{N} < 50 \qquad (2)$$

For uniform, rounded materials with up to 40 percent fines, $\mathbf{w}_s$ should be taken as zero, and a limit shaft resistance of 0.16 MPa (1.7 tsf) applied. For well-graded, angular materials with less than 10 percent fines, $\mathbf{w}_s$ should be taken as 0.05 MPa (0.5 tsf) with a limit shaft resistance of 0.21 MPa (2.2 tsf). Interpolation should be used for intermediate materials.

The three data points on Figure 6 that plot near 0.31 MPa (3.1 tsf) are cases where the auger section of the displacement tool was extended into partially weathered rock or very dense gravel. These are included for reference only.

This relationship should be applied only to granular materials in which displacement will result in densification. Such a response can be assumed in loose to medium dense, clean granular materials, but the fines content above which these materials are no longer free draining may not be readily definable. The most definitive method for assessing marginal materials is to perform cone penetration testing with pore-pressure measurement (CPTU). If there is not a pore pressure response, then densification upon displacement is possible.

Where displacement piles have been extended through unsaturated, low-plasticity, fine-grained materials, shaft resistances greater than would be expected with augercast piles have been noted. However, it is suggested that the shaft contribution of such materials be estimated by conventional methods and not by using the above relationships.

Toe Resistance

Unit toe resistance is plotted verses CPT and SPT values on Figures 7 and 8, respectively. For 6 of the load tests in the database, the pile toe was in granular material with high fines content, or in fine-grained material. These have been included in the figures but isolated. Also shown are cases where the auger section was extended into partially weathered rock or extremely dense gravel. For the remaining cases, the pile toe was in free-draining granular material with varying fines content. It was expected that toe resistances would be somewhat higher than those that would be calculated for augercast piles, as a result of the densification of the material above the pile toe. The SPT data is consistent with that view. Unit shaft resistance based on 0.383N MPa (1.9N tsf) (Neely) provides a reasonable lower bound for the displacement data. As was the case with shaft resistance, the cleaner, more well-graded, angular materials demonstrated relatively higher toe resistances, and the suggested methodology takes this into consideration.

The CPT-based data, on the other hand, was consistently below the Bustamanate and Gianeselli lower limit of 0.5qc. Their study included only systems with a sacrificial point or lost shoe, and it seems reasonable that the toe response in a system of that type would be stiffer than that of a pile toe formed in a manner similar to conventional augercast processes. The following relationships are proposed for estimating unit toe resistance ($\boldsymbol{q}_t$) based on CPT cone resistance ($\boldsymbol{q}_{cm}$) and uncorrected SPT blow count ($\boldsymbol{N}_m$).

$$\boldsymbol{q}_{tcpt} = 0.4\boldsymbol{q}_{cm}+\boldsymbol{w}_t \qquad \boldsymbol{q}_c<19 \text{ MPa (200 tsf)} \tag{3}$$

$$\boldsymbol{q}_{tspt} = 0.19\boldsymbol{N}_m \text{ MPa } (1.9\boldsymbol{N}_m \text{ tsf})+ \boldsymbol{w}_t \qquad \mathbf{N}<50 \tag{4}$$

For uniform, rounded materials with up to 40 percent fines, $\mathbf{w}_t$ should be taken as zero, and a limit toe resistance of 7.2 MPa (75 tsf) applied. For well-graded, angular materials with less than 10 percent fines, $\mathbf{w}_t$ should be taken as 1.34 MPa (14 tsf) with a limit toe resistance of 8.62 MPa (89 tsf). Interpolation should be used for intermediate materials.

The penetration test terms $\boldsymbol{q}_{cm}$ and $\boldsymbol{N}_m$ are modified values that are indicative of conditions in the vicinity of the pile toe, rather than at the level of the pile toe. There are a variety of methods for computing these modified values, but they all seek to address two fundamental issues. First, toe capacity is influenced by the characteristics of the material at some distance above as well as below the pile toe. Second, variations in penetration resistance within the zone which influences toe resistance should be treated conservatively, and the processes seek to mitigate the impact of local highs in penetration resistance, while taking into account local lower values.

In the references cited which deal with this issue, the processes for calculating the penetration value to be used in the estimation of toe resistance are somewhat involved. They have, however, evolved from a long history of estimating pile capacities based on CPT work wherein there is a much better definition of conditions near the pile toe than with SPT data. Those accustomed to estimating toe capacity based on SPT data on 1.5-meter (5-foot) intervals may view these processes as unnecessarily complex. They are, however an appropriate treatment for CPT data, and imply the need for a conservative approach when using SPT data.

For this work, the process described by Fleming and Thorburn was used, except that the zone of influence was taken as 4 diameters above and below the pile toe. The decision to take this approach was not the result of a comprehensive evaluation of available methods; rather, it was a seemingly reasonable compromise.

Conclusion

A database of 28 load tests on augered, pressure-injected displacement piles was evaluated to develop empirical correlations between field penetration test data (CPT and SPT) and load transfer components in granular materials. The data indicates that the shaft and toe components of the system evaluated are higher than those that would be calculated by conventional augercast design methodologies. The data also suggests the fines content, grain size distribution and particle shape are significant factors in both the shaft and toe components of displacement systems, and these factors, taken as a whole, are included in the suggested design process. The relative impact of each of these elements has not been quantified, and should be addressed in future studies.

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