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Design and Installation of Pressure-Grouted, Drilled Displacement Piles

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The use of drilled (screwed) displacement piles is an emerging sector of the United States deep foundation industry. During installation of these systems, a Ko to Kp environment is created in the materials surrounding the pile immediately prior to casting, and this condition results in increased shaft and toe resistance in soils where displacement leads to densification. Current continuous flight auger and drilled shaft design methodologies in the United States assume a Ko to Ka condition in the vicinity of the pile, and are thus an inappropriate starting point for static capacity analysis of drilled displacement piles.

This paper examines constructibility issues, and the sensitivity of performance to installation, and offers an empirical design methodology for drilled displacement piles. The proposed design process has been based on evaluation of 40 load tests, primarily in granular materials, and in a variety of geologic settings, and includes modification for soil types within the coarse-grained range.

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”, and a subcommittee has been formed within the Deep Foundation Institute (DFI) to prepare documents that will facilitate the application of these systems. Although the use of drilled (or screwed) displacement piles is a relatively new concept in the United States, 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 more or less 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 and toe resistances higher than those developed with traditional cast-in-place systems.

A great deal of literature in the U.S. has been devoted to the evaluation of static capacity of drilled shafts and augered, cast-in-place systems. In the construction of either of these systems, the material that occupies the future pile location is transported to the surface, and the soil in the vicinity of the foundation member is brought (to some degree) toward an active state. Given the way the pile area is evacuated, and the impact on the surrounding soil, conventional cast-in-place design methodologies are an inappropriate starting point for the assessment of drilled displacement systems.

Although the use of drilled displacement piles is a fairly mature technology in Europe, information relative to capacity analysis of the current generation of processes is rather limited compared to other deep foundation systems. Work by Bustamante and Gianeselli (1998) and Rizkallah and Burns (1998) do provide specific guidelines for design processes. Drilled displacement systems are essentially absent from the U.S. literature.

This paper exhibits a summary of 40 load tests on a drilled displacement system in 7 geologic settings in the United States. 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

The system addressed in this paper is a drilled, pressure-grouted displacement pile. The displacement tool for this system is shown in Figure 1.

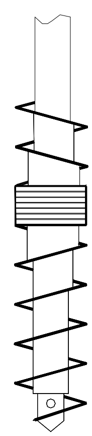

Figure 1. Berkel Displacement Tool

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 (Fig. 2) is typical of those used for European CFA installation. It 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 equipment currently used for this process allows for installation to a maximum depth of 24 meters (79 feet). The system is adaptable to larger equipment, which would increase the maximum depth and the largest diameter that could be installed.

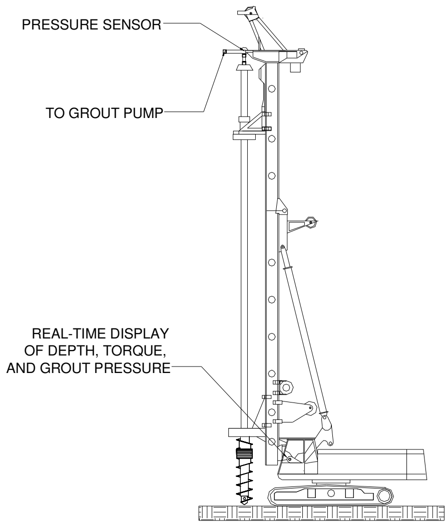

Figure 2. Installation Platform

During installation, 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. Slow rotation of the tool is maintained, and pumping of grout is begun. A pressure monitoring mechanism, consisting of a piston-type sensor and transducer, is mounted at the top of the tools, and real-time grout pressure is displayed in the operator's compartment. When the target lift off 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.

As the tool approaches the surface, the grout pressure is allowed to drop, and pumping is stopped when the tip of the tool is about 2 meters (6 feet) below the surface. The upper portion of the pile is then cast by tremie.

Grout volume is checked in the same fashion as it is for continuous flight auger construction; however, the piles are constructed based on grout pressure. Grout over supply is 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 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 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.

The spacing between fresh piles required to preclude interaction has been shown to be 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 conventional augercast 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 drilled displacement 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. Extending the tooling through such features should be avoided. The current limits will be extended somewhat with the use of more powerful equipment and more robust tooling.

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 40 load tests (32 compression and 8 tension) at 25 different sites. All piles were installed by Berkel & Company Contractors, Inc. The sites represent a wide area geographically and a variety of geologic settings; however, the sites are predominantly in relatively young deposits. In all cases, the major load-carrying features were granular. The database distribution by geologic setting is exhibited in Table 1.

Pile diameters range from 0.36 to 0.46 meter (14 to 18 inches), however about 65 percent were 0.41 meter (16 inches). Pile depths ranged from 6 to 21 meters, and averaged approximately 13 meters (43 feet). Fifteen of the compression load tests had either strain gauges or tell tales. Twenty-eight 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.

Table 1. Sites by Geologic Setting

Geologic Setting	Sites	Major Features
Alluvium, Major River (AR, CA, FL, IA, WA)	7	Loose to dense sand, some gravel, well-graded (primarily), clean to some silt and clay
Post Miocene (FL)	5	Loose (primarily) to medium silty, clayey sand
Barrier Island (FL, AL, MD)	5	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
Atlantic Coastal Plain (SC)		Loose to medium silty clayey sand.
Gulf Coastal Plain (FL)	1	Loose to medium silty clayey sand
Colma (CA)	3	Medium to very dense silty and clayey sand

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. Bustamante and Gianeselli (1998), provides information on methodologies with characteristics similar to the system addressed in this paper. A series of curves 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 (or PMT) values to unit shaft resistance along the appropriate curve. The relevant CPT curves (Q1, Q4, Q5) where adjusted for mechanical versus electric cone data as recommended in the referenced work, and the resulting relationships are shown on Fig. 3. Toe resistance is given as the product of an adjusted cone tip value in the vicinity of the toe, and a factor that ranges from 0.5 to 0.75. A plot of the lower bound value, adjusted for electric cone data, is shown on Fig. 4.

Rizkallah and Burns (1998) presents a shaft resistance relationship based on cone tip resistance and material type. The relationship for silty sand is shown on Fig. 3. Toe resistance is given as discrete values for a range of cone penetration test values. This relationship is shown on Fig. 4.

Dutch standard NEN 6743 (1993) 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.

ULTIMATE LOAD AND LOAD COMPONENTS

From the early load test results from the pile system considered in this paper, 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.

Implementation of the displacement system considered in this paper included the development of ultimate load criteria that would be palatable to Geotechnical Engineers involved in the projects on which it was used. Currently, the use of Davisson’s Limit (an inappropriately conservative process in the Author’s opinion) is common, and a displacement of 0.10 times the pile diameter is almost never used. A recurring feature was of the load displacement relationships for the system was that a displacement rate of 5.7 cm/MN (0.02 in/ton) often marked the beginning of a rapid increase in displacement rate. It was also found that this displacement rate occurred near, or after the load at which the displacement reached 0.06 times the pile diameter. If interpreted ultimate load (IUL) is taken as the lesser of these criteria, displacement at IUL/2 is typically less than 6.4 mm (0.25 in.). A displacement at the working load in this range was found to be acceptable in most cases. These criteria were applied to the load test data to determine the IUL. The limiting movement governs most often, and the limiting movement and displacement rate occasionally occur at about the same load. The displacement rate governed only when there was a "soft toe" condition.

Where the load tests were not taken to the IUL, the load displacement relationship was extrapolated using the method described by Chin (1970). This process, in the author's experience, is a reliable tool for examining the behavior of cast-in-place systems in granular materials, and was used to estimate the shaft and toe components where no instrumentation was available.

Shaft Resistance

It can be seen on Fig. 3 that the results of the shaft resistance evaluation of the Berkel database are reasonably consistent with the referenced procedures. 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 high end of the shaft resistance data range. This observation has only recently come to light, and the individual impact of angularity, grain size distribution and fines content has been defined.

Based on the data available to this point, 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, $\mathbf{w}_s$ may 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 Fig. 4 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 Fig. 5 and 6, respectively. It was expected that toe resistances for this system 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 (1991), provides a reasonable lower bound for the displacement data. Note that the referenced relationship was based on displacement equal to 0.10 times the pile diameter. 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, is, for the most part, well below both the Bustamanate and Gianeselli lower bound, and the Razkallah and Burns relationship. In the referenced works, ultimate load was defined as 0.10 times the piles diameter. The current evaluation uses a displacement of 0.06 times the pile diameter, and a part of the difference in the relationship is attributable the difference in the ultimate load criteria. However, the referenced relationships included only systems with a sacrificial point or lost shoe. It seems reasonable that the toe response in systems with that feature 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 ($\boldsymbol{N}_m$).

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

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

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, $\mathbf{w}_t$ may 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. For this work, the process described in Fleming and Thorburn (1983) was used, except that the zone of influence was taken as 4 diameters above and below the pile toe.

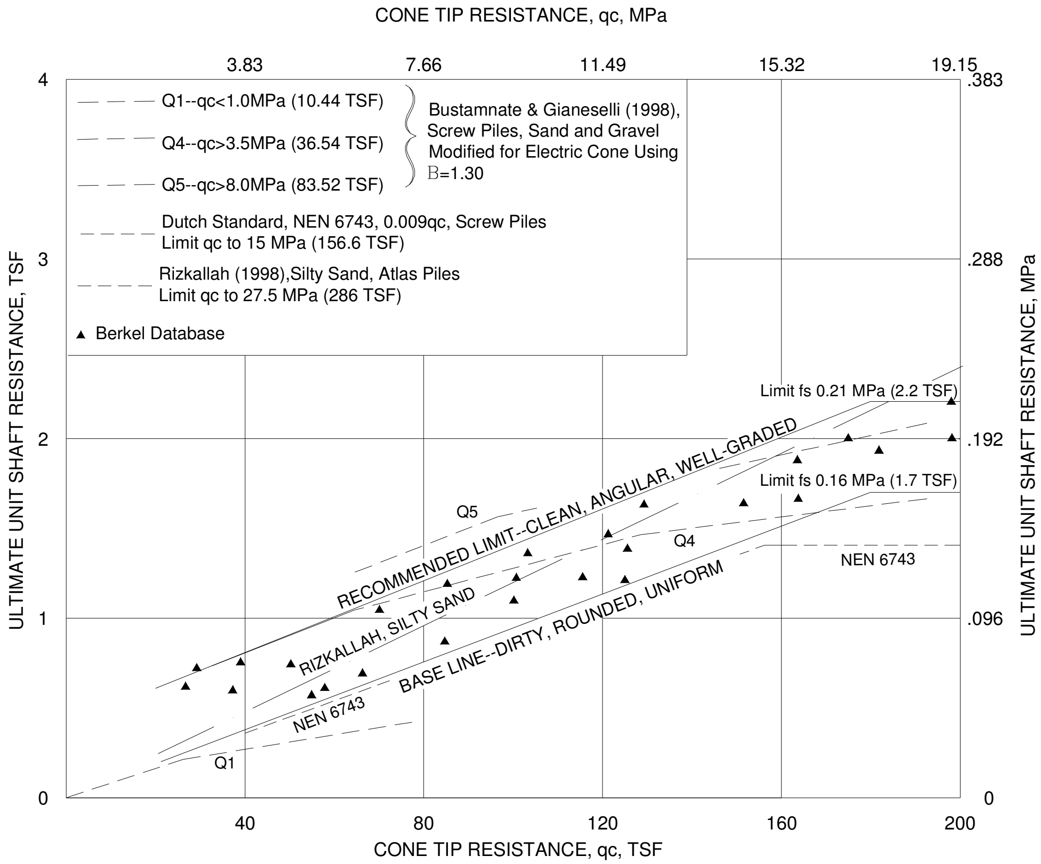

Figure 3. CPT tip resistance versus ultimate unit shaft resistance

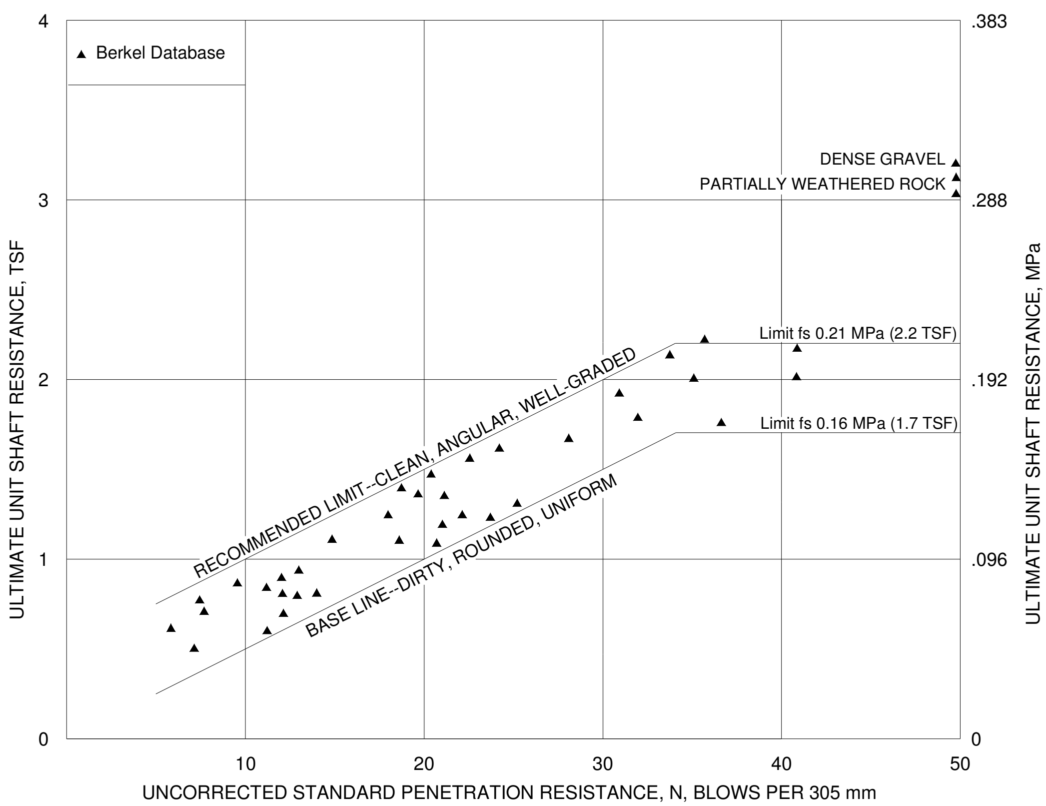

Figure 4. SPT N value versus ultimate unit shaft resistance

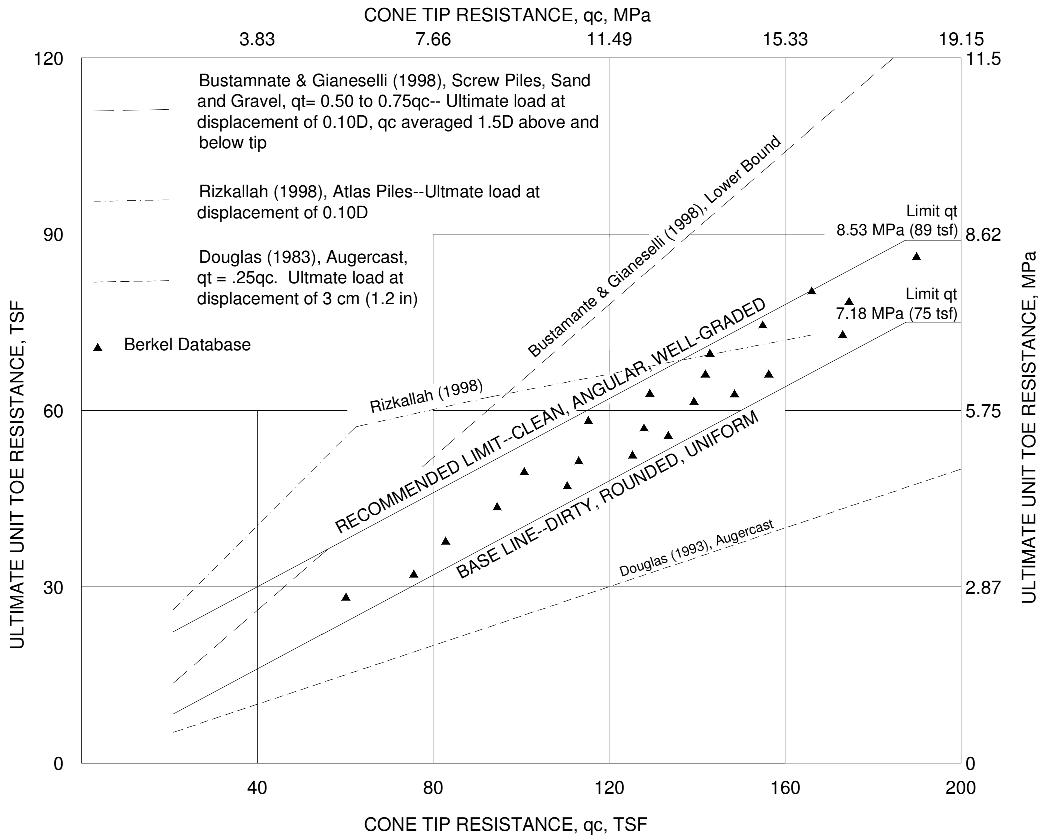

Figure 5. CPT tip resistance versus ultimate unit toe resistance

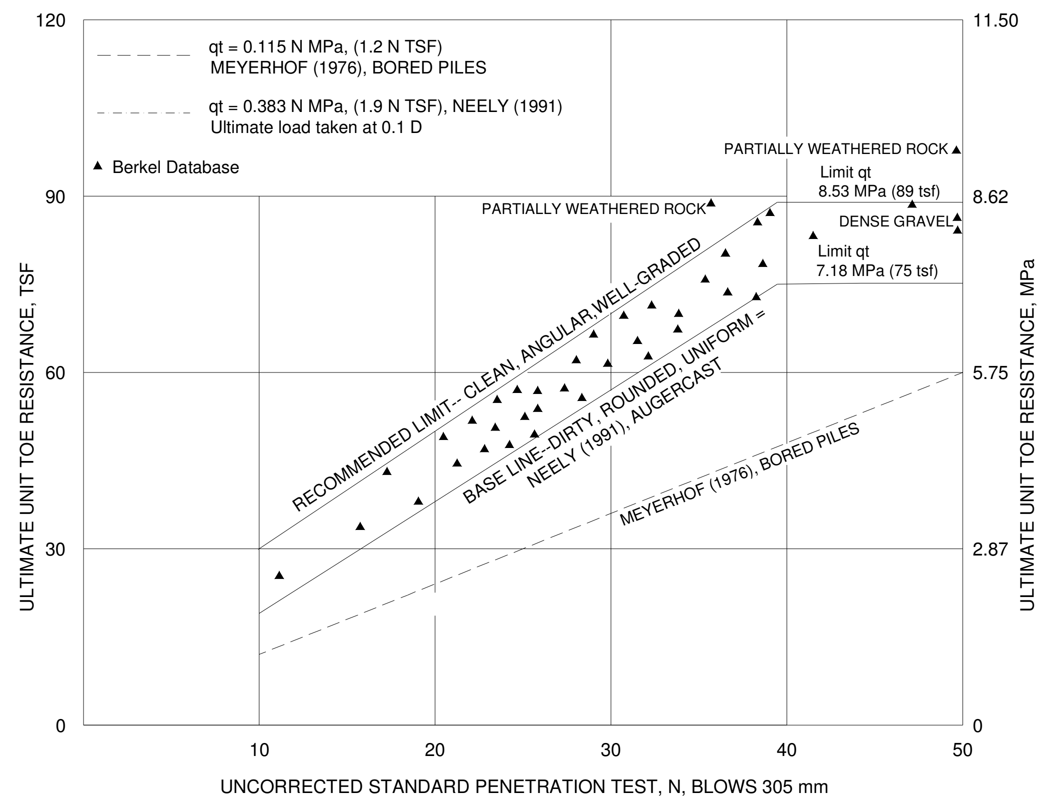

Figure 6. SPT tip resistance versus ultimate unit toe resistance

Conclusion

A database of 40 load tests on drilled, 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 resistances of the system are similar to those reported for other similar systems. The toe component of the system is higher than those that would be calculated for conventional augercast design methodologies, but well below those for displacement systems that use a sacrificial point. 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 is being evaluated on an on-going basis.

REFERENCES

BUSTAMANTE, M., and GIANESELLI, L. (1998), Installation parameters and capacity of screwed piles. Proceedings of the 3rd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles. A.A. Balkema, Rotterdam, pp. 95-108.

CHIN, F.K., (1970), Estimation of unit load on piles not carried to failure, Proceedings of the 2nd Southeast Asia Conference on Soil Engineering, pp. 81-90.

DOUGLAS, D.J., (1983), Discussion on Papers 17-22: Case Histories, Proceedings of the Conference on Piling and Ground Treatment, Institution of Civil Engineers, London, p. 283.

DUTCH STANDARD, (1993) NEN 6743

NEELY, W. J., (1991), Bearing capacity of auger-cast piles in sand. Journal of Geotechnical Engineering Division, ASCE, 117, No. GT2, pp 331-345.

MEYERHOF, G.G. (1976), Bearing capacity and settlement of pile foundations", Journal Of Geotechnical Engineering, ASCE, 102(3), 197-228.

RIZKALLAH, V. and BURNS T. (1998), Estimation of pile bearing capacity of cast-in-place screwed piles. Proceedings of the 3rd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles. A.A. Balkema, Rotterdam, pp. 417-421.