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Giuliana A. Zelada

Richard Wesley Stephenson
Missouri University of Science and Technology, rwstephenson@mst.edu

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Design Methods for Auger CIP Piles in Compression

Giuliana A. Zelada [1], and Richard W. Stephenson[2], Fellow, ASCE

ABSTRACT

This paper reports on a study of 43 compression load tests (five fully instrumented) and ten pullout tests of auger cast-in-place (ACIP) piles in sand. Eight design methods reported in the literature were used to predict the measured pile capacities. Ultimate load capacities of the ACIP load tests were determined using two different definitions of ultimate capacity and compared with predictions. Load transfer behavior was determined from a study of the fully instrumented pile tests and compared to predicted values. Recommendations are made regarding the design approach most appropriate for these foundations and the best procedure for estimating ultimate capacity from pile load tests.

INTRODUCTION

The total ultimate vertical load ($Q_{v,\,ult}$) bearing capacity of ACIP piles in cohesionless soils, as with other types of piles, depends on the relative density of the surrounding soil and the pile geometry, and is expressed by the following equation:

$$Q_{v,ult} = q_p \cdot A_b + f_s \cdot A_s \tag{1}$$

where, q_p = ultimate unit end bearing capacity, A_b = cross-sectional area of the pile base, f_s = ultimate unit friction capacity, and A_s = perimeter area of the pile.

Due to the constant soil support during ACIP pile installation, provided first by the augers during drilling and then by the grout during auger withdrawal, the behavior of ACIP piles falls somewhere between that of drilled shafts and driven piles. The design of ACIP piles has used both approaches, although the drilled shaft method has usually been the most favored by the design engineers. Several empirical design

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1 Geotechnical Engineer, Black and Veatch, Corporation, 8400 Ward Parkway, P.O. Box 8405, Kansas City, Missouri, 64114

2 Professor, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri 65409

methods for axial load capacity of ACIP piles have been proposed in the past several years (e.g. Neely, 1991, and Douglas, 1993), based on the results of ACIP pile load test databases correlated to field tests such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). However, these methods often underestimate the ultimate load carrying capacity of ACIP piles. At present, there is still no standard design method for ACIP piles that predicts the available pile capacity appropriately.

SCOPE OF WORK

This study compared computed theoretical load capacity, using eight different published techniques, with the results from a field load test database of ACIP piles constructed in cohesionless soils. Recommendations are made regarding the design approach most appropriate for these foundations and the best procedure for estimating ultimate capacity from pile load tests.

DESIGN METHODOLOGIES

The design methods that were evaluated in this investigation are described below. All methods using CPT correlations are based on the q_c value (i.e. at the cone tip). Based on a comparison of uplift load test results and strain gauge data from compression load tests performed at the same site, uplift resistance was taken as 2/3 of the friction capacity (Prakash and Sharma, 1989).

Neely (1991)-Neely proposed the following equations for the calculation of the ultimate capacity of ACIP piles, based on the results of the evaluation of an extensive load test database of ACIP piles in sands.

$$f_s = \beta \cdot p'_o \leq 1.4 tsf \ \ (135 \text{ kPa}) \tag{2}$$

$$q_p = 1.9N \leq 75 tsf \ \ (7.2 \text{ MPa}) \tag{3}$$

where p'_o is the average effective stress at the midpoint of the pile length, β is a friction factor correlated to pile length, and N is the SPT value at the tip of the pile. The unit capacities were limited to the recorded maximum database values. Neely defined the failure load as that load corresponding to a pile settlement equal to 10 percent of the pile diameter (10% PD).

Bustamante & Gianeselli (LPC) (1981)-This design procedure is based upon a load test database and CPT values. They developed a relationship for unit skin friction of ACIP piles as a function of cone resistance. The unit base resistance is taken as 15 percent of the CPT value at pile tip level.

Viggiani (1993)-This method is based on load testing performed at three different sites with generally cohesionless soils of volcanic origin. Failure was defined as that load corresponding to a pile butt settlement equal to 25 percent of the pile diameter. The unit base resistance is taken equal to the average CPT value within a specified distance

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from the pile tip. Unit skin friction is a function of the average CPT value along the pile shaft multiplied by a dimensionless factor α.

Rizkallah (German Standard, 1988)-In this method, the unit skin friction is calculated for each soil layer as 0.8 percent of the average CPT value for that layer. The ultimate unit end bearing is calculated for a settlement equal to 5 percent of the pile diameter (5% PD), and is a function of the CPT value at the pile tip. Full frictional resistance is assumed to mobilize at settlements not exceeding 10-20 mm (0.5 to 0.75 in.).

Wright and Reese (1978) (McVay et al., 1994)-This method was derived for the design of large diameter drilled shafts. The average skin friction of the pile in this method is given by:

$$f_s = p'_0 K_s \tan\phi \leq 1.6tsf \text{ (150 kPa)} \tag{4}$$

where K_s is taken as 1.1, p'_o is the average effective stress along the length of the pile and ϕ is the angle of internal friction of the soil.

The unit end bearing capacity is calculated using equation 5, where N is the SPT N-value at the pile tip. Ultimate pile capacity is assumed to be reached at a settlement equal to 5% PD.

$$q_{0.05d} = \frac{2}{3} \cdot N \leq 40tsf \text{ (3.8 MPa)} \tag{5}$$

Reese and O'Neill (FHWA, 1988)-A design procedure for drilled shafts was developed based on an extensive database for both cohesive and cohesionless soils. For sands, the mobilized skin friction at a given point in the pile is given by the previous equation 4. The K·tanϕ term is replaced by a β value which is a function of the depth, z, in feet, as shown below. This factor is not to be confused with Neely's β coefficient, which is a function of the pile length.

$$K \cdot \tan\phi = \beta = 1.5 - 0.135z^{0.5} (\text{z in feet}) \quad 0.25 \leq \beta \leq 1.2 \tag{6a}$$

$$K \cdot \tan\phi = \beta = 1.5 - 0.244\, z^{0.5} (\text{z in meters}) \quad 0.25 \leq \beta \leq 1.2 \tag{6b}$$

The end bearing is taken as 0.6N (tsf) at the tip, for N values less than 75. For N values greater than 75, the unit end bearing is assumed to be a constant 45 tsf (4 MPa).

Douglas (1983)-This method was also developed from the results of loading tests on ACIP piles in sand. In this method, failure was taken as the load producing a pile head movement of about 30-mm (1.2 inches). Shaft resistance was assumed to be fully mobilized at this movement. The unit skin friction was calculated similarly to Wright and Reese. The unit end bearing is taken as 25 percent of the CPT value at pile tip level.

Coyle and Castello (1981)-Data obtained from full-scale load tests were used to evaluate the bearing capacity factors for piles driven into cohesionless soils. These

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factors were correlated with the friction angles (ϕ) and relative density of the sands, as well as pile geometry. The design correlations developed for unit friction and unit end bearing capacities are given as a function of the slenderness ratio, L/d (pile depth over pile base diameter).

DATABASE

The load test database collected for this investigation consisted of a total of 53 pile tests, (43 compression load tests and 10 tension load tests). Five of these load tests were instrumented with strain gauges throughout the length of the piles. Load tests were conducted at 28 different locations throughout the U.S. and Europe.

Pile diameters ranged from 0.3 to 0.6 meters (12 to 24 inches) with the majority being 0.40 to 0.45 meters (16 to 18 inches) in diameter. The pile diameter was assumed equivalent to its nominal diameter. The pile lengths varied from 7.5 to 21 meters (25 to 70 feet), with most in the 9 to 12 meters (30 to 40 feet) range. All piles were constructed by pumping sand-cement grout down the hollow stem auger as it was withdrawn, except for those piles tested by Roscoe (1983), which were constructed using concrete.

The test loading generally followed ASTM D 1143-81 procedures, reaching a maximum load of about two to three times the design load. The piles were tested to failure in ten cases. Ten of the 43 compression load tests were performed using the ASTM Quick Test Method.

Subsurface Characterization-Soil borings were obtained for each test site. Average SPT N-values were calculated along the length of the test piles, at depth increments of 5 to 10 feet (1.5 to 3 meters). The relative density and unit weight of the sands were determined using correlations by Terzaghi and Peck (1948) and Bowles (1988). The angle of internal friction was estimated using published correlations (Peck, Hanson and Thornburn, 1953).

Soils in this study ranged generally from medium dense to dense, silty to fine, sands. No piles with more than 25 percent clay along their shafts were included in this study. In order to evaluate those design methods that use correlations with CPT data, Robertson and Campanella's correlation between SPT N-values and CPT values was used (Robertson and Campanella, 1983).

EVALUATION OF DESIGN METHODS

The performance of each of the design methods was assessed by determining the average ratio of $Q_{ult\text{-}measured}$ to $Q_{ult\text{-}predicted}$, its standard deviation, and coefficient of variation. Each method was evaluated for its accuracy in predicting total ultimate pile capacity, accuracy in predicting ultimate pile friction capacity, and accuracy in predicting ultimate pile end bearing capacity.

In addition, an evaluation was made of the apparent trends of unit pile capacities as a function of pile geometry and soil conditions. Finally, recommendations for improvements in ACIP pile capacity design methodology are presented.

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Ultimate Capacity Determination-Several definitions of failure load were investigated and compared to the results of ten load tests in the database that were carried to failure. The comparison showed that the most consistent method was that failure was the load applied at the pile butt when the normalized pile settlement S_o/D is equal to 10 percent (i.e. 10% PD).

The ultimate load as determined by a settlement of 5% PD was also used in the evaluation of design methodologies presented in this study. This allowed for the evaluation of design methods such as that of Wright and Reese, who developed their equation based on that settlement.

DATABASE EVALUATION RESULTS

The results for the $Q_{ult\text{-}measured}/Q_{ult\text{-}predicted}$ ratios for the 53 pile tests investigated are presented in Table 1. A total of 38 compression and 9 tensile pile tests were compared to the 5% PD criteria, while 28 compression and 5 tensile pile tests were compared to the 10% PD criteria. The difference in test numbers is due to the lack of load-settlement data or curves for all the piles. The 5% PD failure loads published in McVay's study (1994) were used for the pile tests taken from his database. In addition, a 90 percent confidence band was determined for each method including all tests compared to a 10% PD failure load (Figure 1). The circle denotes the average ratio computed for each method. The best methods are those that have narrow bands (more precision) and average measure to predicted load ratios close to 1.0.

Total Ultimate ACIP Pile Capacity- From Table 1 and Figure 1, Reese and O'Neill's drilled shaft approach gave a consistently overall higher correlation with measured ultimate ACIP axial pile capacities, no matter which of the ultimate failure criteria was used. In the case of ultimate load at 5% PD and considering all tests in the database, the average measured to predicted capacity ratio for this method was 1.08, with a moderate standard deviation of 0.30. In the case of ultimate load at 10% PD, these values remained almost the same with slight differences in standard deviation. This method gave good correlations with the measured values, whether the piles tested were in compression or tension.

Wright and Reese's design methods also gave good overall correlations, with average measured to predicted load ratios of 1.19 (5% PD) to 1.21 (10% PD), for all cases evaluated. However, standard deviations were consistently higher than those obtained by Reese and O'Neill, ranging from 0.26 to 0.57. Neely's method showed good correlation with the compression load tests. However, it underestimated the tensile capacity by more than 70 percent and the standard deviations computed for this

Table 1. Evaluation Of Ultimate ACIP Pile Capacity Design Methods

Q ult-measured / Q ult Predicted Ratios ± Standard Deviations (Coefficients of Variation)									
	Q-ult	Wright & Reese	Viggiani	Neely	Coyle & Castello	Reese & O'Neill	Doug.	LPC	Rizkallah
Comp Tests Only	5% PD	1.19 ± 0.33	0.54 ± 0.22	1.09 ± 0.41	1.00 ± 0.43	1.10 ± 0.26	1.59 ± 0.45	1.24 ± 0.40	1.45 ± 0.56
	10% PD	1.25 ± 0.47	0.64 ± 0.30	1.31 ± 0.54	1.25 ± 0.52	1.19 ± 0.38	1.72 ± 0.70	1.45 ± 0.55	1.72 ± 0.75
Tens Tests Only	5% PD	1.20 ± 0.57	0.74 ± 0.25	1.73 ± 0.74	1.54 ± 0.72	1.00 ± 0.42	1.93 ± 0.63	0.91 ± 0.29	1.12 ± 0.56
	10% PD	0.96 ± 0.26	0.86 ± 0.31	1.76 ± 1.12	1.94 ± 0.72	0.82 ± 0.25	1.66 ± 0.54	0.94 ± 0.36	1.49 ± 0.57
All Tests	5% PD	1.19 ± 0.38 (0.32)	0.58 ± 0.24 (0.41)	1.21± 0.54 (0.45)	1.10 ± 0.54 (0.42)	1.08 ± 0.30 (0.28)	1.65 ± 0.50 (0.30)	1.17± 0.40 (0.34)	1.39 ± 0.57 (0.41)
	10% PD	1.21± 0.45 (0.38)	0.67 ± 0.30 (0.45)	1.38 ± 0.65 (0.47)	1.36 ± 0.59 (0.44)	1.13 ± 0.38 (0.34)	1.71 ± 0.67 (0.39)	1.37 ± 0.55 (0.40)	1.69 ± 0.72 (0.43)

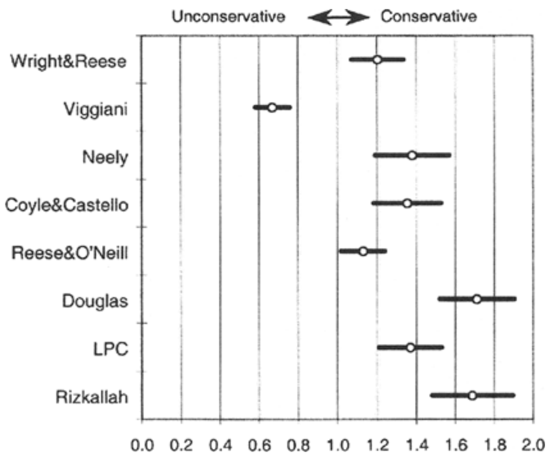

Figure 1 90% Confidence Band at 10% PD.

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case were high, from 0.74 to 1.12. The overall average measured to predicted load ratios for this method were 1.21 (5% PD) and 1.38 (10% PD) with corresponding standard deviations of 0.54 and 0.65.

Coyle and Castello's driven pile approach gave good average correlation with measured values at 5% PD, with an average measured to predicted load ratio of 1.00 when compared to compression tests only, and 1.10 including all tests. However, the standard deviation was above 0.50. Furthermore, the average ratio changed widely depending on the type of load test and failure criteria used. When compared to failure load at 10% PD, the average measured to predicted load ratios increased to 1.25 including compression tests only, and 1.36 including all tests. The method underestimated the tensile friction load by about 50-100%. This was expected since driven pile approaches give more weight to the end-bearing component than to the side friction component of the pile capacity. In contrast to the ratios calculated in the present study, McVay's study estimated that Coyle and Castello's driven pile approach overestimated the pile capacities by about 44%. LPC gave average ratios of 1.17 for the 5% PD and 1.37 for the 10% PD criteria, with moderate to high standard deviations (0.29 to 0.55). This method underestimated the compression capacity by about 20 percent while the tensile capacity was overestimated by about 10 percent.

Douglas and Rizkallah showed the greatest underestimation of ACIP pile capacity for all the methods investigated, with relatively high standard deviations. Douglas' side friction correlation puts a limit to the effective overburden pressure at a depth of 10 pile diameters, and this is most likely the reason for the underestimation in his values.

Viggiani is the only method that overestimates the total ultimate ACIP pile capacity. This is most probably because he assumes the unit end bearing capacity is equal to the CPT value at the tip without any adjustments. Overall, Viggiani tends to overestimate ACIP pile capacity by about 50 percent.

Table 2 shows evaluations of each method as a function of pile diameter. Only compression load tests were considered. Table 2 indicates that, in most cases, the accuracy of the method increases with increasing pile diameter. There was also a tendency to give a better correlation with higher length to diameter (L/D) ratios. Again, the drilled shaft approach of Reese and O'Neill is the most consistent regardless of the pile diameter.

Ultimate ACIP Pile Capacity Components-Chin's stability plot (1970) was used to separate the friction and end bearing components. As can be observed in Figure 2, the shaft resistance for the ACIP piles evaluated averages about 65% of the derived ultimate load, no matter what failure criterion is used. This is further reinforced with the strain gauge data available. Coyle and Castello (1981) had determined that 42% of the shaft capacity determined for driven piles in sands was the result of shaft friction.

As can be seen in Table 3, Neely, Coyle and Castello, Douglas and Rizkallah underestimated the friction capacity significantly. Reese & O'Neill gave the best correlation with an overall average measured to predicted load ratio of 0.92 and a standard deviation of 0.41 for the case of no limiting effective stress value. The driven

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Table 2. Evaluation of Design Methodologies by Pile Diameter.

		Q ult-measured / Q ult Predicted Ratios ± Standard Deviations							
Pile Dia. (in.)	Q-ult	Wright & Reese	Viggiani	Neely	Coyle & Castello	Reese & O'Neill	Douglas	LPC	Rizkallah
14	5% PD	1.30± 0.42	0.58 ± 0.21	1.21 ± 0.47	0.92 ± 0.38	1.15 ± 0.31	1.83 ± 0.50	1.20 ± 0.43	1.34 ± 0.62
	10% PD	1.46 ± 0.39	0.74 ± 0.23	1.36 ± 0.38	1.29 ± 0.34	1.29 ± 0.31	2.05 ± 0.47	1.64 ± 0.37	1.96± 0.51
16	5% PD	1.10 ± 0.32	0.58 ± 0.21	1.25 ± 0.60	1.31 ± 0.61	1.01 ± 0.29	1.48 ± 0.41	1.10 ± 0.34	1.38 ± 0.50
	10% PD	1.10 ± 0.32	0.62 ± 0.31	1.30 ± 0.74	1.36 ± 0.69	0.99 ± 0.28	1.50 ± 0.47	1.07 ± 0.39	1.43 ± 0.57
18	5% PD	1.08 ± 0.26	0.61 ± 0.34	1.27 ± 0.83	1.27 ± 0.62	1.02 ± 0.30	1.54 ± 0.63	1.41 ± 0.56	1.68 ± 0.71
	10% PD								
24	5% PD								
	10% PD	1.12 ± 0.32	0.41 ± 0.07	0.79 ± 0.19	0.73 ± 0.22	1.03 ± 0.30	1.34 ± 0.35	0.96 ± 0.17	1.08 ± 0.30

* A minimum of three pile tests per diameter category were required for this statistical analysis.

pile approach of Coyle and Castello gives the highest underestimation. Standard deviations were high in all cases.

When evaluating the end-bearing component, Table 4 shows that all methods but one significantly underestimated the tip capacity. Viggiani, who takes the full value of the CPT at the pile tip as the unit end bearing capacity overestimated the end bearing capacity by about 50%. This was expected considering the construction method of ACIP piles, which would definitely cause disturbance at the pile tip due to the augering and grouting process, making the unit end bearing value smaller than the undisturbed CPT at the tip. Neely's and Coyle and Castello's methods gave the average measured to predicted ratios for tip capacity closer to unity. However, the standard deviations ranged from 0.71 to 0.92. The remaining five methods greatly underestimated the end bearing capacity by more than a factor of 2 and the standard deviations were very high. Both, Neely's as well as Coyle and Castello's methods

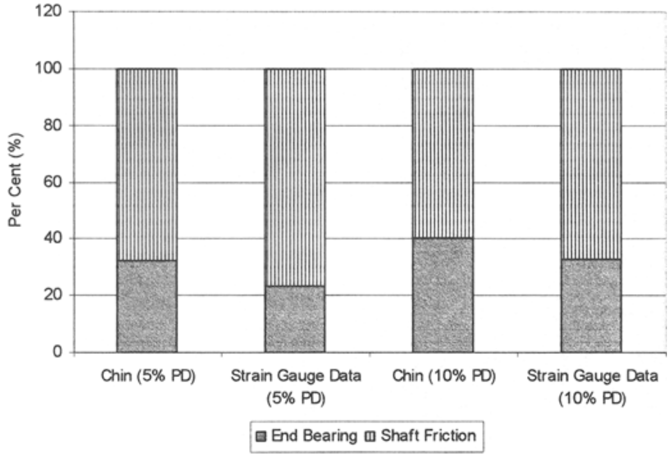

Figure 2: Measured Load Distribution

gave better correlations when compared to the end bearing loads at 10% PD ultimate load capacity definition.

Drilled shaft methods could be underestimating the end bearing capacity because they are accounting for a large reduction in end bearing capacity due to soil decompression while drilling. It would appear that during the installation of an ACIP pile such a drastic reduction in the end bearing capacity due to augering is prevented by the pressurized grout injection.

Table 3. Evaluation Of Ultimate ACIP Pile Friction Capacity.

Q ult-measured / Q ult-predicted Ratios ± Standard Deviations							
Wright & Reese	Viggiani	Neely	Coyle & Castello	Reese & O'Neill	Douglas	LPC	Rizkallah
1.09 ± 0.57	0.83 ± 0.42	1.50 ± 0.78	1.94 ± 1.20	0.92 ± 0.41	1.81 ± 0.75	1.05 ± 0.51	1.39 ± 0.78

The fact that Reese and O'Neill, LPC and Wright and Reese underestimated the end bearing capacity by such a high magnitude, yet gave very good correlations when compared to the measured total ultimate capacity, gives further indication of the importance of side friction resistance as a major component of an ACIP pile load capacity.

ULTIMATE UNIT CAPACITY ANALYSIS

Ultimate unit side friction and ultimate unit tip capacities were derived using average effective soil stresses along the piles together with the load distribution values obtained using the stability plot method for the case of ultimate pile capacity at a settlement of 10% PD. Strain gauge data was used when a correlation against soil

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Table 4. Evaluation of Ultimate ACIP Pile End Bearing Capacity.

	Q ult-measured / Q ult Predicted Ratios ± Standard Deviations							
Q-ult Method	Wright & Reese	Viggiani	Neely	Coyle & Castello	Reese & O'Neill	Douglas	LPC	Rizkallah
5% PD	2.35 ± 2.40	0.43 ± 0.44	0.87 ± 0.88	0.73 ± 0.71	2.47 ± 2.43	1.62 ± 1.59	2.70 ± 2.66	3.01 ± 2.84
10% PD	3.05 ± 2.51	0.56 ± 0.46	1.11 ± 0.92	0.98 ± 0.71	3.26 ± 2.57	2.14 ± 1.69	3.57 ± 2.81	4.01 ± 2.98

depth was required. Only those methods that gave the best correlations for each, side friction and end bearing capacity, were evaluated, i.e. Wright and Reese and Reese and O'Neill's method for side friction, and Neely and Coyle and Castello's method for end bearing capacity.

Ultimate Unit Skin Friction-Figure 3 shows the relationship of the derived unit skin friction capacities from the measured friction load capacity values and the average SPT N-values along the shaft.

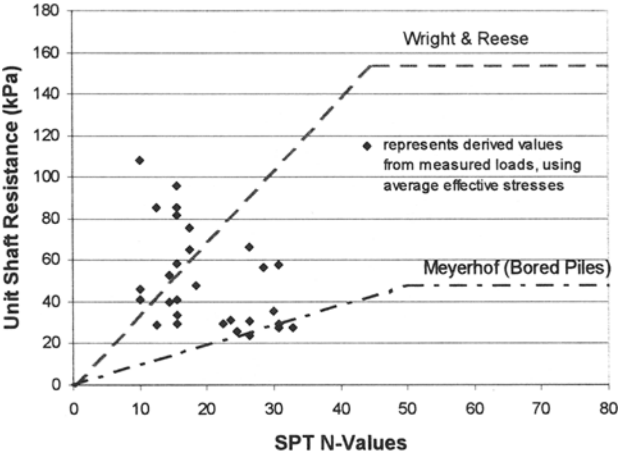

Figure 3. Unit Skin Friction Capacity vs. SPT N-Values.

The data in Figure 3 is compared to Wright and Reese's curve for ultimate unit skin friction, as well as to Meyerhof's curve for bored piles in sands (for reference). On average, the data seems to follow the trend of drilled shaft behavior established by Wright and Reese. However, the scatter in the data is quite high and when looking at the unit shaft resistances obtained for SPT N-values greater than 20 bpf, the data correlates better with Meyerhof's curve for bored piles.

The $\beta = K \cdot \tan(\phi)$ values derived from the measured side resistances obtained from those pile tests with strain gauge data are shown in Figure 4. Only two data

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points exceed the limiting value of 1.2 suggested by Reese and O'Neill, while two other data points were lower than the 0.25 limit expressed by their β equation. The highest derived β value is taken as unreliable and is not considered when evaluating the correlation of β with soil depth.

The correlation between the derived β factors with Reese and O'Neill's equation tends to overestimate the β factor, and thus the unit shaft capacity. Reese and O'Neill's β equation was adjusted by a factor less than one, in order to obtain a better correlation with the data. A final correction factor of 0.8 was determined and the resulting corrected Reese and O'Neill curve is shown by the dashed line in Figure 4. This corrected curve gave a better data fit with the strain gauge data, and also obtained a slightly better average measured to predicted friction load ratio: 0.98 with a standard deviation of ± 0.47.

Reese & O'Neill had also indicated that the β factor was not dependent on any measure of soil consistency, such as the SPT blow count, but was primarily controlled by the pattern of lateral effective stress between the soil and the drilled shaft. The lateral effective stress is, in turn, affected by the installation procedure.

Ultimate Unit End Bearing Capacity-Figure 5 is a plot of derived ultimate unit pile capacity versus SPT N-values for 10% PD ultimate pile capacity. It can be seen that again, the ACIP pile is behaving within the limits set by driven and drilled shaft methods. The average correlation equation is:

$$q_b = 1.7 \cdot N \text{ (in tsf)} \quad \text{or} \quad q_b = 0.163 \cdot N \text{ (in MPa)} \qquad (7)$$

This is somewhat lower than Neely's prediction curve ($q_b = 1.9 \cdot N$). Four of the piles evaluated showed unit tip resistances greater than the limit of 75 tsf (7.1 MPa) set by Neely in his study. This 7.1 MPa (75 tsf) limit corresponded to the highest value that Neely obtained while evaluating his load test database. It can be seen that two of these four piles obtained unit tip capacities greater than 75 tsf at relatively low SPT N-values (less than 15 bpf) and therefore it would appear that soil conditions for these piles may be different than those encountered during the site exploration.

Equation 7 gave an average measured to predicted tip load ratio of 1.08 with a standard deviation of ± 0.73 when compared to the derived end bearing loads at 10% PD. This is a relative better correlation than the 1.11 ± 0.92 average ratio obtained previously for Neely's method (Table 4).

Coyle and Castello correlated unit tip capacity to the angle of friction of the soil and to the slenderness ratio (length/diameter) of the pile. The trendline plotted on Figure 6 shows that unit tip resistance does tend to increase with increasing length to diameter ratios, with only one value exceeding the 100 tsf (9.5 MPa) limit set by Coyle and Castello. However, no trend or limiting value could be found for any specific angle of friction. Since the unit end bearing capacity is more a function of the soil properties than of pile geometry, no best fit line was determined for these set of data.

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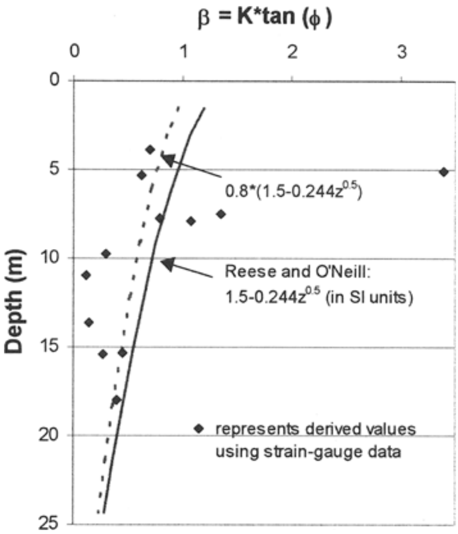

Figure 4. β = K·tan(φ) vs. Depth

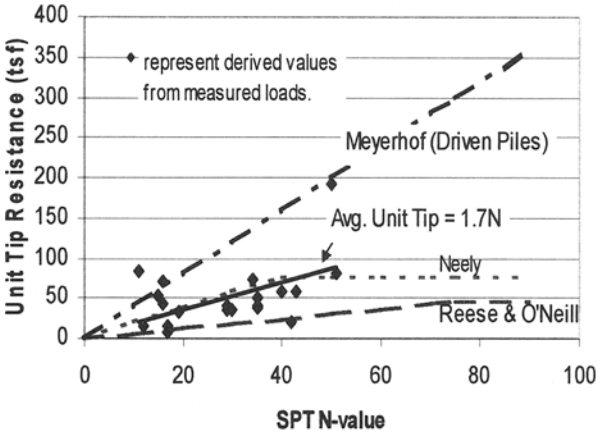

Figure 5. Unit Point Capacity vs. SPT N-values (10% PD).

In summary, it would appear that ultimate ACIP pile unit capacity can be better estimated by using the following correlations:

$$q_b = 1.7 \cdot N \leq 75\ tsf \tag{8a}$$

$$q_b = 0.163 \cdot N \leq 7.2\ \text{MPa} \tag{8b}$$

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$$f_s = \beta \cdot p'_o \tag{9}$$

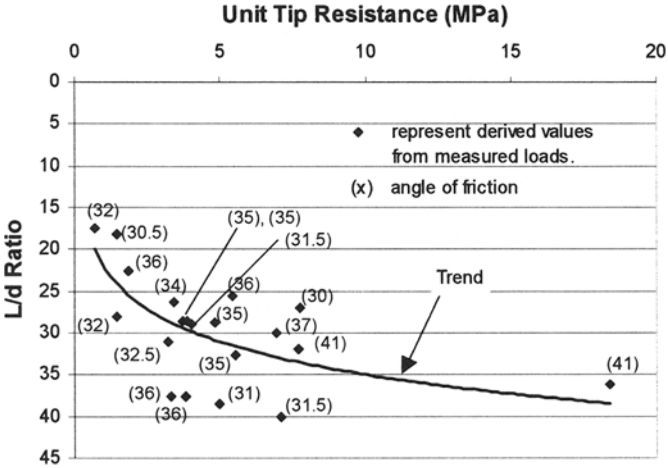

Figure 6. Unit point Capacity vs. Length to diameter Ratio.

The β factor is now represented by the following equation:

$$\beta = 1.2 - 0.108z^{0.5} \text{ (z in feet)} \qquad \text{where } 0.2 \le \beta \le 0.96 \tag{10a}$$

$$\beta = 1.2 - 0.0195z^{0.5} \text{ (z in meters)} \tag{10b}$$

The correction applied to Reese and O'Neill's friction correlation can also be explained by studying Tables 1, 3, and 4. Reese and O'Neill gave the best correlation for total ultimate ACIP pile capacity, with an average measured to predicted load ratio of 1.13 for the case of ultimate load capacity at 10% PD. However, inspection of Tables 3 and 4 reveals that the actual load distribution between the friction and end bearing components determined by Reese and O'Neill is inadequate. The end bearing component is underestimated by a factor of 3, while the friction component is slightly overestimated, with a measured to predicted friction load ratio of 0.92. If the end-bearing component is to be increased, the friction correlation would have to be adjusted down. This is accomplished by correcting Reese and O'Neill's correlation by a factor of 0.8.

In his study, Neely had determined that the friction component of the total ultimate ACIP pile capacity was equal to about 55% of the total load. In this study, 65% of the total ultimate ACIP pile load was found to be carried in side friction. In order to increase the friction component of Neely's method, the end bearing load component would have to decrease. This is precisely what was done when correcting his end bearing correlation to be 1.7·N instead of 1.9·N.

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Figure 7 shows the results obtained when comparing the measured and predicted total ultimate ACIP pile load capacities at 10% PD, using the corrected correlations. An average measured to predicted ratio of 1.08 was obtained, with a standard deviation of ± 0.40. The use of this combination method is enhanced when considering that each component of the ACIP pile capacity is being estimated in the best way possible, leading to greater accuracy by appropriate calculation of the corresponding friction and end bearing capacities.

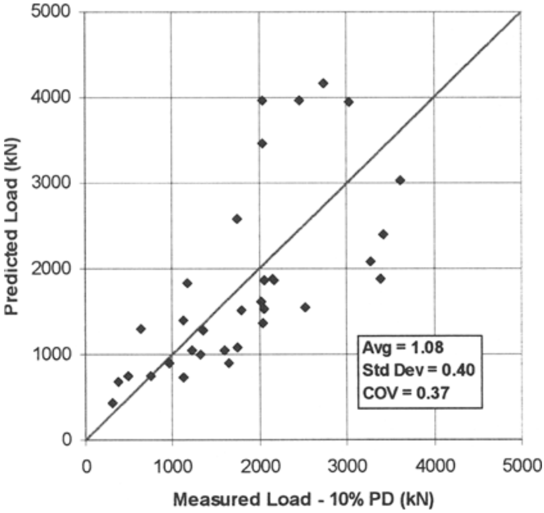

Figure 7. Measured to Predicted Load Ratios at Qult = 10% PD – New Correlation.

CONCLUSIONS.

- ACIP pile behavior in cohesionless soils is similar to that of drilled shafts. About 65 percent or more of their ultimate pile capacity is carried in side friction.
- Reese and O'Neill's drilled shaft approach was found to give the best correlation for the computation of the ultimate side friction resistance.
- Coyle and Castello's driven pile design method, and the empirical ACIP pile design method developed by Neely gave the best correlations for the computation of the ultimate end bearing capacity for ACIP piles.
- Applying a correction of 0.8 to Reese and O'Neill's β factor equation and lowering Neely's end bearing capacity correlation from $q_b = 1.9 \cdot N$ to $q_b = 1.7 \cdot N$, appears to be the best way to estimate ACIP pile capacity. The new unit capacity correlations are shown in the equations below:

$$q_b = 1.7 \cdot N \le 75\ tsf \qquad (11a)$$

$$q_b = 0.163 \cdot N \le 7.2\ \text{MPa} \qquad (11b)$$

$$f_s = \beta \cdot p'_o \qquad (12)$$

where,

$$\beta = 1.2 - 0.108z^{0.5} \qquad \text{where } 0.2 \le \beta \le 0.96 \qquad (13)$$

$$\beta = 1.2 - 0.0195z^{0.5} \text{ (z in meters)} \qquad (13b)$$

An average measured to predicted total ultimate load ratio of 1.08, with an average standard deviation of 0.40 is obtained with this procedure. The lowest measured to predicted total ultimate load ratio obtained for this database using this new correlation and the 10% PD failure criteria was 0.48.

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