Design and Construction of Auger-Cast Piles in Florida

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The use of augered cast-in-place piles has seen a tremendous growth in Florida because of the price and ease of installing them in coastal shell-filled sands. Discussed is the construction of augered cast-in-place piles, including equipment selection, drilling rate, grout fluidity, grout's aggregate size, grout pumping, and auger removal. Also presented is a comparison between a data base of 21 pile load tests (17 compression and 4 tension) from Florida and five design methods. Three of the methods were developed for augered cast-in-place piles, the other two for driven piles and drilled shafts. The predicted capacities of these methods were compared with three types of settlements of the piles' diameters. All of the methods compared most favorably to the 5 percent criterion. The drilled-shaft approach gave the best prediction for the whole data base, with a mean of 1.08 and a standard deviation of 0.28 for the ratios of predicted to measured capacity and, in the case of compression loadings only (17 piles), a mean of 0.98 and a standard deviation of 0.16. The latter finding suggests that augered case-in-place piles behave more like drilled shafts than driven piles because of the installation method.

The use of augered cast-in-place piles under 3- to 6-story structures has grown tremendously in Florida during the past 20 years. Problems with densification (vibration) and heave associated with driven piles in loose to dense sands do not occur with properly installed augered piles. Augered cast-in-place piles have been constructed with diameters of .31 m (12 in.) and lengths up to 6.1 m (20 ft) since the 1950s. However, with the advent of better drilling equipment, diameters varying from .41 m (16 ft) to .51 m (20 in.) and maximum depths ranging from 18.3 m (60 ft) to 24.4 m (80 ft) are achievable. Reinforcement may vary froma single high-strength rebar (Grade 60) at the top of the pile to a continuous steel cage, depending on loading (compression, tension-uplift, and lateral). Typical axial design loads (compression) for a single augered cast-in-place pile range from 445 kN (50 tons) to 890 kN (100 tons).

As with most drilled shafts, the quality of augered cast-in-place piles is strongly affected by their construction: equipment selection, drilling rate, grout fluidity, grout's aggregate size, grout pumping, and auger-removal process all significantly affect the quality of the pile and its load carrying capacity. In Florida it is common practice for an architect or engineer to ask prospective pile contractors for evidence that they have sucessfully installed augered cast-in-place piles under similar job and subsurface conditions. If there are questions regarding the quality or load-test results of installed piles, dynamic testing of pile integrity is usually performed as well.

Design of augered cast-in-place piles varies; the pile is either considered a drilled shaft or a large displacement-driven pile. Both effective- and total-stress methods are often used. Five of the most common design methods are presented and compared to a data base of 21 augered cast-in-place piles in Florida soils.

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CONSTRUCTION

An augered pile's capacity is strongly influenced by its construction. Augered cast-in-place piles are constructed using an electrically or hydraulically powered, continuous hollow-stem auger mounted on either a steel lattice or on pipe leads. The power supply and the auger both play a significant role in a successful pile installation. The power supply should be rated at or above 27 kN-m (20,000 ft-lb), and the auger should have pitch equal to one-half its diameter, for drilling in either cohesionless or cohesive soils. In cohesionless soils, the use of lower power torques and greater flight pitches may result in "weak drilling" (*1*). In this practice, which is not evident to the average client (*2*), the vertical speed of the auger, v, is less than the pitch of auger's flight, p, multiplied by the rotational speed of the auger, w (revolutions per minute). Since the auger's vertical flight speed is greater than the rate of auger penetration, soil is transported to the ground surface, loosening the soil adjacent to the auger and possibly resulting in the auger partially filling with soil. The diminished in situ stresses (soil loosenings) will result in a diminished pile capacity; the partially filled auger will cause the grout to flow up and down the auger, possibly contaminating the pile. The practice of "weak drilling" allows the contractor to employ less powerful, less expensive equipment; penetrate deeper depths; and have high production rates at the expense of pile capacity and quality control. Since cohesive soils are more difficult to drill because the soil adheres to the auger, use of higher power torques and lower flight pitch [such as 27 kN-m (20,000 ft-lb) and pitch equal to one-half the auger's diameter] will aid successful installation.

After reaching the required depth, the auger is usually raised approximately .61 m (2 ft) and grout is pumped in. The auger is then lowered to its original depth to establish a positive head of grout. Finally, the auger is raised while continuously pumping group out of the bottom or side of the hollow stem auger. Care must be exercised to maintain the grout head approximately 1.5 m (5 ft) to 3 m (10 ft) above the tip of the auger, to ensure that soil does not mix with the grout and the pile diameter does not neck inward. To maintain the positive grout head, 10 to 15 percent more than the theoretical volume is pumped in for each 1.5 m (5 ft) interval. The grout take of a pile segment is much more important than the average for the whole pile. Typical grout factor ratios of pumped to theoretical volumes are 1.4 to 1.5 for piles .35 m (14 in.) to .41 m (16 in.) in diameter constructed in sand (*1,3*). In the case of South Florida's cemented sands or Miami's oolites, which are vuggy (solution channels), a pressure gauge mounted near the auger on the grout feed is monitored closely for pressure loss. The gauge indicates the loss of grout head at the auger tip but not the grout pressure in the pile (*2*). If the grout head is lost at any stage of the auger withdrawal, then the auger should be lowered 5 to 10 ft (1.5 to 3 m) into the grout and withdrawal reinitiated. When grouting has been completed as far as the ground surface, a single rebar or cage is placed while the grout is fluid. The reinforcement should be installed so it can move to the final depth of the pile without obstruction. In the event the steel is refused, the pile should be redrilled and re-grouted. The free advancement of the steel to the pile tip is one of the best indicators that inclusions have not occurred. The practice of dipping or scooping grout out of the top of the pile while the grout is still fluid, for example, when these are pile cutoff elevations below the ground surface, is not recommended. The practice has been known to contaminate the top portion of the pile with soil and cause pile failure. Piles should be cast to the drilling grade, allowed to set (harden or hydrate), and then cut off.

The grout used in augered cast-in-place piles must be of low enough viscosity to be pumped and of high enough viscosity to displace fines as the auger tip is extracted. The proportions by weight of cement, water, fine aggregate, and fly ash in a typical grout mix are 1: 0.59: 2.5: 0.15. This mix is very similar to ASTM C-109, which is used for mortar cube testing (but without fly ash). Fly ash is used in the grout mix for two reasons: it increases the fluidity of the grout, and it results in a hydrated grout that is less permeable. One disadvantage is that strength gain with time is slower with this grout mix than with a mix wherein an equivalent amount of cement is used in lieu of fly ash. Whereas the grain-size characteristic of the fine aggregate is considered important to preventing segregation problems by some researchers (*1*) it is not deemed important by others (*3*). Sands with a fineness modulus of about 1.2 are recommended. Plasticizers are added to increase fluidity, other additives to control shrinkage. The optimal grout viscosity for pumping and displacement of fines corresponds to flow rates of approximately 15 to 25 sec through an ASTM C939-81 cone fitted with an outlet 19 mm (¾ in.) in diameter. Typical compressive strengths are 27.6 MPa (4000 psi) after 28 days on 51 mm (2 in.) cubes. Samples usually are tested after 7 and 14 days as well.

DESIGN

During the past 15 years a number of different methods have been proposed to estimate the capacities of augered cast-in-place piles. The methods vary; some consider augered piles driven piles, others view them as drilled shafts. Lately, a number of design methods specific to augered piles have been proposed (*1*). What follows is a comparison of three commonly used methods as well as a drilled-shaft and a driven-pile approach for 21 Florida sites whose load tests were performed. Seventeen of the cases were compression loadings and four were tension loadings. Since all the methods are empirical, the predicted capacities were compared to failure as defined by Davisson (*4*), 2 percent, and 5 percent pile-diameter settlements. A brief discussion of each method is given first, followed by a summary of the data base.

Wright and Reese (1979)

In 1979 Wright and Reese (*5*) published a design method for constructing bored piles and augered cast-in-place piles in sand. The average mobilized skin friction stress on a pile is given by

$$fs = Po'\ Ks \tan \phi \qquad \leq 0.15 \text{ MPa (1.6 tsf)} \tag{1}$$

where

Po' = average effective stress along the pile,
Ks = lateral earth pressure coefficient (taken as 1.1), and
ϕ = angle of internal friction of the sand.

The ultimate tip stress for the pile is given by

$$q_{ip} = 2\ N/3 \qquad \leq 3.8 \text{ MPa (40 tsf)} \tag{2}$$

where N is the standard penetration test (SPT) value at the pile tip. The skin and tip stresses are limited to 1.6 tsf and 3.8 MPa (40 tsf), respectively.

Neely (1991)

Neely (*1*), using a data base of augered cast-in-place piles founded in sand, established the following relationship for the average skin friction stress along a pile:

$$fs = \beta\ PO' \qquad \leq .13 \text{ MPa (1.4 tsf)} \tag{3}$$

where Po' is the average vertical effective stress along the pile and β is an empirical parameter. The β factor was found to be independent of the soil's relative density but a function of the pile's length, as given in Figure 1. Evident from the figure, β has a maximum value of 2.5 and a minimum value of 0.2, depending on total pile length. Using data from both compression and tension testing, Neely (*1*) estimates the ultimate pile tip stress at:

$$q_p = 1.9\ N \qquad \leq 7.2 \text{ MPa (75 tsf)} \tag{4}$$

where N is the SPT value at the pile tip. The maximum skin friction and tip resistance are limited to 0.13 MPa (1.4 tsf) and 7.2 MPa (75 tsf), respectively. Both fs and q_p were limited by Neely to the recorded maximum data-base values.

Laboratorie Des Ponts et Chausses (LPC)

Bustamante and Gianeselli (*6*) in France have developed a design procedure for various pile types, including H, driven, and bored

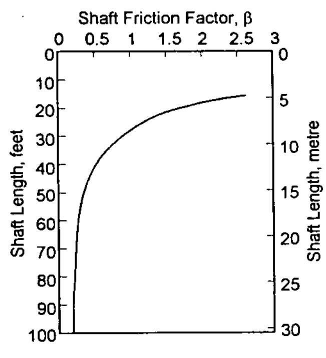

FIGURE 1 Mobilized skin friction coefficient, Neely (1991).

from a data base for use in both cohesive and cohesionless soils. The in situ cone-point resistance, *qc*, is used to calculate both the maximum side friction, *fs*, and the mobilized point resistance, q_p. For an augered cast-in-place pile, Figure 2(a) or 2(b) is used, depending upon soil type, to obtain the average skin friction stress, *fs*, for a particular soil layer. Each figure has two curves (upper and lower bounds), and *fs* is determined by interpolation between the two curves based on the average *qc* for the layer. Since only in situ SPT data were available for the data base evaluated in this paper, the following correlation was used between *qc* and the *N* values for Florida sands (*7*):

$$qc = 3.5 \cdot N \qquad (5)$$

and, for clay,

$$qc = Su\ Nc + Po \qquad (6)$$

where

Su = soil's undrained strength;
Nc = bearing capacity factor, usually taken as 17 (*8*) and
Po = total stress at the layer's center

The ultimate end bearing, q_p, of an augered cast-in-place pile founded in sand by the LPC approach is

$$q_p = 0.15\ qc \qquad (7)$$

In the case of clays, LPC recommends an end bearing, q_p, of

$$q_p = 0.375\ qc \qquad (8)$$

Reese and O'Neill (1988)

Under the sponsorship of FHWA, Reese and O'Neill (*9*) developed a design procedure for drilled shafts on the basis of an extensive data base for both cohesive and cohesionless soils. In the case of sands, the mobilized skin friction at a given point on the pile is given by

$$fs = K\ Po' \tan \phi \qquad (9)$$

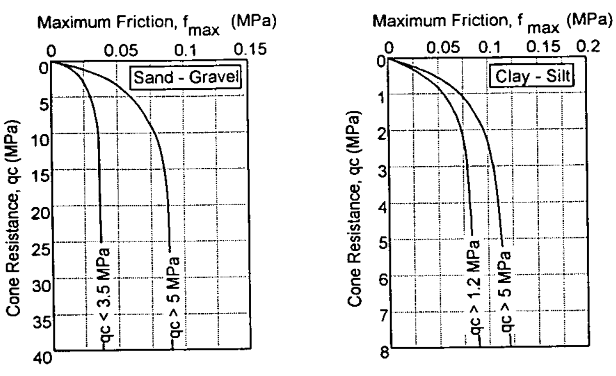

FIGURE 2 LPC's skin friction on pile from cone *qc* data: (*a*) *f* (versus) *qc* for sands; (*b*) *f* versus *qc* for clays.

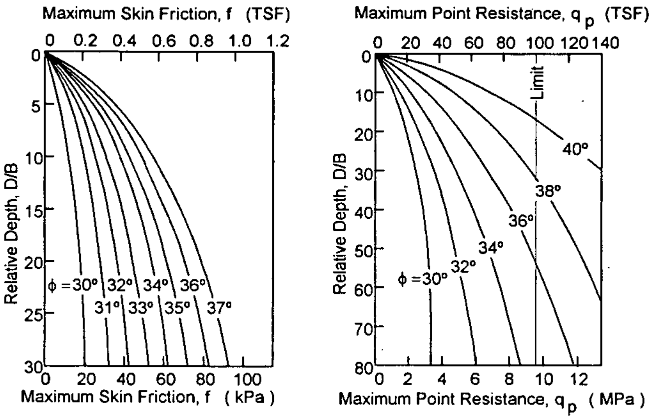

FIGURE 3 Coyle and Castello's pile capacity versus friction angle and embedment: (*a*) skin friction versus embedment; (*b*) end bearing versus embedment.

where, at the depth *z*,

Po' = effective stress,
K = earth pressure coefficient, and
ϕ = angle of internal friction of the soil.

K tan ϕ is replaced by β, given as

$$\beta = K \tan \phi = 1.5 - 0.135(Z)^{0.5} \qquad 0.25 \le \beta \le 1.2 \qquad (10)$$

where *Z* is the depth in feet. Equation 10 must be substituted into Equation 9 and integrated over the entire depth of the pile to determine the mobilized skin friction on the pile.

The end bearing, q_p, is based on the SPT *N* value at the drilled shaft's tip, according to the following:

$$q_p = 0.6\ N \qquad 0 \le N \le 75, \text{ or} \qquad (11)$$

$$q_p = 4.3 \text{ MPa } (45 \text{ tsf}) \qquad N > 75 \qquad (12)$$

In the case of cohesive soils, the average mobilized skin friction stress, *fs*, on the pile is determined from

$$fs = 0.55\ Su \qquad \le 0.26 \text{ MPa } (2.75 \text{ tsf}) \qquad (13)$$

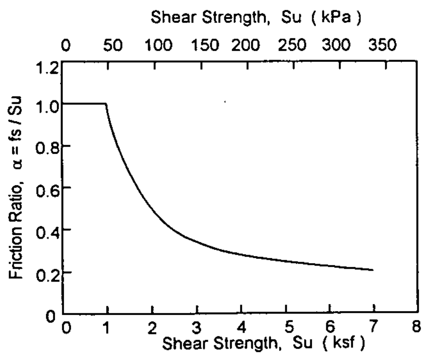

FIGURE 4 α versus undrained strength.

where Su is the average undrained strength along the pile. The end bearing resistance, q_p is determined as

$$q_p = Nc\ Su \qquad \leq 3.8 \text{ MPa (40 tsf)} \tag{14}$$

where Nc is the bearing capacity factor and Su is the soil's undrained shear strength in the vicinity of the pile tip. A value of 9 is recommended for Nc (*9*).

Coyle and Castello (1981)

The only driven-pile approach to be presented is the one Coyle and Castello (*10*) developed to estimate pile capacities in sand based on a data base. To determine the average skin friction along the pile, Figure 3(a) is used with the angle of internal friction, ϕ, of the sand and the ratio of the pile's embedded depth, D, to its width, B. Coyle and Castello recommend that the angle of friction, ϕ, be obtained by correlation to SPT N on the basis of work by Peck et al. (*11*), if laboratory strength-data are unavailable. In the case of silty sands below the water table, Coyle and Castello recommend that the SPT N values first be corrected with the following expression:

$$N' = 15 + 0.5\ (N - 15) \tag{15}$$

TABLE 1 Boring Logs

Pile No.	1	2	3	4	5	6	7	8	9	10	11
Location	Plm B.	T. Verd.	Ver. B.	Tr. Isl.	St. Pete.	W. Hav.	Ruskin	St. Pete.	St. Pete.	Tr. Isl.	St. Pete.
Dia. (mm)	.36	.36	.36	.36	.36	.36	.30	.36	.36	.36	.36
Length (m)	9.1	10.7	12.2	9.1	9.1	7.6	9.1	15.2	9.1	13.7	12.2
Depth (m)	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N
1.5	14	18	5	38	16	2	19	30	14	38	30
3.0	5	15	8	22	46	16	15	65	48	22	65
4.6	40	21	10	17	18	20	28	30	45	17	30
6.1	2	30	17	43	2	25	35	14	40	42	14
7.6	40	15	80	21	64	29	25	15	31	21	15
9.1	32	25	33	17	31	25	28	9	29	17	9
10.7		20	15	4	71	34	42	11	32	4	18
12.2			19	3	50	37		23	25	24	23
13.7			45	24				11	2	3	11
15.2			60	3				30	10	11	80
16.8			34	11				60		18	30
18.3			23	18				18			18
19.8								36			36

Pile No.	12	13	14	15	16	17	18	19	20	21
Location	Tampa	Jacks.	Savana	St. Aug.	Palatka	Cocoa	Poinc.	Palm B.	Tallah.	Tallah..
Dia. (mm)	.36	.36	.41	.46	.41	.36	.36	.36	.36	.36
Length (m)	12.2	7.6	13.7	10.7	12.2	9.1	10.7	10.7	21.3	25.9
Depth (m)	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N	SPT-N
1.5	4	7	4	6	11	7	4	31	N.A.[a]	N.A.
3.0	1	9	5	4	12	9	21	39	N.A.	N.A.
4.6	1	8	18	34	17	12	17	40	N.A.	N.A.
6.1	5	35	26	17	14	34	15	28	N.A.	N.A.
7.6	28	26	42	21	49	36	11	17	N.A.	N.A.
9.1	15	19	32	11	62	39	4	2	N.A.	N.A.
10.7	8		36	21	92	37	14	41	N.A.	N.A.
12.2	50		13	14	75	38	41		N.A.	N.A.
13.7	50		16		68		51		N.A.	N.A.
15.2	80		21		82		15		N.A.	N.A.
16.8							5		N.A.	N.A.
18.3							22		N.A.	N.A.
19.8										

NOTE: 0.3048 m = 1 ft 25.4 mm = 1 in 0.1572 kN/m³ = 1 lb/ft³
[a] N.A. = Not Available

The mobilized end bearing, q_p, on the pile is found from Figure 3(b), a plot of q_p versus D/B as a function of the friction angle. A maximum end resistance of 100 tsf is stipulated for piles founded in sand.

In the case of clays, Tomlinson's method (*12*) is recommended. The average skin friction stress, *fs*, is given by

$$fs = \alpha\ Su \tag{16}$$

Alpha, which varies between 0.2 and 1.0, is given in Figure 4 as a function of clay layer's undrained shear strength, *Su*. The end-bearing stress, q_p, is given by

$$q_p = 9\ Su \tag{17}$$

where *Su* is the undrained strength of the clay layer.

DATA BASE

The locations and dimensions of the 21 augered cast-in-place piles studied are presented in Table 1. The first 19 sites were located in sands, and the last 2 were found in clays. Cases 1, 4, 8, and 19 were tension (pullout) tests and the rest were compression tests. The uncorrected SPT data are given for each of the sand sites. Table 2 lists the location of the water table and total unit weights. Also provided are the soils' internal angle of friction, based on work by Peck et al. (*11*) for sand sites, and the laboratory-measured, undrained shear strength for clay sites.

Presented in Table 3 are the predicted capacities for the 21 sites for each of the design methods using the soil information provided in Tables 1 and 2. *Qs* is the predicted skin friction, *Qp* is the tip

TABLE 2 Soil Properties

Pile No.	1	2	3	4	5	6	7	8	9	10	11
Location	Plm B.	T. Verd.	Ver. B.	Tr. Isl.	St. Pete.	W. Hav.	Ruskin	St. Pete.	St. Pete.	Tr. Isl.	St. Pete.
UW (kN/m³)	18.1	18.1	18.1	18.1	18.9	18.1	18.1	18.9	18.9	18.1	18.9
GWT (m)	.3	.3	2.7	1.5	2.1	2.1	2.1	.3	.3	1.2	7.6
Depth (m)	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Phi
1.5	31	33	28	38	32	27	33	36	31	38	36
3.0	28	32	29	34	40	32	32	43	40	34	43
4.6	39	34	30	32	33	33	36	36	40	32	36
6.1	27	36	32	39	27	35	37	31	39	39	31
7.6	39	32	45	34	43	36	35	32	36	34	32
9.1	37	35	37	32	36	35	36	30	36	32	30
10.7		33	32	28	44	37	39	30	37	28	33
12.2			33	27	41	38		34	35	34	34
13.7								30	27	27	30
15.2								36	30	30	45
16.8											
18.3											
19.8											

Pile No.	12	13	14	15	16	17	18	19	20	21
Location	Tampa	Jacks.	Savana	St. Aug.	Palatka	Cocoa	Poinc.	Palm B.	Tallah.	Tallah..
UW (kN/m³)	18.1	18.1	18.1	18.1	18.9	18.9	18.1	18.1	17.3	17.3
GWT (m)	3.4	1.5	1.5	1.5	2.1	2.1	2.1	2.1	2.1	2.1
Depth (m)	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Phi	Su (kPa)	Su (kPa)
1.5	28	29	28	29	30	29	28	36		
3.0	27	30	28	28	31	30	34	38		
4.6	27	29	33	37	32	31	32	39	57.4	57.4
6.1	28	37	35	32	31	37	32	36		
7.6	36	35	39	34	40	38	30	32	62.2	62.2
9.1	32	33	37	30	43	38	28	27		
10.7	29		38	34	47	38	31	39	86.2	86.2
12.2	41		31	31	45	38	39	26		
13.7								26		
15.2								36	119.7	119.7
16.8										
18.3									129.3	129.3
19.8										

NOTE: 0.3048 m = 1 ft 47.88 kN/m² = 1 ksf 0.1572 kN/m³ = 1 lb/ft³

TABLE 3 Pile Capacities

Predicted Capacities (kiloNewtons)							Measured Capacities (kiloNewtons)		
Pile No.	Capacity	Wright	Neely	LPC	FHWA	Coyle	2% Dia.	Davisson	5% Dia.
	Qs	338	418	587	391	489	196	205	294
1	Qp	205	578	160	205	516			
Tension	Qt	543	996	747	596	1005			
	Qs	445	436	774	525	498			
2	Qp	125	365	98	125	952			
	Qt	570	801	872	649	1450	436	365	560
	Qs	801	614	1014	898	489			
3	Qp	125	347	98	98	952			
	Qt	925	961	1112	996	1441	1005	970	979
	Qs	489	516	890	569	480			
4	Qp	107	311	89	80	267			
Tension	Qt	596	827	979	649	747			
	Qs	489	569	872	560	454			
5	Qp	196	560	151	169	943			
	Qt	685	1130	1023	729	1397	445	400	667
	Qs	320	578	605	427	302			
6	Qp	187	525	142	151	552			
	Qt	507	1103	747	578	854	649	578	783
	Qs	391	489	658	480	454			
7	Qp	133	374	107	107	658			
	Qt	525	863	765	587	1112	498	454	694
	Qs	881	427	996	898	694	285	249	445
8	Qp	187	543	151	116	952			
Tension	Qt	1068	970	1148	1014	1646			
	Qs	391	418	898	569	614			
9	Qp	187	525	142	169	943			
	Qt	578	943	1041	738	1557	569	623	818
	Qs	810	480	1103	890	676			
10	Qp	18	53	18	18	294			
	Qt	827	534	1121	907	970	783	667	890
	Qs	1112	827	1156	1130	952			
11	Qp	142	418	116	116	890			
	Qt	1254	1245	1272	1245	1841	1050	827	1201
	Qs	845	623	916	952	356			
12	Qp	320	712	249	285	667			
	Qt	1165	1334	1165	1237	1023	934	1139	1156

(continued on next page)

resistance, and Qt is their sum. None of the SPT data were corrected for overburden, and the pile-tip capacities were based on N values measured at the pile tip. Also given in the table are the measured capacities determined from load-test data by the Davisson method (*4*) as well as the measured loads at the pile top for settlements of 2 percent and 5 percent of the pile diameters. Neither Wright's nor Neely's methods are applicable, since the methods apply only to sands. The K value of 1.1 was used in the Wright and Reese approach for all cases. For each design approach, Table 4 presents the ratio of the predicted to measured capacities for each failure criterion and case. Also given at the bottom of Table 4 are the mean and standard deviation for the various failure criteria, considering all piles in the data base and considering compression piles only. It is evident from comparing mean values that the 5 percent failure criterion compares much more favorably than the 2 percent Davisson criteria—for all 5 prediction methods. Also apparent is that all of the methods compare much more favorably if the tension piles are not considered.

Presented in Figure 5 are the predicted versus measured capacities (5 percent settlement) for each design method for all piles in the data base. It is evident from Table 4 and Figure 5 that the methods proposed by Wright and FHWA (a standard deviation less than 29 percent) are the best methods of predicting the failure capacity, whereas Coyle's driven-pile approach is too high. The finding may suggest that augered cast-in-place piles behave more like drilled shafts than like driven piles. Using the 5 percent failure

TABLE 3 *(continued)*

Predicted Capacities (kiloNewtons)							Measured Capacities (kiloNewtons)		
Pile No.	Capacity	Wright	Neely	LPC	FHWA	Coyle	2% Dia.	Davisson	5% Dia.
	Qs	329	516	641	489	258			
13	Qp	142	400	107	125	952			
	Qt	471	916	747	614	1210	712	445	712
	Qs	979	569	1272	925	792			
14	Qp	133	374	107	116	685			
	Qt	1112	943	1379	1041	1477	979	979	979
	Qs	623	641	979	649	418			
15	Qp	222	632	169	196	605			
	Qt	845	1272	1148	845	1023	578	596	890
	Qs	961	614	1406	845	756			
16	Qp	623	934	489	552	1245			
	Qt	1583	1548	1895	1397	2002	1779	1245	1975
	Qs	418	525	774	560	463			
17	Qp	249	703	196	214	952			
	Qt	667	1228	970	774	1414	890	694	979
	Qs	480	525	649	703	489			
18	Qp	89	249	71	80	952			
	Qt	569	774	721	783	1441	391	302	560
	Qs	560	525	907	703	596	445	445	667
19	Qp	258	712	205	231	952			
Tension	Qt	818	1237	1112	934	1548			
	Qs	N.A.[a]	N.A.	1824	1414	1343			
20	Qp	N.A.	N.A.	116	196	169			
	Qt	N.A.	N.A.	1939	1610	1512	1646	1557	1690
	Qs	N.A.	N.A.	2002	1984	1637			
21	Qp	N.A.	N.A.	133	222	196			
	Qt	N.A.	N.A.	2135	2206	1833	1557	1557	2135

NOTE: 8.9 kN = 1 ton

[a] N.A. = Not Available

criterion, the design load (approximately 50 percent of capacity) would generally result in a settlement of less than 9 mm (0.35 in.), which most structures could sustain without damage (that is, no load-settlement approach is needed).

CONCLUSIONS

Augered cast-in-place piles are being used more and more in Florida, especially on the coast. They are used mainly under three- to six-story structures and provide uplift resistance in the event of a hurricane. They are much easier to install in the coastal, shell-filled sands than are driven, prestressed concrete piles, and they are usually less expensive. However, care in the construction of augered cast-in-place piles is important. It was identified that equipment selection, drilling rate, grout fluidity, grout's aggregate size, grout pumping, and auger removal process all significantly affect both the quality and load-carrying capacity of the pile. For instance, to prevent "weak drilling" in cohesionless sands or premature refusal in fat clays, 27 kN-m (20,000 ft-lb) power torques should be used with the auger's flight pitch equal to one-half its diameter. Grout factors between 1.2 and 1.5 should be measured. Loss of pressure on the grout feed is a good indication that there is a problem that can be corrected only by relowering the auger.

Also presented in the paper were a comparison between a data base of 21 augered cast-in-place piles (17 compression and 4 tension) and five design approaches. Three of the methods were developed for augered cast-in-place piles, the other two for driven piles and drilled shafts. The predicted capacities of various designs were compared with three different failure capacities determined from the load-settlement curves. The failure criteria used were Davisson's 2-percent and 5-percent settlements of the piles' diameter. All of the methods compared most favorably with the 5 percent criterion. Those methods proposed by Reese and O'Neill (FHWA) and by Wright and Reese gave the best predictions of capacities at settlements of 5 percent of the pile diameters. Typical ratios of predicted to measured capacity were from 0.95 to 1.04, with an average standard deviation of only 29 percent for compression and tension piles. In the case of compression loading only (17 piles), FHWA gave a mean of 0.98 with a standard deviation

TABLE 4 Ratio of Predicted to Measured Capacities

Pile No.	Wright			Neely			LPC			FHWA			Coyle		
	Davisson	2% Dia.	5% Dia.	Davisson	2% Dia.	5% Dia.	Davisson	2% Dia.	5% Dia.	Davisson	2% Dia.	5% Dia.	Davisson	2% Dia.	5% Dia.
1	1.71	1.64	1.14	2.15	2.06	1.43	3.00	2.87	2.00	2.00	1.91	1.33	2.50	2.39	1.67
2	1.30	1.56	1.01	1.82	2.18	1.42	2.01	2.40	1.56	1.49	1.78	1.16	3.33	3.98	2.59
3	0.92	0.95	0.95	0.95	0.99	0.98	1.11	1.15	1.14	0.99	1.03	1.02	1.43	1.49	1.47
4	0.81	0.87	0.70	0.86	0.92	0.75	1.47	1.58	1.28	0.94	1.02	0.82	0.79	0.86	0.69
5	1.55	1.72	1.03	2.54	2.82	1.69	2.30	2.55	1.53	1.64	1.82	1.09	3.14	3.49	2.09
6	0.77	0.87	0.64	1.69	1.90	1.41	1.15	1.29	0.96	0.89	1.00	0.74	1.32	1.48	1.09
7	1.04	1.14	0.75	1.72	1.89	1.24	1.52	1.67	1.09	1.18	1.29	0.85	2.23	2.45	1.60
8	3.10	3.54	1.98	1.51	1.72	0.96	3.49	3.99	2.24	3.16	3.61	2.02	2.44	2.79	1.56
9	1.01	0.93	0.71	1.66	1.52	1.15	1.83	1.68	1.28	1.30	1.19	0.90	2.73	2.50	1.90
10	1.06	1.25	0.94	0.69	0.81	0.61	1.42	1.67	1.25	1.16	1.36	1.02	1.24	1.45	1.09
11	1.19	1.52	1.04	1.18	1.50	1.03	1.21	1.54	1.06	1.19	1.51	1.04	1.75	2.23	1.53
12	1.24	1.02	1.00	1.43	1.17	1.15	1.25	1.02	1.01	1.32	1.09	1.07	1.10	0.90	0.88
13	0.66	1.05	0.66	1.28	2.05	1.28	1.06	1.69	1.06	0.86	1.38	0.86	1.70	2.72	1.70
14	1.13	1.13	1.13	0.97	0.97	0.97	1.41	1.41	1.41	1.06	1.06	1.06	1.51	1.51	1.51
15	1.46	1.41	0.95	2.19	2.13	1.43	1.99	1.93	1.29	1.46	1.42	0.95	1.77	1.72	1.15
16	0.89	1.27	0.80	0.87	1.24	0.78	1.07	1.52	0.96	0.79	1.12	0.71	1.13	1.61	1.01
17	0.75	0.96	0.68	1.38	1.77	1.25	1.08	1.39	0.99	0.87	1.12	0.79	1.59	2.04	1.45
18	1.46	1.90	1.02	1.99	2.57	1.39	1.84	2.38	1.29	2.00	2.59	1.40	3.68	4.76	2.57
19	1.26	1.26	0.84	1.18	1.18	0.79	2.03	2.03	1.36	1.58	1.58	1.05	1.34	1.34	0.89
20	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	1.18	1.25	1.15	0.98	1.03	0.95	0.92	0.97	0.89
21	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	1.40	1.40	1.02	1.42	1.42	1.03	1.18	1.18	0.86
i. Mean	1.23	1.37	0.95	1.48	1.65	1.14	1.66	1.83	1.28	1.35	1.49	1.04	1.85	2.09	1.44
i. St. Dev.	0.52	0.59	0.29	0.50	0.56	0.28	0.63	0.68	0.32	0.53	0.61	0.28	0.81	1.01	0.52
ii. Mean	1.10	1.24	0.89	1.49	1.70	1.19	1.46	1.64	1.18	1.21	1.36	0.98	1.87	2.14	1.49
ii. St. Dev.	0.26	0.30	0.16	0.50	0.58	0.27	0.38	0.43	0.19	0.31	0.39	0.16	0.93	1.05	0.53

i. includes both compression and tension piles
ii. includes compression piles only
N.A. - not applicable

of 0.16, and Wright gave a mean of 0.89 with a standard deviation of 0.16. Another important conclusion from the case studies is that augered cast-in-place piles behave more as drilled shafts than as driven piles. The use of 5 percent of the pile's diameter for the failure criterion is believed to be acceptable for typical augered cast-in-place piles in the .30 m (12 in.) to .41 m (16 in.) range, since settlements of 7.6 mm (.3 in.) to 10.2 mm (0.4 in.) are considered acceptable for most structures.

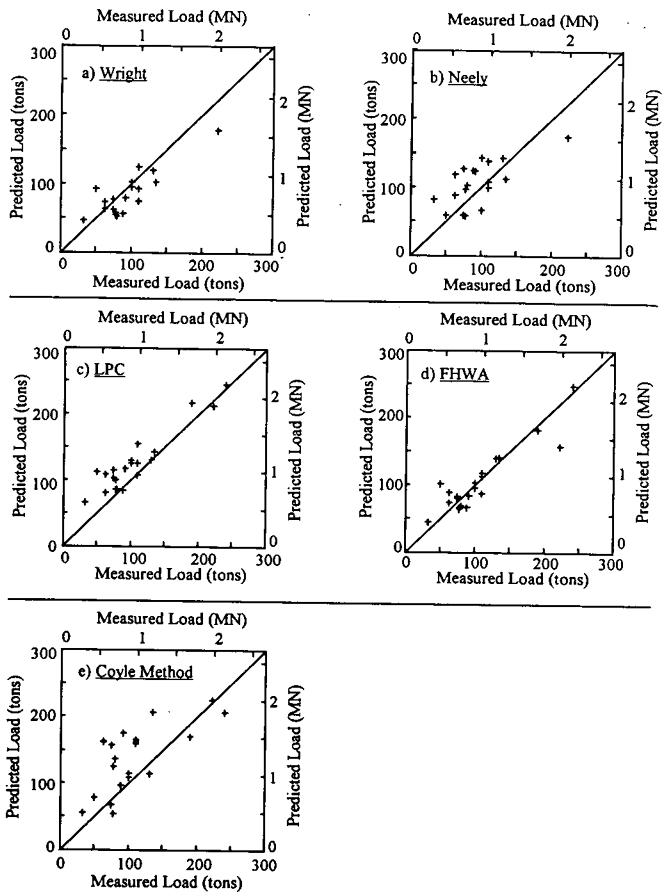

FIGURE 5 Predicted versus measured capacities at 5 percent pile diameter settlement.

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