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ABSTRACT

Auger Cast In Place (ACIP) Piles have been used increasingly for various types of projects including industrial and commercial buildings, multi-story parking garages, docks and other structures. The performance of auger cast in place piles is dependent on several factors including the soil type, rate of auger extraction and pumping of grout, grout strength, grout pressures and grout ratio. Performance of piles can be judged by pile load tests performed on test piles constructed under similar conditions. This paper presents case studies involving eight pile load tests performed on auger cast piles installed at four different sites in Texas Gulf Coast Area. The stratigraphies at these sites include over-consolidated stiff to very stiff clay and loose to medium dense sandy silt and silty sands. The test piles had diameters ranging from 14-inches to 24-inches and extended to depths ranging from 55 feet to about 95 feet. Test piles were constructed in general accordance with Deep Foundation Institute's (DFI) specifications for construction of auger cast-in-place piles. Compression load tests were performed to failure and load-movement relationships were developed. The load test results were compared with the load carrying capacity calculated using some available methods and skin frictional resistance was back calculated and examined.

INTRODUCTION

Auger Cast-In-Place (ACIP) piles are constructed using a continuous flight, hollow shaft auger that is advanced into the ground to a specified depth in a continuous operation by rotation. After the desired depth is reached, grout is pumped through the auger shaft while the auger is being retrieved from the ground. During the installation process, the flights of the auger capture the soil and inhibit the borehole from caving, avoiding the use of drilling fluids or casing during construction. Van Impe, W. (1988) has discussed some of the construction issues related to the rate of auger extraction versus rate of pumping. Model specifications and guidelines for grout properties, and the method of installation for auger cast-in-place piles can be found in DFI (1993). Depending on the actual soil conditions found at a project site, some adjustments to these guidelines are required. The performance of auger cast in place piles is dependent on several factors including the soil type, rate of auger extraction and pumping of grout, grout strength, grout pressures and grout ratio.

This paper presents case studies involving eight pile load tests performed on ACIP piles installed at four different sites in Texas Gulf Coast Area. Pile load tests on 14-inch to 24-inch diameter auger-cast-in-place piles installed in over-consolidated stiff to very stiff clay (Pleistocene deposits) and loose to medium dense sands were studied.

DESCRIPTION OF TEST SITES

The four sites located in the Texas Gulf Coast Region are: 1) Port of Freeport in Freeport, Texas designated as PF; 2) Pasadena, Texas, designated as PT; 3) Downtown Houston, designated as DH; and, 4) Freeport, Texas designated as FP. The stratigraphy at these sites is discussed below.

The first site is located at Port of Freeport in Freeport, Texas and is designated as 'PF'. Table 1 below shows the soil stratigraphy at this site.

Table 1. Soil Conditions at Port of Freeport (PF), Texas Site

Depth (feet)	Description	Soil Strength
0 to 6	I-Very soft Dredged Fill Soils	-
6 to 24	II- Stiff Fat Clay	Su: 1.4 ksf
24 to 48	III- Loose to Medium Dense Silty Sand	SPT-N: 18
48 to 100	IV-Stiff to Very Stiff Fat Clay	Su: 1.9 ksf

Note: Ground water at 15 feet; Su-Average Undrained Shear Strength; Average SPT-N Field Standard Penetration Test blows per foot.

Site 'PT' is located in the Pasadena, Texas. Table 2 below shows the soil stratigraphy at this site.

Table 2. Soil Conditions at Pasadena, Texas (PT) Site

Depth (feet)	Description	Soil Strength
0 to 28	Firm to Very Stiff Clay	Su: 1.4 ksf
28 to 38	Medium Dense Silt w/ sand	SPT-N: 25
38 to 48	Firm Fat Clay	Su: 1.0 ksf
48 to 62	Stiff Fat Clay	Su: 2.0 ksf
62 to 100	Medium-Very Dense Sand	SPT-N: 68

Note: Ground water at 25 feet; Su-Average Undrained Shear Strength; Average SPT-N Field Standard Penetration Test blows per foot.

Site 'DH' is located in the downtown Houston, Texas. Table 3 below shows the soil stratigraphy at this site.

Table 3. Soil Conditions at Downtown Houston (DH) Site

Depth (feet)	Description	Soil Strength
0 to 5	Stiff Lean Clay	Su: 1.4 ksf
5 to 30	Medium Dense Sandy Silt	SPT-N: 18
30 to 55	Very Stiff Fat Clay	Su: 2.8 ksf
55 to 70	Stiff Fat Clay	Su: 2.0 ksf
70 to 100	Very Stiff Fat Clay	Su: 2.6 ksf

Note: Ground water at 18 feet; Su-Average Undrained Shear Strength; Average SPT-N Field Standard Penetration Test blows per foot.

Site 'FT' is located in Freeport, Texas. Table 4 shows the soil stratigraphy at this site.

Table 4. Soil Conditions at Freeport, Texas (FT2)

Depth (feet)	Description	Soil Strength
0 to 35	Firm to Stiff Fat Clay	Su: 1.0 ksf
35 to 80	Stiff Fat Clay	Su: 1.4 ksf
80 to 100	Very Stiff Fat Clay	Su: 2.8 ksf

Note: Ground water at 25 feet; Su-Average Undrained Shear Strength.

PILE LOAD TEST RESULTS

A total of eight pile load tests in axial compression were performed on auger cast-in-place piles located at four different sites. Three pile load tests were performed on 14-inch diameter ACIP piles installed at Port of Freeport, Texas, PF site. Two pile load tests were performed on 16-inch diameter ACIP piles installed at Pasadena Texas, PT site. Two pile load tests were performed on 18-inch diameter ACIP piles installed at Downtown Houston, Texas, DH site. One test was performed on 24-inch diameter ACIP pile at FT site. The length of the test piles ranged from 55 feet to about 95 feet. The compressive pile load tests were performed in general accordance with ASTM D 1143.

Based on the stratigraphy and pile lengths, it is noted that all the piles tips, except for piles PT-1 and PT-2 were bearing in stiff to very stiff clay soil. During the installation of the test piles, rate of auger extraction and pumping of grout, grout strength, grout pressures and grout ratio were monitored for most piles. The grout ratio i.e., the ratio of the actual volume of the grout pumped into the hole to the theoretical volume of the pile was found to be at least 1.30. The grout pressure at the pump outlet ranged from approximately 200 psi to 400 psi. Table 5 summarizes the pile load test results or the measured failure load on the test piles.

Table 5. Summary of Pile Load Test Results

Test Pile ID	Diameter (inch)	Pile Length (feet)	Grout Ratio	Observed Failure Load (kips)
PF-1	14	55.0	1.80	300
PF-2	14	60.0	1.70	440
PF-3	14	90.0	1.80	480
PT-1	16	65.0	1.38	420
PT-2	16	85.0	1.57	480
DH-1	18	68.5	1.34	600
DH-2	18	93.0	1.86	900
FT-1	24	55.0	1.30	286

For each pile tested, the applied load and pile movement observed were normalized. Figure 1 shows the normalized curves of pile load versus pile movement. From each pile load test data, the failure load was divided by the applied load and the pile movement was divided by the respective pile diameter. From Figure 1, it can be seen that failure occurred at movement equal to 3 to 6 percent of the pile diameter. Also, for a load equal to 50 percent of the failure load (typical design load), the pile movement could be equal to 0.5 to 1 percent of the pile diameter.

Out of the three tests (designated as PF-1 through PF-3) performed at Port of Freeport site, two tests PF-1 and PF-3 were instrumented with load cells and strain gauges. The load cell and strain gauges were mounted on a 2-inch diameter PVC pipe and all the wires were secured inside the pipe and a cap was placed at the bottom. The PVC pipe assembly was then pushed into the just grouted ACIP pile. Results of the instrumented pile load tests are shown on Figure 2 and Figure

3. The plots show the load distribution along the length of ACIP piles, PF-1 and PF-3. It should be noted that both piles are bearing in the stiff to very stiff fat clay layer (IV). Table 1 shows the generalized stratigraphy at the pile locations. From Fig 2 and Fig 3 below, it is observed that the pile tip load was about 20 kips for piles PF-1 and PF-3. From the load distribution plots, side friction along the length of the pile was estimated for the four layers separately and the results of this analysis are discussed later.

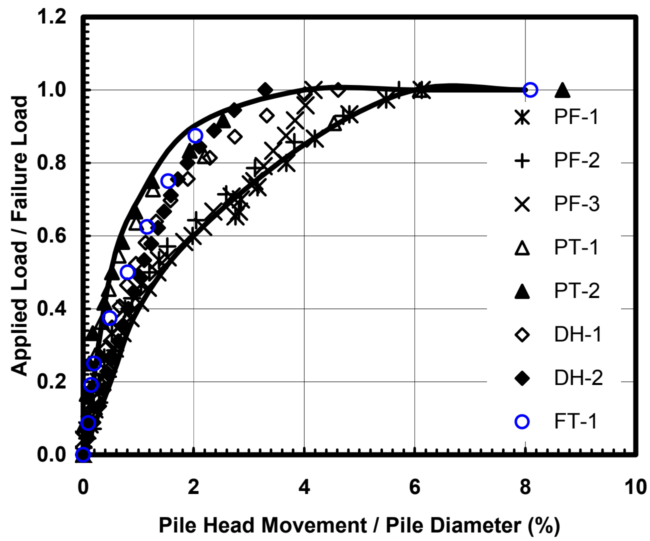

Fig 1. Normalized pile load versus normalized movement

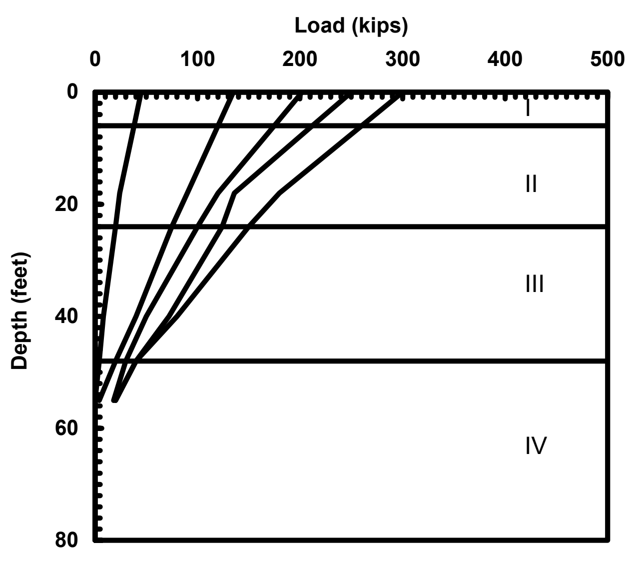

Fig 2. Load Distribution of Pile PF-1 (55 ft long;14-in Dia.)

PREDICTED VERSUS OBSERVED CAPACITY

Axial compressive capacities of the piles were computed using two different methods: 1) FHWA-88 method (Reese and O'Neill 1988) and 2) Modified FHWA-88 method (Long and Wysockey 1999). Both methods predict axial capacity for drilled shafts. Currently, procedures to compute axial capacity for auger cast piles are not available to our knowledge. An attempt is made to use the drilled shaft FHWA method for ACIP piles. For piles embedded in clay, the procedures used in predicting the side resistance is known as α-method; i.e., unit skin friction is a factor α multiplied by the shear strength of the soil. For FHWA 88 method, α is taken as 0.55, while for the Modified FHWA-88 method, α value varies (Chen & Kulhaway, 1994). For piles embedded in sands, the procedures involved in predicting the side resistance and end bearing are different for both methods. Table 6 below shows the observed and predicted axial capacities for the ACIP piles tested. It is noted that both methods generally under predict the axial capacity, except for those piles bearing sands, i.e., PT-1 and PT-2, in which case Modified FHWA 88 method overestimated the capacity.

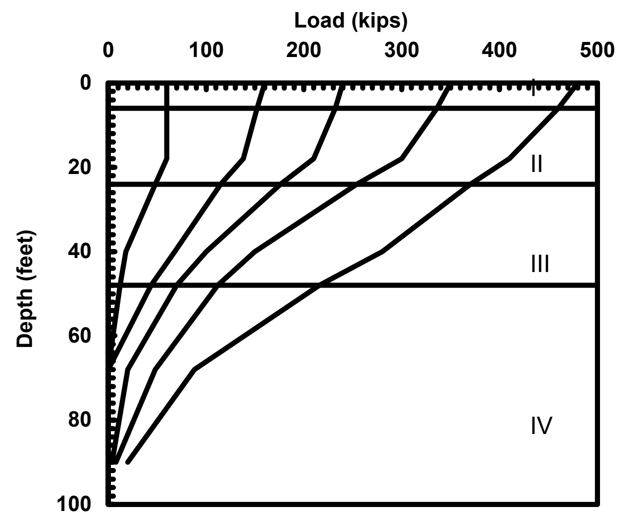

Fig 3. Load Distribution on Pile PF-3 (90 ft long; 14-in Dia.)

Table 6. Observed Axial Capacity Versus Predicted Capacity

Test Pile ID	Observed Failure Load (kips)	FHWA-88[1] (kips)	Modified FHWA-88[2] (kips)
PF-1	300	221	221
PF-2	440	240	239
PF-3	480	356	343
PT-1	420	388	435
PT-2	480	409	891
DH-1	600	488	286
DH-2	900	660	348
FT-1	286	239	264

Note: (1) Reese and O'Neill (1988); (2) Long and Wysockey (1999)

SKIN FRICTION

For the Port of Freeport site, skin friction was estimated for the various layers using the data from the load distribution curves shown in Fig 2 and Fig 3. The slope of the line within a layer indicates the shear stress or skin friction transferred to the soil. Table 7 shows the unit skin frictional resistance for the four layers identified at PF site (see Table 1 for stratification) using the load distribution data for piles PF-1 and PF-3. Table 7 also shows the unit skin frictional resistance computed using the FHWA 88 and Modified FHWA-88 methods.

Table 7. Skin Friction in ksf for the soil layers at PF site

Depth (feet)	Soil Layer	PF -1	PF-3	Col. (1)	Col. (2)
0 to 6	I – Fill	1.77	0.89	-	-
6 to 24	II– Clay	1.63	1.33	0.77	0.79
24 to 48	III– Sand	1.22	1.71	1.42	1.44
48 to L	IV– Clay	0.76	1.35	1.05	0.95

Col. (1) FHWA-88; Col. (2) Modified FHWA–88; L is the pile length of 55 feet for PF-1 and 90 feet for PF-3.

The skin friction values for Stratum I are ignored due to variability associated with material types found during sampling and testing. For clay strata i.e., II and IV the measured skin friction values were generally greater than the predicted values. For the sand strata, it appears that the average friction value using the data from both piles is more or less similar to the values obtained by FHWA-88 and Modified FHWA-88 method. Based on the data, it is can be seen that both methods under predict the skin frictional resistance for clay soils. However, for sand strata, it is possible that the methods may result in a reasonable estimate of skin frictional resistance. For clay soil strata, the α values were back calculated from the observed skin frictional values. The back-calculated α values range from 0.40 to 1.16, with an average value of 0.87, which is comparatively higher than the recommended value of 0.55 according to FHWA method.

Although, all load-tested piles were not instrumented, an attempt is made to estimate the skin friction transferred along the length of the pile from the data observed. Skin frictional resistance was estimated for clay stratifications only, as the piles were mostly piles were embedded in clay profiles. To obtain the load carried in skin friction from the clay layers, the following procedure was adopted. From the observed failure load, the load carried by end bearing and the amount of frictional load carried by sand layers were subtracted. Appropriate end bearing load and the frictional load carried was computed by FHWA-88 method. The load obtained after subtracting the end bearing and sand friction from the failure load was divided by the pile surface area that is embedded in the clay stratification. This process could result in average combined skin frictional resistance within clay layers. Using this skin frictional resistance and weighted average shear strength of the combined clay layers, the α– values were back calculated. Back-calculated α–values and the weighted average shear strength for each pile are shown on Table 8.

Table 8. Back Calculated 'α–Values'

Test Pile ID	Weighted Average Shear Strength* (ksf)	α–Value
PF-1*	1.40 (II) & 1.90 (IV)	1.16 (II) & 0.40 (IV)
PF-2	1.60	0.55
PF-3*	1.40 (II) & 1.90 (IV)	0.95 (II) & 0.70 (IV)
PT-1	1.48	0.53
PT-2	1.48	0.53
DH-1	2.42	0.78
DH-2	2.49	0.67
FT-1	1.15	0.61

Note: (*) The alpha values were back calculated from the instrumented pile load tests.

Based on the back calculated α–values, it is noted that some values were greater than 1.0 and, are also greater than 0.55 (a values suggested for drill shaft according to FHWA 88). It is established by various researchers (Reese and O'Neill 1988) and in this study that the α values for ACIP piles are greater than those used for drilled shafts as mentioned in the FHWA-88 method. It is noted that ACIP piles do not behave like drilled shafts due the differences in the installation techniques. Two major noted differences are: 1) the pressure with which the grout is placed in the open borehole and, 2) the amount of grout placed in the open borehole or grout ratio.

Typically during pile installation, the grout is pumped at a pressure ranging between 200 psi and 400 psi. With these pressures, the grout pressures at the exit point or at the tip of the auger shaft could be large enough to either expand the borehole diameter or spread enough grout paths by pushing the soil and thus creating a lager surface area of the pile than the theoretical embedded pile surface area. Although no attempt was made to measure the grout pressure at the exit point, a theoretical estimate can be made by accounting for the loss of pressure in the pipe leads. It is estimated that the net exit pressure could be about 150 psi, which is enough to fracture the clay soil having shear strength of up to 2 ksf.

Moreover, for the piles considered in this study, the grout volumes pumped into the boreholes vary from 1.34 to 1.80 times their respective theoretical volume of the boreholes drilled. During installation, DFI recommends to place a grout volume equal to or greater than 1.3 times the theoretical volume of the pile. It is believed that if more grout can physically be pumped into a borehole of certain volume, then in order to accommodate this increased volume of grout, the borehole size has to increase. This increase in volume could result in increase in diameter of the borehole or dilated borehole walls. It is believed that due to this increase in diameter, the embedded surface area is increased resulting in greater skin frictional capacity of the pile. However, it would be interesting to compare the size and shape of an excavated pile with the theoretical size and shape of a pile.

CONCLUSIONS

Based on the results of the load tests and the analyses of the results the following is concluded for ACIP piles installed Texas Gulf coast soils.

1. For all piles failure occurred at pile movement equal to 3 to 6 percent of the pile diameter.

2. At the design load or a load equal to 50 percent of the failure load the pile movement could be equal to 0.5 to 1 percent of the pile diameter.

3. The axial capacity predicated using FHWA 88 method used drilled shafts is lower than the observed failure load of the tested ACIP piles.

4. For ACIP piles, embedded clay soils, skin frictional resistance predicted by FHWA 88 method is lower than the measured values obtained from the load tests. This is due to the installation techniques adopted i.e., grout pressure and grout ratio, two factors that may greatly influenced axial carrying capacity of ACIP piles.

5. Based on the back-calculated α-values, an average α-value of 0.69 was observed and therefore a value greater than 0.55 could be used for clay stratifications of Texas Gulf Coast soils.

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