EVALUATION OF PILE LOAD TEST IN THE BEAUMONT FORMATION

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ABSTRACT: Results of full scale load tests on two auger-cast piles constructed in the Beaumont Formation are presented. Load tests were conducted on two, 18-inch diameter piles extended to a depth of 45 feet as part of the design for a 30-story apartment building to be constructed in Houston, Texas.

Results of the two load tests indicated settlement of less than 0.1 inch for a design load of 300 kips. Both piles were able to withstand maximum test loads of 500 and 636 kips, with 0.9 and 0.6 inches of movement, respectively.

Use of auger-cast piles reduced the foundation costs for the high-rise by over $600,000.00 compared to use of conventional piers or a mat foundation. Geotechnical conditions and pile load deflection curves are provided.

INTRODUCTION

Auger-cast piles are constructed by drilling a continuous flight auger into the ground and, upon reaching the required depth, pumping cement-sand grout down a hollow-stem as the auger is steadily withdrawn. The sides of the holes are supported at all times by the soil-filled auger, eliminating the need for temporary casing or bentonite slurry. It is an economical and less time consuming technique than most other pile or drilled pier installation techniques. Furthermore, these is no vibration, very little noise, and no ground movement commonly associated with driven piles (Neely, 1991). Since the grout is pumped under pressure, it penetrates the soil and provides a good bond with the surrounding soil. The pressure also provides some compaction of soil (Caduto, 1994). The method can be used with relative ease even below ground water which causes difficulties for drilled shaft construction.

The method is very sensitive to the skills of the construction personnel. Proper coordination between the auger withdrawal and grout pumping is extremely important. If the auger is withdrawn too quickly or not rotated sufficiently, the grout may become contaminated with the soil sloughing into the shaft from the sides or with the excavated soil. Placement of heavy reinforcing steel cage can be difficult limiting the lateral load capacity of the pile. The method is not suitable in soils containing large boulders and cobbles because of the difficulty associated with excavation of such soils with an auger.

Even though there is significant difference between the construction techniques, no distinction in design methods for auger-cast piles and other soil-replacement-type piles exists in most published design methods (Neely, 1991). Neely (1991) compared the values from two empirical design methods and proposed a new method for estimating pile capacity in sand.

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GEOLOGY

The site is underlain by the Beaumont Formation overlying the Lissie Formation. The Beaumont Formation is an alluvial environment composed of clay and sandy clay grading with depth to clayey sand, sand, and gravel with occasional lenses of silty and sandy clay. Due to depositional nature, this formation includes many point-bar, stream channel, and marshland deposits. The thickness of this Formation is estimated at about 100 feet. The underlying Lissie Formation is composed of clay with silt and sand deposits. Minor amounts of gravel are also known to be present. The thickness of the Lissie Formation is estimated at 200 feet.

Site specific conditions consisted of 1 to 8 feet of fill overlying high to moderate plasticity, firm to stiff clays and sandy clays (Units I and II) grading below approximate depths of 28 to 38 feet to very dense clayey sand with some thin gravel lenses (Unit III).

Unit III extended to depths of 48 to 50 feet and is underlain by red to reddish-brown and yellowish-red very hard clay (Unit IV), which is in turn underlain below depths of 88 to 98 feet by very dense red clayey sand (Unit V).

Ground water was present at depths of 19 to 22-1/2 feet below site grades at the time of field investigation. A typical boring log is presented on Figure 1. Standard penetration test results are plotted graphically on Figure 1.

GEOTECHNICAL PARAMETERS AND DESIGN PILE CAPACITY

Pile load capacity and working load were evaluated using two procedures, one by Neely (1991) and the second by Quiros and Reese (1977). A brief discussion of the methods and results are presented in the following paragraphs.

Based on the results of 66 load tests, Neely (1991) proposed empirical formulas for computing the unit end bearing and skin friction resistance of auger-cast piles in sand. The load tests were conducted on piles with diameters between 12 and 24 inches and lengths varying from 15 to 85 feet.

The unit end bearing is correlated with Standard Penetration Test values (SPT values) as:

$$q_p = 3.8N \leq 150 \tag{1}$$

Where:

$q_p =$ unit point (or end) bearing resistance in kips per square foot (ksf)

$N =$ SPT values without overburden correction.

Similarly, the average unit skin friction resistance is computed using the β method:

$$q_s = \beta\sigma'_v \leq 2.8 \text{ ksf} \tag{2}$$

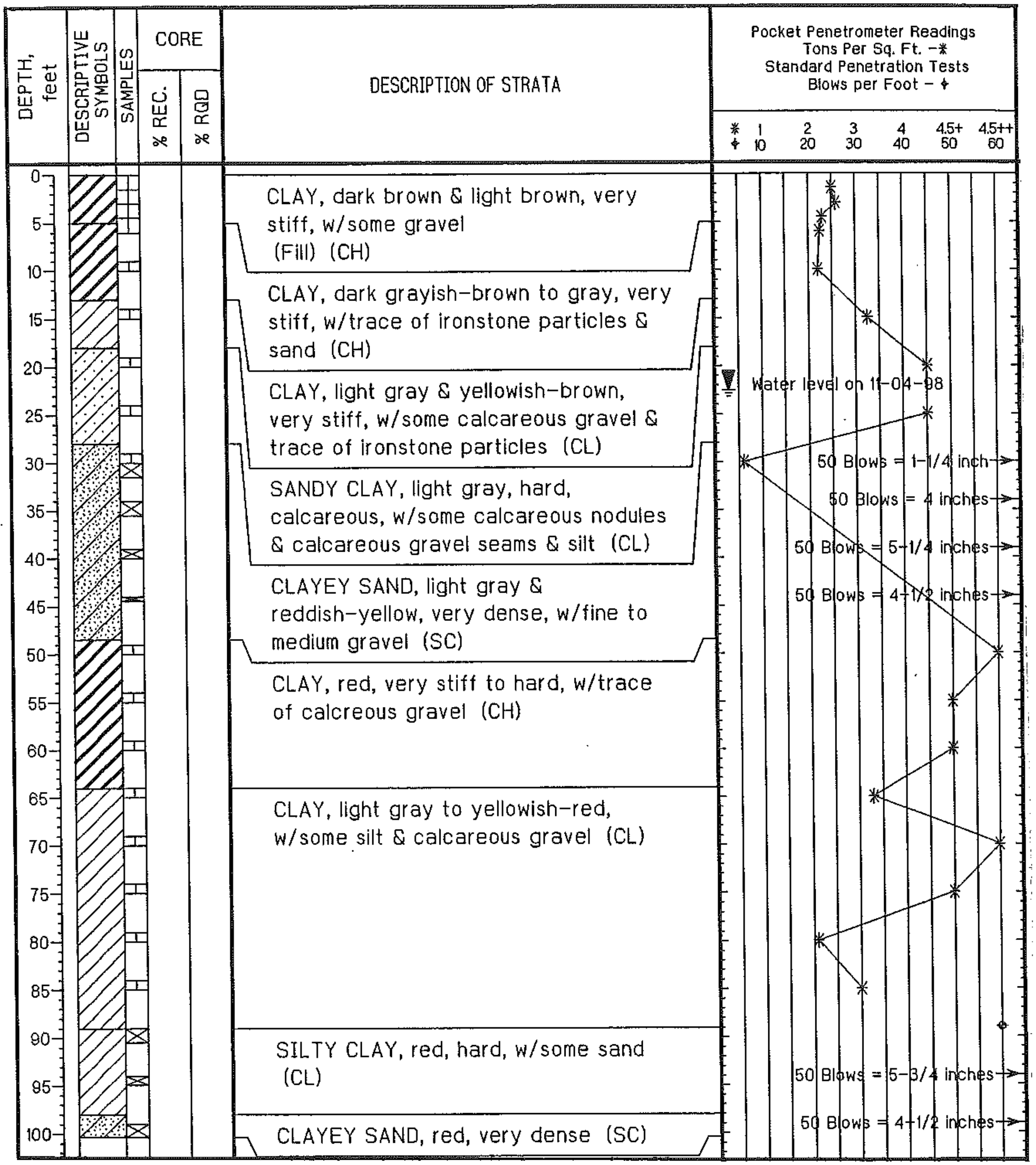

Figure 1. Generalized Log of Boring.

Where:

q_s = average unit skin friction resistance in ksf

β = average skin friction factor

σ'_v = average vertical effective stress along length of pile in ksf

The ultimate end bearing and skin friction values were computed using eqs (1) and (2). The ultimate end bearing value was found to be governed by the limiting value of 150 ksf. The ultimate skin friction value computed ranged from 1.6 to 1.7 ksf.

Quiros and Reese (1977) proposed correlations based on load tests on drilled shafts founded on various types of soil in Texas including the type encountered at this site.

Skin friction in clayey soils is given by:

$$q_s \text{ (ksf)} = \alpha_{avg}\, s_u \quad \text{(with a limiting value of 4.0 ksf for shaft drilled dry)} \qquad (3)$$

Where:

α_{avg} = strength reduction factor (0.6 for shaft drilled dry)

s_u = undrained shear strength in ksf.

Similarly for shafts drilled dry or by slurry method, the skin friction and end bearing in sand are given by:

$$q_s \text{ (ksf)} = 0.052N \qquad \text{(with a limiting value of 4.0 ksf)} \qquad (4)$$

$$q_p \text{ (ksf)} = 80/k \text{ (for very dense sand and for tip movement of one inch)} \qquad (5)$$

Where:

$k = 1.0$ for $D \leq 1.67$; $k = 0.6D$ for $D \geq 1.67$

D = shaft base diameter in feet.

The term, k, is a factor which reduces the tip (or point) capacity for shafts with a base diameter larger than 20 inches so as to limit the shaft settlement to one inch (Quiros and Reese, 1977). An overall factor of safety of at least 2.5 is recommended by the authors for computing working load capacity using the correlations presented by eqs. (4) and (5).

The ultimate skin friction and end bearing values were computed using the Quiros and Reese procedure (eqs. 3, 4 and 5). The ultimate skin friction values resulted in of 1.2 ksf for Units I and II, and 11.0 ksf for Unit III. The skin friction value is limited to 4.0 ksf for eqs. 3 and 4. The ultimate end bearing value was found to be governed by the limiting value of 80.0 ksf.

As one can see by comparing the calculated ultimate skin friction values, the β method (eq. 2) gives a lower skin friction value than Quiros and Reese method (eqs. 3 and 4). Moreover, while using the β method, it is not recommended to divide the soil into layers to determine the β value for each layer. Only one β value should be determined based on the length of the pile. However, the ultimate end bearing value obtained using eq. (1) is higher than the one obtained by eq. (5)

The design values obtained using equations (3), (4), and (5) were recommended for computing the pile capacity for the reason that they represented the local conditions more closely than the other design method. The recommended design values are presented in the following table. The design values contain an overall factor of safety of three considering a shear or plunging failure.

DESIGN SKIN FRICTION AND END BEARING VALUES			
Pier Bearing Strata	Minimum Penetration Below Surface (feet)	Allowable End Bearing (ksf)	Skin Friction (ksf)
Piers founded on top of Unit III (clayey sands)	Top of very dense clayey sand, depths of 28 to 38 feet	14, D.L. 21, D.L. + L.L.	0.6 (D.L.) 0.9 (D.L. +L.L.) (within Units I and II)
Piers founded within or below Unit III.	28 to 38 feet	14, D.L. 21, D.L. + L.L.	2.5, D.L. 3.5, D.L. + L.L. (within Unit III and below)

D.L. = Dead Load, L.L. = Live Load

Based on the design values presented in the above table, the working load capacity of 300 to 350 kips were estimated for an 18-inch diameter, 45-foot long auger-cast pile.

INSTALLATION OF TEST PILES

Two test piles were installed at the site in April, 1999. A set of four reaction piles were also constructed for each test pile. The piles were 18 inches in diameter and extended to a depth of 45 feet below the ground surface. For each test pile, the reaction piles were installed in a rectangular fashion with center to center spacing of 22 feet along the long direction and 10 feet along the short direction. Test pile was installed at the center of the rectangle. The piles were constructed using a continuous flight auger which allowed for the piles to be grouted in place. High strength grout consisting of a mixture of cement, sand, and fly ash was used to cast the piles.

For the first test pile (Test Pile 1), a single, 45-foot long, 1-1/4 inch diameter, threaded steel bar (Dywidag bar) was placed in each reaction pile. A steel reinforcing cage, 15 feet in length consisting of four No. 5 bars with No. 3 ties on 12 inch centers was inserted into the test pile after the completion of grouting. A design grout strength of 4000 psi was used for this test pile.

The reaction piles for the second test pile (Test Pile 2) were installed adjacent to the reaction piles installed for Test Pile 1. A minimum of 11 feet of clear spacing was maintained between the two sets of reaction piles. Pile dimension and depths were similar to the Test Pile 1 and reaction piles (i.e. the piles were 45 feet deep and 18 inches in diameter). The Dywidag bars on the reaction piles were 45 feet long and 1-3/8 inch in diameter. A 5,000 psi compressive strength grout was used for this test pile.

Approximately 20 minutes was required to drill, grout, and complete each pile. Delays were primarily due to the slow delivery of grout to the site. No control on the slump or water content of the grout was noted during the construction of the test set-up. The maximum grout pump pressure varied from 1100 to 1600 pounds per square inch (psi). Test cubes were made from both types of grout mixes.

PILE LOAD TEST PROCEDURE

Load tests were conducted on Test Pile 1 and Test Pile 2 on April, 19 and 20, 1999 after the results of the compressive strength tests on the test cubes indicated that the grout had reached desired strength. Test Pile 1 was subjected to a maximum load of 250 tons applied at an increment of 12.5 tons at a time interval of 2-1/2 minutes. Test Pile 2 was subjected to a maximum load of 318 tons applied at an increment of 15.9 tons at a time interval of 2-1/2 minutes. For both test piles, the maximum load represented the upper limit of the loading frame and jack. Pile settlement readings were taken for each load increment immediately after application of the load and immediately prior to application of the next load increment. The test piles were unloaded after the settlement readings corresponding to the final load increment (maximum total load) were taken. Subsequently, the corresponding reading for the pile rebound was taken five minutes after the total load had been removed.

TEST RESULTS AND DISCUSSION

The load test results are presented graphically on Figs. 2 and 3. The elastic compression of the pile grout was computed for each load increment and deducted from gross pile settlement to obtain the net pile settlement. The design grout strengths of 4000 psi and 5000 psi for the Test Pile 1 and Test Pile 2, respectively were used to compute the elastic compression of the pile grout.

The Young's modulus of concrete is given by:

$$E_c = 57000\,(f_c')^{1/2} \qquad (6)$$

Where:

E_c = Young's modulus of concrete in pounds per square inch (psi)

$f_c^{'}$ = 28-day compressive strength of concrete (psi)

The elastic compression was computed using the relationship given by:

$$\varepsilon_c = PL/AE_c \qquad (7)$$

Where:

ε_c = elastic compression of pile concrete along the pile length;

P = applied load ; L = length of pile; and A = cross-sectional area of the pile

The computed elastic compressions of the pile grout were comparable to the field observed rebound of the test piles following unloading. As shown on Figs. 2 and 3, the results of the load test show that the piles experienced net settlement of less than 0.1 inch for the design loads of 250 kips and 318 kips for Test Pile 1 and Test Pile 2, respectively. Both piles were able to withstand maximum test loads of 500 and 636 kips, with 0.9 and 0.6 inches of gross settlement, respectively. Due to limitations of the test load equipment, neither pile was tested to ultimate failure.

Pile load test results proved that the recommended skin friction and end bearing values were valid for design with adequate factors of safety considering both settlement and shear failure.

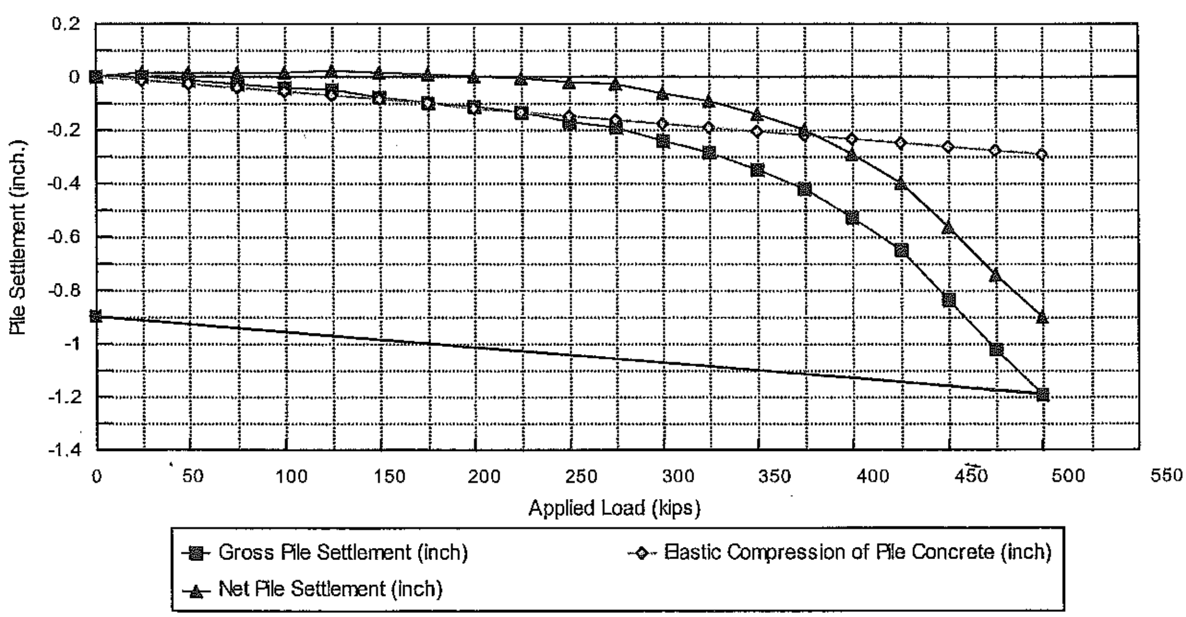

Figure 2. Load Test Results, Test Pile 1.

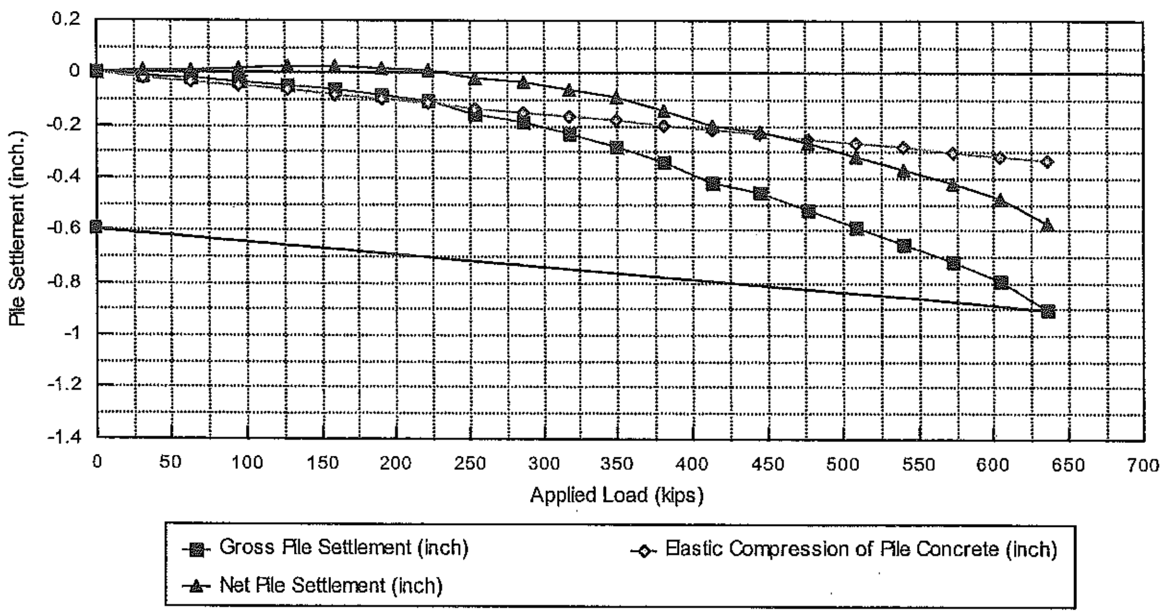

Figure 3. Load Test Results, Test Pile 2.

CLOSING

The load tests proved that the auger-cast piles provided a more economical foundation alternative than the conventional pier or mat foundations. The mat foundation alternative were also analyzed. Based on the analysis, it was concluded that a mat with considerable rigidity would be required to account for the settlement that the upper clays are likely to undergo under this type of foundation. Subsurface conditions were not favorable for a conventional drilled pier foundation. The auger-cast piles because of its ease of construction both above and below the ground water and less time consumption, provided a viable alternative to other foundation alternatives considered. Use of auger-cast piles reduced the foundation costs for the high-rise by over $600,000.00 compared to use of conventional piers or a mat foundation.

APPENDIX. REFERENCES

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