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Axial Performance of ACIP Piles in Texas Coastal Soils

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Abstract

Methods were evaluated for assessing axial compressive capacities of augered, cast-in-place (ACIP) piles in the Pleistocene terrace deposits of the Texas Gulf Coast and Recent to Modern alluvial soils. The study involved a combination of data base analyses and the performance of new loading tests on instrumented piles. The results in sand deposits mirrored earlier studies that indicated that common methods used for designing drilled shafts are also appropriate for ACIP piles, although the patterns of load transfer in the tested piles suggested zones of both lower load transfer (running sands below the water table) and higher load transfer (compacted surface soils) than would be expected in drilled shafts. The action of drilling and grouting resulted in net increases in lateral effective stresses in the soil around the tested piles, near the ground surface, where such measurements were made. In overconsolidated clays, slightly higher unit side resistances were inferred for ACIP piles than are predicted by common drilled shaft design methods. However, common drilled shaft design methods, notably the FHWA method, produced generally accurate capacities for mixed soil profiles.

Introduction

Augered, cast-in-place (ACIP) piles are well suited to the stiff clays and generally medium dense fine sands of the Texas Coastal Plain. These piles are generally constructed using a continuous flight, small-diameter, hollow-stem auger that is advanced to a prescribed toe elevation in a continuous operation and then withdrawn while pumping fluid grout through the hollow stem, under pressure, into the space vacated by the tip of the auger. If a reinforcing cage is needed, it is inserted into the fluid grout to depths of 15 m or more before the grout begins to loose its fluidity. A photograph of an ACIP rig is shown in Fig. 1. In coastal Texas, the torque applied to the auger, through a power plant such as the one in the foreground of the photo, is relatively low, since the soils to be penetrated are relatively weak. However, the torque is not high enough to screw the auger into the ground. Instead, soil is scraped off the bottom of the borehole, and perhaps off the sides, and routed up the turns of the auger to the surface in order to advance the borehole. The soil that was captured

Figure 1. ACIP pile rig

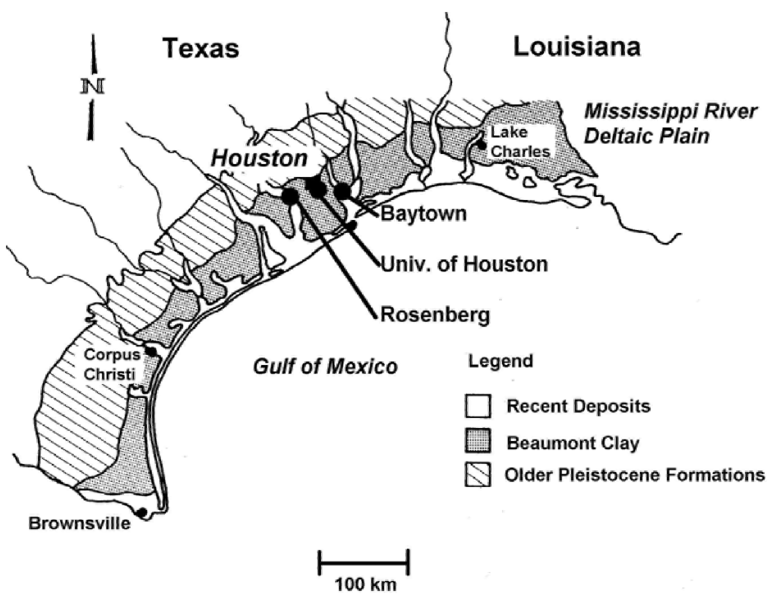

Figure 2. Geology of the Texas Gulf Coast, showing ACIP pile test site locations

within the flights of the auger at any one time inhibits caving of the borehole, making it unnecessary to use drilling fluids or casing in the process. If the soil is of the type and consistency to "run" into the borehole and up the auger during excavation, this operation can result in "mining" of the soil, in which more soil is excavated than the volume of the borehole. This can reduce effective stresses and relative density in the soil around the pile being installed and perhaps around piles that have already been installed. Soil "mining" with lower-torque rigs is minimal in the overconsolidated stiff clays and sands typical of the Pleistocene terrace formations of the Texas and western Louisiana Gulf Coasts, but it is nonetheless an issue in developing and applying design equations.

Another important issue in capacity evaluation relates to whether the auger extraction process leads to voids or necks in the finished pile, which can occur if the rate of auger extraction exceeds the rate of pumping of grout into the space vacated by the auger. This issue, which impacts structural integrity, has hindered the application of ACIP piles on publicly funded projects. Van Impe (1988) and Esrig et al. (1994) address procedures related to these and other construction issues.

The Texas Department of Transportation (TxDOT) began using ACIP piles as foundations for sound walls in the mid-1990's in the Houston area for reasons of environmental safety, economy and minimization of disruption of human habitats adjacent to the location of pile construction. Earlier studies to develop design rules for laterally loaded ACIP piles in Texas coastal soils, in association with the design of sound wall foundations, were reported by O'Neill et al. (2000). With the advent of new quality control devices in the late 1990's, TxDOT contemplated the use of ACIP piles as bearing piles for bridges and other structures in the coastal areas of Texas. Design rules for ACIP piles were unclear in the coastal terrace formations, however, and concerns with issues such as soil mining led to the conclusion that a limited research program should be undertaken before ACIP piles should be used in actual structures. This paper summarizes that study.

The technical literature and files of consultants and contractors in coastal Texas were reviewed for loading test data and related geotechnical information. Several methods for computing axial compressive capacities of driven piles and drilled shafts were then selected to compare with the loading tests in the resulting data base, which contained data for 43 loading tests (O'Neill at al., 1999). Tests were classified according to the general soil profile (clay, sand or mixed clay and sand), and pile capacities were defined as the loads corresponding to a settlement of 5 per cent of the nominal pile diameter (generally, 0.46 m or less). Since relatively little test data could be found for ACIP piles in sands in coastal Texas, the data base was augmented with tests reported by McVay et al. (1994) performed on ACIP piles in Florida sands. In order to develop a more detailed understanding of the performance of ACIP piles in Texas coastal soils, beyond the insights provided by the data base study, instrumented test piles were constructed and load tested at three geologically diverse sites in coastal Texas. These tests were performed specifically to evaluate the effects of site geological details on pile capacity.

The three test sites, located as shown in Fig. 2, consisted of a site in overconsolidated clay (University of Houston National Geotechnical Experimentation Site, or "UH" site) (O'Neill and Yoon, 1995); loose, normally consolidated fine, submerged sand beneath a layer of compacted surface soil and above a layer of stiff clay ("Baytown" site); and moist, medium dense to dense sand ("Rosenberg" site). The first site was on a pro-delta deposit that was preconsolidated in Pleistocene times by desiccation and can be considered a typical deep foundation site in the coastal region of Texas; the second site was on the Modern alluvium of the San Jacinto River / Houston Ship Channel at the location of the Fred Hartmann cable stayed bridge (where running sand and soil mining were a concern); and the third site was in Recent alluvium (where the sand was unsaturated and had the potential of removing water from the grout mix so quickly that the grout might not flow properly or accept the insertion of the reinforcing cage). Summary soil profiles of the three test sites are given in Figs. 3 - 5.

The three test piles contained full-length reinforcing cages that were instrumented with foil-resistance strain-gauged sister bars for the measurement of distribution of load to the soil. The lengths of these piles are indicated on Figs. 3 - 5. Each had a nominal diameter (auger diameter) of 0.456 m (18 in.).

Construction

Details of the construction operations, including continuous profiling of maximum and minimum grout pressure, incremental grout volume, rate of auger penetration, rate of auger withdrawal, and similar factors were reported by O'Neill et al. (1999).

If soil mining has an important effect on axial capacity, a net reduction in lateral effective stress in the soil adjacent to the pile would be expected to occur during construction. That is, the reduction in effective stress that would be produced by soil flowing into the borehole and being routed up the auger during drilling would be greater than the increment of positive effective stress that would be produced by introducing the pressurized grout. Therefore, one lateral effective stress cell was placed in the soil adjacent to each test pile, at a depth of 3.05 m, as shown in Fig. 6. Readings were made continuously as the auger was inserted and extracted. A typical record, for the Baytown site, is shown in Fig. 7. In this case the stress cell was located at the bottom of a compacted fill layer directly above the running sand. As seen, the stress cell sensed the downward passage of the auger by registering an increase in effective stress in the form of a spike. Once the auger tip passed the cell, there was a small negative increment of effective stress (decrease from *in situ* value), which suggests some mining of the soil at the elevation of the cell. However, when the auger was withdrawn and the grout was simultaneously pumped under approximately 1500 kPa of systolic pressure (at the outlet of the pump located on the ground), the lateral effective stress began to increase when the auger tip was well below the cell and remained elevated until the auger was completely withdrawn. The increase in soil stress with the auger tip below the sensor was a result of the contractor's keeping a significant head of grout in the flights of the auger as the auger was withdrawn — a highly desirable practice. A net increase in lateral effective stress, σ'_h, of 20 kPa was realized at the depth of the effective stress cell (3.05 m) through the entire process, suggesting that the effects of the mining that occurred at the depth of the sensor were more than compensated by the pumping of the grout. Similar information was recovered from effective stress sensors at the other two test sites at a depth of 3.05 m.

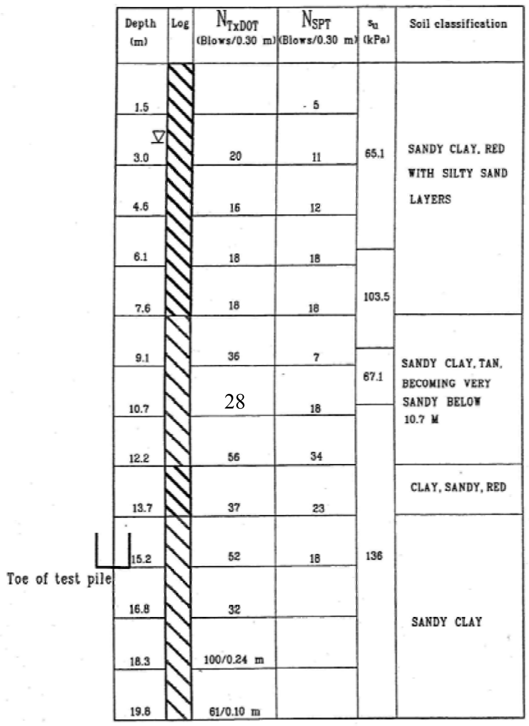

Figure 3. Soil profile at UH test site (clay)

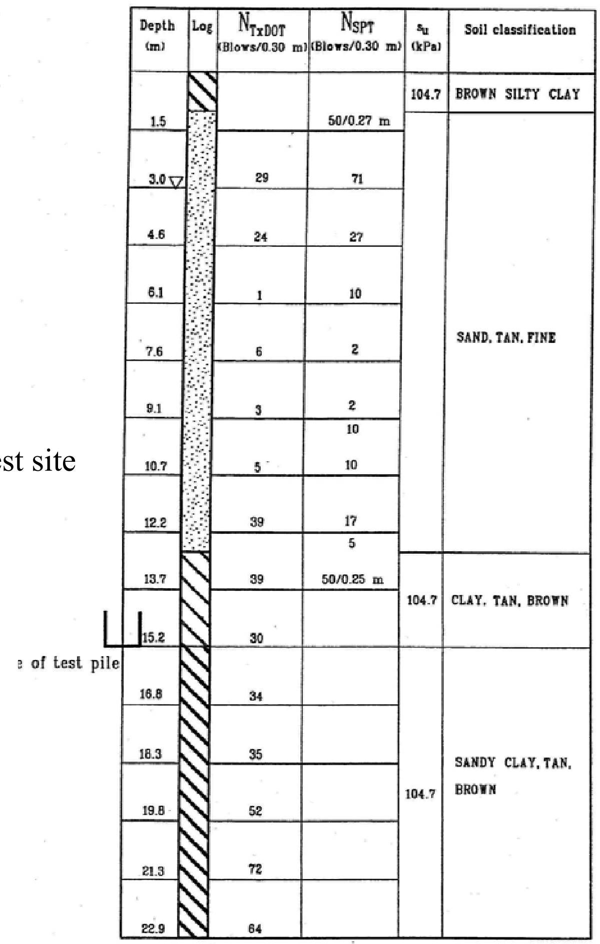

Figure 4. Soil profile at Baytown test site (mixed sand and clay)

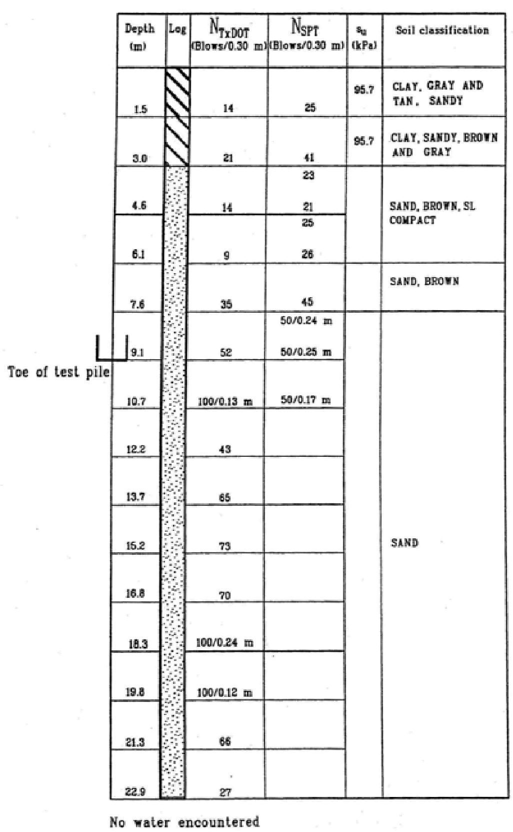

Figure 5. Soil profile at Rosenberg test site (mixed sand and clay)

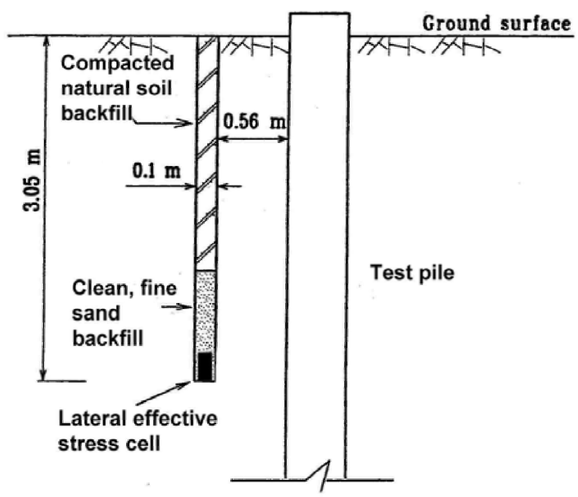

Figure 6. Position of lateral effective stress cells

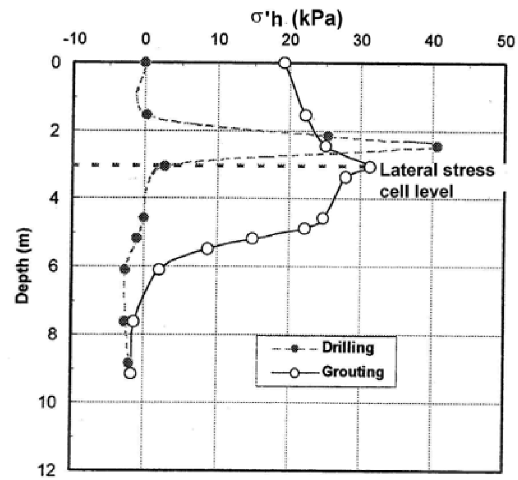

Figure 7. Lateral effective stress (σ'_h) vs. depth of tip of auger (Baytown site)

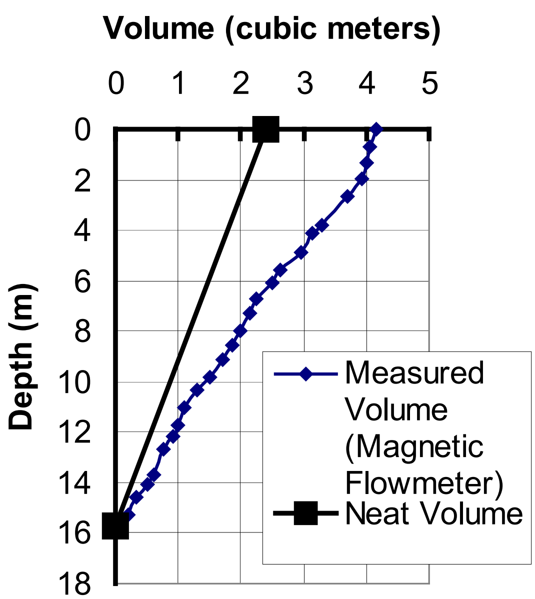

Figure 8. Grout volume pumped vs. depth of auger tip (Baytown site)

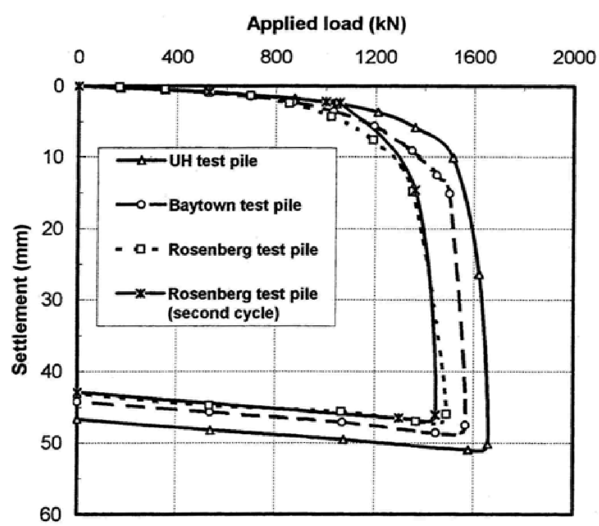

Figure 9. Load vs. settlement (3-minute readings) for all loading tests

Soil stress sensors could not be placed within the running sand below a depth of 4.6 m at the Baytown site, where the net effect of soil mining and grouting may have been different. For that effect, the grouting and load transfer data from the load test must be consulted. The incremental volumetric grouting record for the Baytown test pile, (from magnetic flow meter and auger depth sensor), Fig. 8, suggests a higher rate of grout placement (in response to maintaining constant systolic pressure) in the depth range of the grout discharge port on the auger tip of about 3 to 10.5 m than in depth ranges above and below. The systolic pump pressure was relatively constant when the auger tip was below about 4 m (1200 kPa) and actually increased (to about 1800 kPa) above a depth of about 4 m. This suggests recompression of loose soil around the pile in the running sand zone below 4.6 m and perhaps occupation of space outside the neat diameter of the borehole by grout in that depth range, replacing soil that had been mined. Load transfer data for axial loading in that zone are described subsequently.

Loading Tests

The results of axial compressive loading tests on the three project-specific test piles are shown in Fig. 9. The tests were conducted according to the Texas Quick Load Test Method, in which small increments of load are added every three minutes monotonically until failure (settlement of 5 per cent of the nominal pile diameter) occurs. One test was performed at UH and Baytown; however, two successive tests were performed on the pile at Rosenberg to assess whether any cyclic degradation or strengthening (as by compaction of soil at the pile toe) occurred. Little difference in the results of the two Rosenberg-site load tests can be observed in Fig. 9. Upon their completion, the results of these tests were placed in the data base described above, bringing the total number of cases to 46.

Estimation of Ultimate Resistance and Comparisons with Measurements

Several existing deep foundation capacity estimation methods were tested against the data base for the prediction of axial resistance of the ACIP piles (O'Neill et al., 1999). One method, which proved to be relatively accurate for the tests in the data base that were conducted in mixed clay and sand profiles, was the Coyle-Castello method for driven piles with augmentation for clay layers using Tomlinson's method (Coyle and Castello, 1981). Another method, which proved to be accurate within the data base for piles in sand profiles was the method of Neely (1991). However, the most accurate methods (defined as those with the smallest coefficient of variation in the ratio of measured to predicted capacities and the smallest average error) were the TxDOT UU triaxial compression test (TAT) method for drilled shafts in clay soils (O'Neill et al., 1999) and the FHWA method for drilled shafts (Reese and O'Neill, 1988) for sand profiles and mixed sand and clay profiles.

The TxDOT TAT and FHWA capacity estimation methods are summarized succinctly in Fig. 10. In clay soils, or in clay layers in mixed profiles, the TxDOT TAT method assigns a higher ratio of ultimate unit side resistance f_{max} to undrained shear strength s_u than does the FHWA method (0.70 vs. 0.55) and has no exclusion

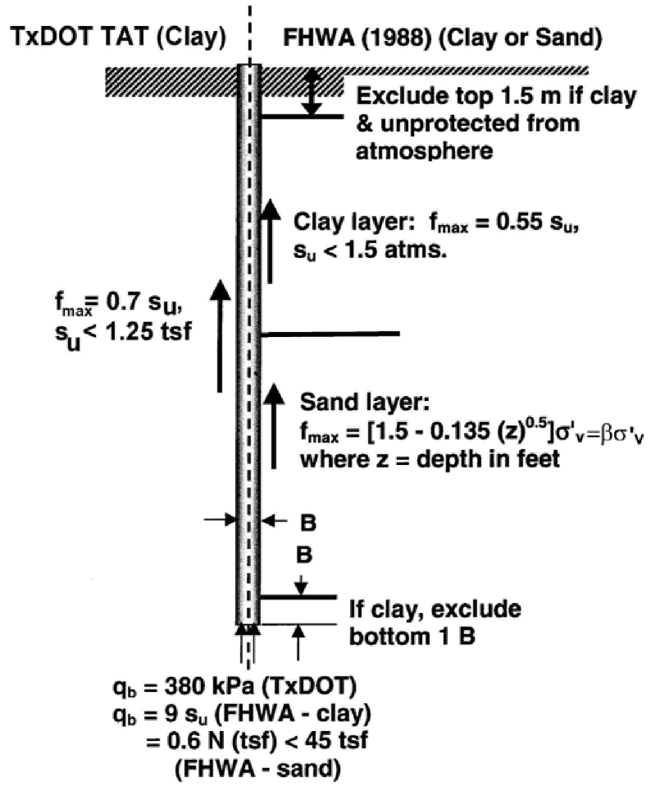

Figure 10. Summary of TxDOT TAT (Clay) and FHWA design methods for drilled shafts

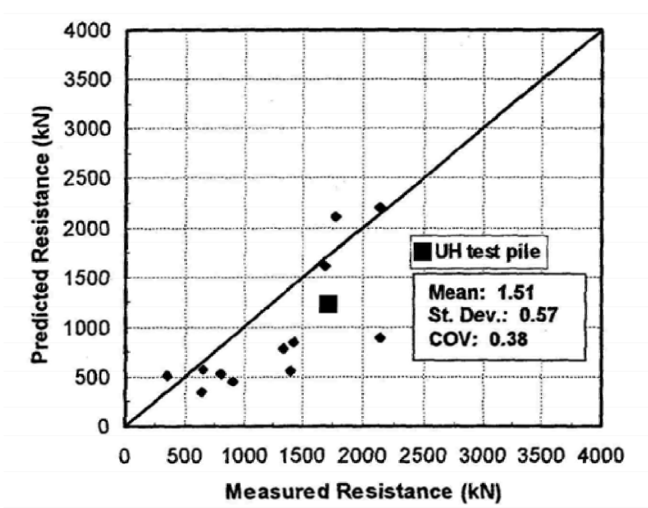

Figure 11. Predicted vs. measured resistance — FHWA method in clay

zones. It also assigns a limiting unit toe resistance, q_b, of 380 kPa regardless of the strength of the soil around the toe (because it assumes that toes in small-diameter shafts cannot be cleaned fully), whereas the FHWA method assigns resistances in proportion to s_u (clay) or to SPT blow count (sand).

Results of the correlations between measured and computed capacities from the data base are shown in Figs. 11 - 15 for the FHWA and TxDOT TAT drilled shaft methods in clay, the FHWA drilled shaft method in sand, and the Coyle-Castello-Tomlinson driven pile and FHWA drilled shaft methods in mixed sand and clay profiles. In those figures the mean values are ratios of mean measured resistance (corresponding to settlement of 5 per cent of the nominal pile diameter) to computed resistance, while the standard deviations and coefficients of variation (COV) apply to the same ratios. Based on these comparisons and similar comparisons for other methods investigated, the TxDOT TAT drilled shaft method was selected as the most appropriate method for design in clay profiles, and the FHWA drilled shaft method was selected as the most appropriate method in sand and mixed sand / clay profiles.

In other notable data base studies with ACIP piles in sand, McVay et al. (1994), working with a data base similar to, but not identical to, the data base for sand considered in this study, concluded that ACIP piles in Florida sands behave more like drilled shafts than driven piles. They concluded that the FHWA method yielded mean predictions that were nearly equal to the measured mean compression capacities when compared to the loads corresponding to head settlements of 5 per cent of the nominal pile diameter. More recently, Zelada and Stephenson (2000), using a different data base than those used either the present authors or McVay et al., also concluded that ACIP piles in coarse-grained soils developed about the same compressive capacities that are developed by drilled shafts. The Coyle-Castello driven pile method and Neely's ACIP pile method were deduced to give the most accurate toe resistances. The FHWA drilled shaft method was found to give the closest prediction of unit side resistance, although a superior correlation method was found when factor β in the FHWA method in Fig. 10 was reduced to 80 per cent of the value shown on that figure and Neely's unit toe resistance was modified to 1.7 N (tsf), which is higher than the unit toe resistance given by the FHWA design method for drilled shafts (Fig. 10) by a factor of 2.83. This trend toward high unit toe resistance was not observed in the current study, however, as described later.

While the studies of McVay et al. and Zelada and Stephenson conclude that in sand ACIP piles behave generally as drilled shafts for design purposes, the observation in the present study is that the TxDOT TAT method, with its high ratios of f_{max} to s_u, is more accurate than the FHWA drilled shaft method for ACIP piles in clay. This suggests that the unit side resistance of ACIP piles in clays of the type found along the Texas Gulf Coast (stiff and insensitive) may be somewhat higher on the average than unit side resistance developed in drilled shafts in similar soil.

Detailed Load Transfer Analysis

Although the selected design methods tended to predict the overall axial capacity of the test piles accurately, observation of the load transfer patterns suggests that this may be an artifact of compensating errors — over-prediction of resistance in some layers and under-prediction in others. Consider Fig. 16, which shows measured load versus depth for the Baytown test pile. A significant portion of the load was

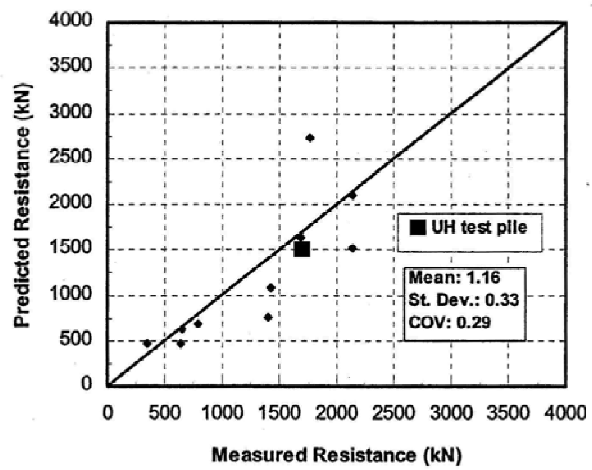

Figure 12. Predicted vs. measured resistance — TxDOT TAT method in clay profiles

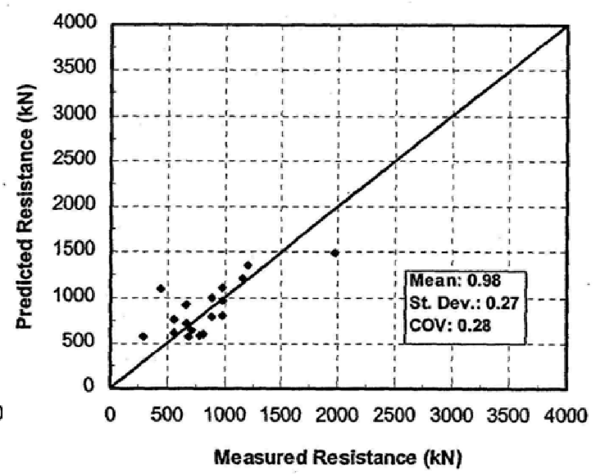

Figure 13. Predicted vs. measured resistance — FHWA method in sand profiles

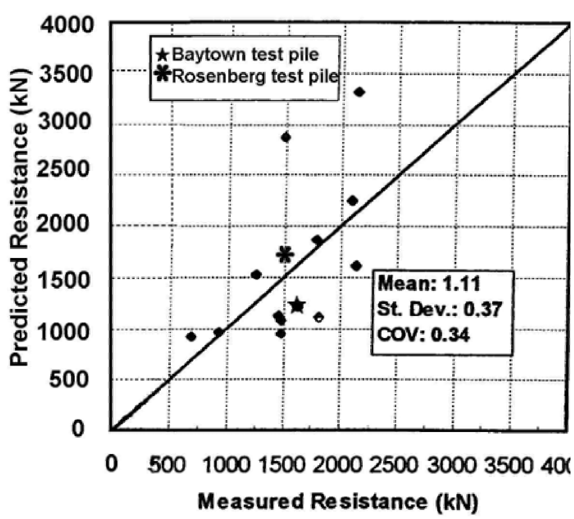

Figure 14. Predicted vs. measured resistance — Coyle-Castello-Tomlinson method in mixed sand-clay profiles

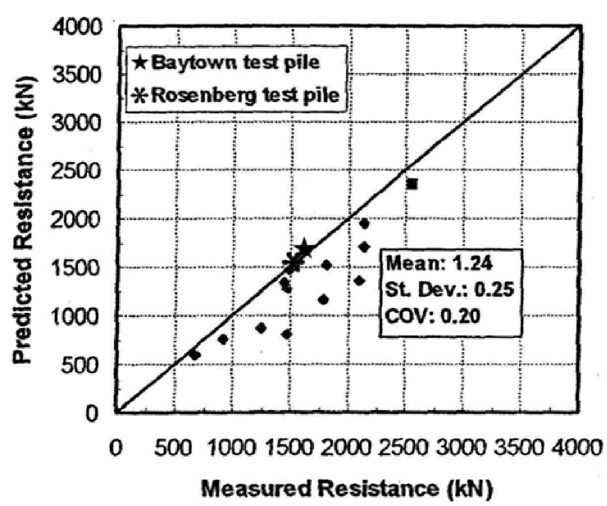

Figure 15. Predicted vs. measured resistance — FHWA method in mixed sand-clay profiles

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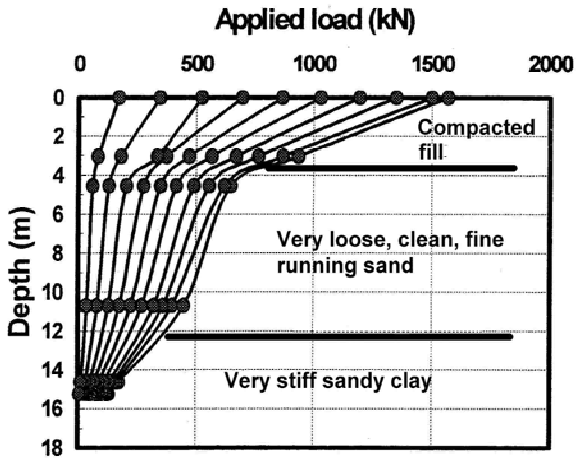

Figure 16. Load-depth relations for Baytown test pile

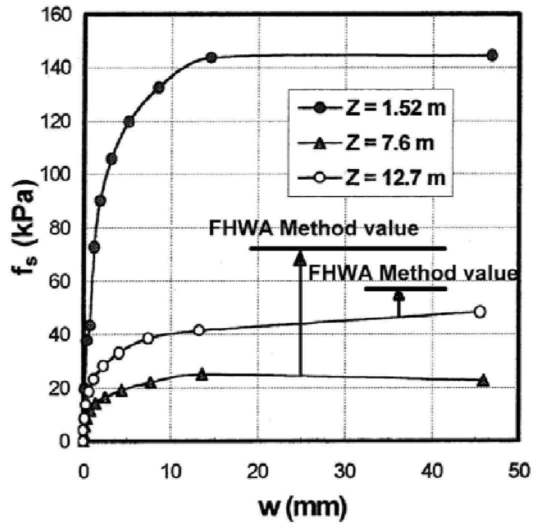

Figure 17. Side load transfer relations for Baytown test pile

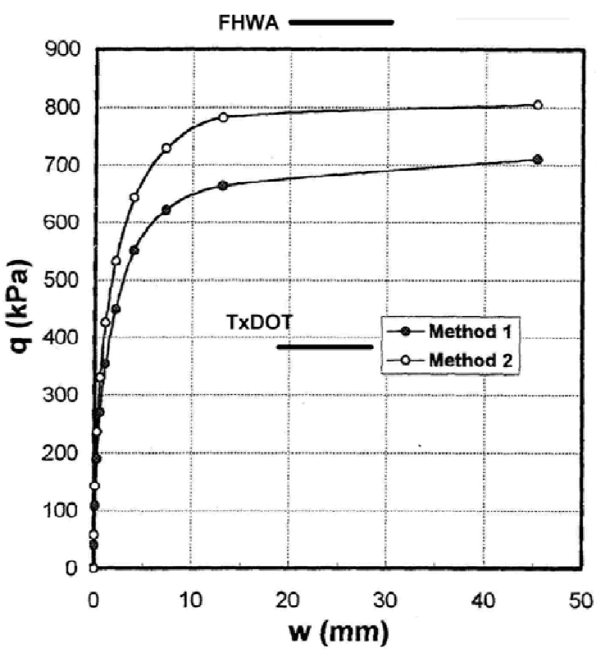

Figure 18. Toe load transfer relations for Baytown test pile

transferred to the soil in the upper 4 m, within the compacted fill. In the natural soil below 4 m, the rate of load transfer was considerably less, especially within the loose, running sand. The load-depth and load-settlement curves have been transformed into relationships of unit side resistance (f_s) versus settlement (w) in Fig. 17, where it becomes obvious that the ultimate side resistance in the running sand (z = 7.6 m) was much lower than the value predicted by the FHWA design method, and the ultimate value in the lower clay (z = 12.7 m) was slightly lower than the value predicted by the FHWA (and the TxDOT TAT) method. The high load transfer in the compacted fill near the surface may be at least partially due to the oversizing of the borehole in the top of the pile and is associated with the increased value of σ'_h in Fig. 7. The ultimate toe resistance (q) shown in Fig. 18 (interpreted using two different methods of extrapolating measured load to the toe of the pile, O'Neill et al., 1999) is slightly less than is predicted by the FHWA method but greater than the 380 kPa design limit imposed by the TxDOT TAT method.

A generally similar pattern was observed in the tests on the Rosenberg test pile in Fig. 19 — high load transfer in the surficial soil (stiff, unsaturated clay) and values in the moist sand near the middle and toe of the pile that are slightly less than are predicted by the FHWA drilled shaft design method. The toe load transfer at a settlement of 45 mm, or 10 per cent of the pile diameter (Fig. 20) was less than that predicted by the FHWA method, but it appears that had deflections been greater the toe resistance may have eventually reached the value predicted by the FHWA method. This finding is in contrast to the findings of Zelada and Stephenson (2000), who suggested that unit toe resistances were higher in sands for ACIP piles than for drilled shafts. The fact that the sand at Rosenberg appeared to be slightly cemented at the pile toe, as reflected by the high N values in Fig. 5, may have contributed to this result, in which any effect of cementation reflected in the SPT test is destroyed by the excavation of the borehole with the continuous flight auger.

Although space does not permit showing the corresponding data from the UH site (overconsolidated clay), the measured ratios of f_{max}/s_u varied from 0.59 to 0.87, which are higher than specified by the FHWA drilled shaft design method and, on the average, approximately equal to those specified by the TxDOT TAT drilled shaft design method. The toe resistance at a movement of 10 per cent of the pile diameter was 930 kPa, or about 75 per cent of the value generated by the FHWA method and more than twice the limiting value specified by the TxDOT TAT method.

Detailed analysis of the load transfer data therefore suggest that the ACIP piles tested specifically for this study developed unit side resistances in overconsolidated clay or compacted surface soils that were higher than those expected for drilled shafts (in proportion to the shear strength of the soil). Unit side resistances in loose, running sand and unit toe resistances in both sand and clay were lower than expected in drilled shafts. Future design procedures should consider these details.

Further Design Considerations

Factor of safety. Bias and scatter in predictions of compressive axial geotechnical resistances of ACIP piles should be considered in selecting factors of safety. Factors of safety corresponding to various levels of safety, as indicated by the target safety index β_T, were computed from the data in Figs. 11 - 15, using a technique first applied

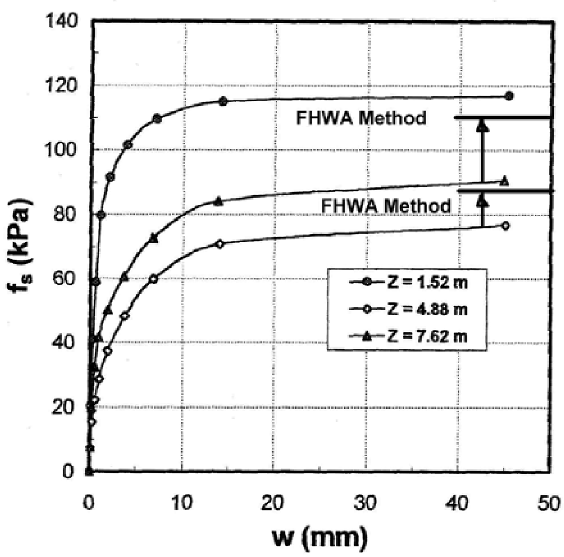

Figure 19. Side load transfer relations for Rosenberg test pile

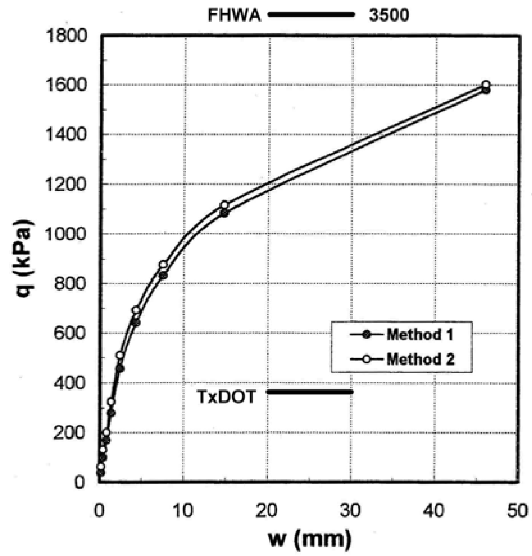

Figure 20. Toe load transfer relations for Rosenberg test pile

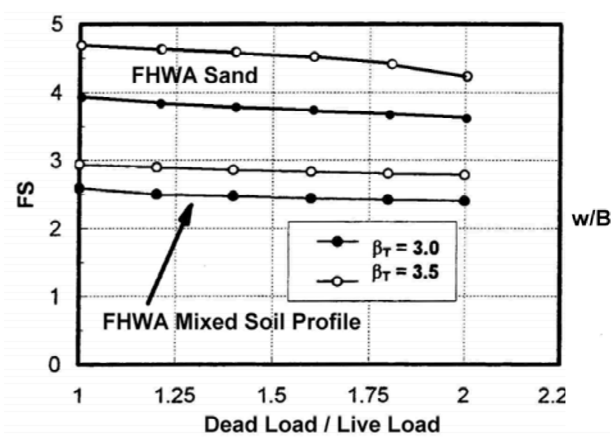

Figure 21. Computed safety factors (FS) for selected safety indices (β_T) and dead/live load ratios

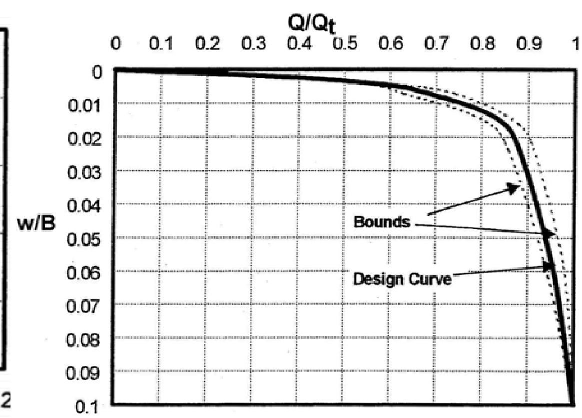

Figure 22. Normalized load-settlement curves for design

by Barker et al. (1993) and documented further by Yoon and O'Neill (1997). This technique uses typical biases and variances in loads from nominal AASHTO design methods and the biases and coefficients of variation measured in the set of field tests to assess the factor of safety. It is generally sufficient to assume $3.0 < \beta_T < 3.5$, so that the factors of safety inferred from Figs. 11 - 15 on the basis of ratio of nominal dead load to nominal live load are as shown in Fig. 21 for sand profiles and soil profiles that have a mixture of sand and clay layers. Since there is an overall conservative bias for the design method in the mixed sand-clay soil profiles, the factor of safety corresponding to a selected safety index is smaller than in sand profiles, where there is no conservative bias but where considerable scatter occurs. For the TxDOT TAT method in clay, the factors of safety for the same values of β_T are between those for the FHWA method for sand and for mixed profiles. The study suggests that the safety factor should depend on the design method and the type of soil profile in which the piles are installed, e. g., 2.6 to 2.8 with the FHWA method in mixed soil profiles.

Load-settlement behavior. For the three load tests conducted in diverse soil profiles, it was possible to construct a normalized load-settlement curve that fit all of the sites quite well. This curve, in which load is normalized by ultimate total resistance Q_t and settlement is normalized by nominal pile diameter B, is given in Fig. 22. This design curve should be suitable for single ACIP piles of similar diameter in the geologic formations of the Texas Gulf Coast.

Conclusions

The methods used to measure grout pump pressures and incremental grout volume continuously indicated that ACIP test piles were structurally sound. Although there was some indication of minor soil mining in sands, the process of excavating and grouting produced a net increase in lateral soil effective stress at a depth of 3 meters at each of the three test sites (overconsolidated clay, compacted fill over running sand, and moist sand). In clays, the TxDOT TAT drilled shaft method, which employs f_{max}/s_u = 0.70 and a base resistance limit of 380 kPa, provided better prediction of capacity than the FHWA drilled shaft method [f_{max}/s_u = 0.55 with exclusion zones and unit base resistance of 9 s_u]. In sands and mixed sand-clay profiles, the FHWA drilled shaft method provided generally better predictions than other methods. Through statistical analysis of the body of test data (46 tests) factors of safety were shown in Fig. 21 to vary with design method and soil profile type (sand, clay, mixed).

Acknowledgments

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