INTERPRETED RESIDUAL LOAD IN AN AUGERED CAST-IN-PLACE PILE

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Accurate representation of the geotechnical resistance distribution is critical in evaluating the performance of piles and pile groups for design. A current practice is to infer the geotechnical resistance distribution based on measured changes in strain during conventional top load pile testing. Especially for drilled and bored piles, the residual load is typically assumed to be negligible both during testing and after the application of the structure load. A practical result is that the compressive structural load is anticipated to be opposed primarily by the shaft resistance with little or no mobilized toe resistance. The predicted top deflection is estimated as the combined elastic compression plus the movement necessary to mobilize shaft resistance. The development of residual load (and associated negative skin friction) results in a significantly different design condition. Negative skin friction tends to force the pile into the ground and develops residual load in the pile that contributes to the internal compressive load. Significant toe resistance can be mobilized in order to resist the combination of negative skin friction and the sustained compressive load. The pile movement becomes a function of the toe penetration required to mobilize sufficient resistance to reach a balance of forces. In an effort to study the development of residual load in augered cast-in place (ACIP) piles, the authors installed and monitored strain gages that were embedded in an 18-inch diameter ACIP pile at a site in southeast Georgia. The results of this study indicate that residual load develops over time and reaches a distribution that is consistent with the principles of the unified design of piles. It is concluded that residual load develops in ACIP piles (as well as all drilled and bored piles by the same mechanisms) and that the residual load should be considered in the interpretation of load tests and in the pile design, particularly in the estimation of the single pile and pile group settlement.

Introduction

Accurate representation of the geotechnical resistance distribution is critical in evaluating the performance of piles and pile groups for design. A current practice is to infer the geotechnical resistance distribution based on measured changes in strain during conventional top load pile testing. For drilled and bored piles in particular, the residual load is typically assumed to be negligible both during testing and after application of the structure load. A practical result is that the compressive structural load is anticipated to be opposed primarily by the shaft resistance with little or no mobilized toe resistance. The predicted top deflection is estimated as the combined elastic compression plus the movement necessary to mobilize shaft resistance (usually 0.1 to 0.2 inches).

Where residual load is present, the shaft resistance has reversed (*i.e.*, negative skin friction) along the upper portion of the pile and tends to force the pile into the ground. Significant toe resistance can be mobilized in order to resist the combination of negative skin friction and the sustained compressive load. The pile movement becomes a function of the toe penetration required to mobilize sufficient resistance to reach a balance of forces.

The proper consideration of residual load is critical to the design of pile and pile groups and to settlement predictions in particular. In order to better understand the development of residual load in drilled piles, this study continuously monitored the strain in an augered cast-in-place (ACIP) pile for a period of 103 days. Although the pile was not subjected to a top load, the results show that a significant residual load develops as conditions between the soil and pile progress toward equilibrium. The results of this study compare favorably to other published literature regarding the development of residual load in drilled and bored piles and driven piles.

Literature Review

Smith (1962) first reported that significant residual stresses can develop in driven piles. Norland (1963) recognized that erroneous interpretation of load test data could result from ignoring the presence of residual stresses. Briaud and Tucker (1984) proposed a method for evaluating the resistance distribution for driven piles in sand that explicitly considers residual stresses. Their discussion is limited to driven piles although the rebound during driving and reconsolidation of the soil after driving are identified as contributors to the development of residual stresses. Fellenius (1989) established the importance of the residual load to pile behavior and consequently developed the unified design of piles for the rational consideration of the interaction between residual load, applied sustained and transient loads, and settlement.

Fellenius (2001a) presented an analysis of instrumented bored piles that were dynamically tested and illustrated the presence of residual load. Fellenius (2001b) also analyzed strain gage data from statically loaded piles which showed that residual load develops in bored piles. On the basis of long term monitoring of driven piles, Fellenius (1998; 2006) concluded that essentially all piles will progress toward equilibrium where the sustained top load and the cumulative negative skin friction (*i.e.,* drag load) will act downward and be opposed by the positive shaft resistance and mobilized toe resistance. The location where the negative skin friction transitions to positive shaft resistance is the neutral plane.

According to Fellenius, even drilled and bored piles with no applied top load will develop a neutral plane. That is, the negative skin friction will be fully mobilized along the upper portion of the pile will be opposed by fully mobilized positive shaft resistance along the lower portion of the pile with a relatively small mobilized toe resistance. As a result, the entire pile will experience a compressive residual load where the load will be the greatest at the neutral plane.

Because the pile progresses toward a state of stress equilibrium with the surrounding soil over time, changes in stress and volume within the pile-soil system are believed to be primarily responsible for the development of residual load in bored piles. Fellenius (1989) identifies changes in effective stress during reconsolidation of the soil as a possible cause of residual load development. Hayes and Simmonds (2002) conclude that the physical expansion and contraction of the pile shaft concrete during curing are also responsible for the development of residual load.

Perhaps the most fundamental explanation is that the residual load develops as a result of the stiffness contrast between pile and soil (Fellenius, 2002a). Although the grout is initially fluid, the mature grout results in a stiffness ratio of roughly 400 to 10,000 (pile-to-grout) depending on the soil conditions. For the pile-soil system to reach a state of equilibrium, the soil resistances and internal pile loads must be in balance and the relative soil-pile movements must be compatible. As result, the full shaft resistance is mobilized [either upward (positive) or downward (negative)] and a portion of the toe resistance is mobilized. The residual load alone can mobilize a relatively small portion of the available toe resistance.

Others (Kim *et al.*, 2004; Vipulanandan *et al.*, 2007) have monitored the residual strain in ACIP piles, but they did not differentiate between strain resulting from grout set and curing and the strain resulting from shear at the soil-pile interface. It is solely the latter component of strain that corresponds to the development of residual load.

Strain-to-Load Interpretation

For this study, the residual load in an ACIP pile was interpreted from strain gage data. That is, the internal compressive load in the pile is of primary interest rather than strain itself. As a consequence, the strain data required transformation into load (Q) using the relationship $Q = EA\varepsilon$ where E is the Young's modulus of the pile, A is the cross-sectional area of the pile, and ε is the axial strain determined from strain gage readings.

While the equation is simple, its application to ACIP piles becomes quite complex upon close examination. Because the ACIP pile is cast-in-place. rather than pre-fabricated, the cross-sectional area is not necessarily uniform throughout its length. Also, the modulus of the pile varies with both time and stress. There are also internal stresses and associated strains associated with grout set and curing that require consideration in the interpretation of residual load.

In consideration of the variation of modulus with strain, a reasonable estimate of the pile modulus can be determined from the strain gages embedded near the top of the test pile, where the load in the pile accurately assumed to be the applied load. Otherwise, displacement measurements from tell tales can also be used to calculate strain. However, the installation and monitoring of multiple telltales is often so involved as to be considered impractical.

It is possible to determine a modulus from grout cube compressive strength tests, but in the experience of the authors, this is not representative of the actual pile modulus. The modulus is most often underestimated and the resulting resistance distribution is erroneous. Notably, an unrealistic high shaft resistance in the upper portion of the pile is characteristic of an interpretation using a low modulus.

Site Characterization

The site of the test pile is near the town of Rincon in southwest Georgia as shown in Figure 1. It is located in the Coastal Plain Geographical Province which is characterized by sedimentary deposits (Hunt, 1967). The near-surface conditions are characterized as interbedded sands, silts and clays of the Pleistocene age. Figure 2 graphically presents the results of a cone penetration test performed at the location of the test pile prior to its installation.

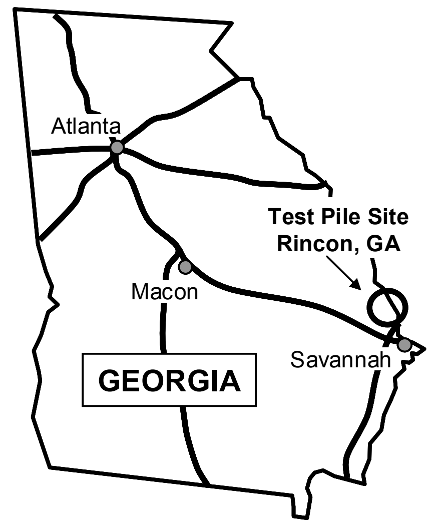

Figure 1. Test Pile Site

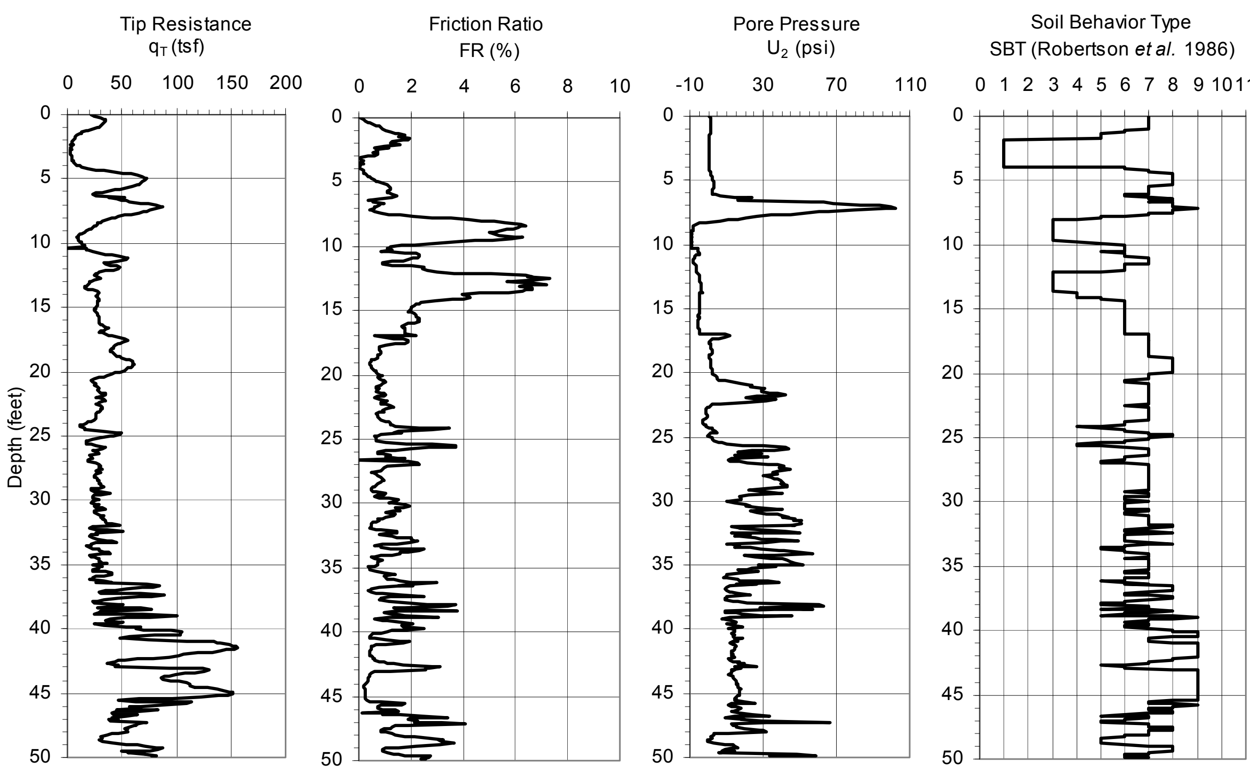

Figure 2. Cone Penetration Test Data at the Test Pile Location

According to the Soil Behavior Type (SBT), the upper 15 ft of soils behave as interbedded clays (SBT of 3), silty clays (SBT of 4 or 5) and sandy silt (SBT 6 or 7). Below 15 ft the soils primarily behave as sandy silts or silty sands. The corrected cone tip resistance (q_t) is typically below 50 tsf from the ground surface extending to a depth of 36 ft below the ground surface. Below a depth of 36 ft, q_t increases to between 50 and 100 tsf. Dissipation tests performed at the test location indicate that the static groundwater level is approximately 16 ft below the ground surface.

Test Pile and Strain Gage Data

The test pile was installed using conventional equipment. Typical ACIP pile installation involves rotating a continuous flight auger with a hollow stem into the ground to the planned pile toe depth. A fluid grout mixture of Portland cement, sand, fly ash and water is then pumped through the hollow stem. The grout exits via a hole at the tip of the auger and immediately begins filling any space between the auger and the adjacent ground. Once a satisfactory quantity of grout is pumped at the toe depth, then the auger is extracted in a controlled manner as grout continues to be pumped through the hole at the tip of the auger. Once the auger fully exits the ground, then grout pumping is terminated and the steel reinforcement is placed through the fluid grout.

The test pile was installed using a Berkel manufactured gear box, power unit, and grout pump. The gear box included a KYB 525 hydraulic motor. The power unit included a 3406 Caterpillar engine that is capable of delivering 3700 ft-lbs of torque. The grout (ball and seat style) pump was theoretically capable of delivering 1.05 cubic ft of grout per pump stroke and 90 cubic yards of grout per hour.

The test pile is an 18 inch nominal diameter ACIP pile installed to a depth of 32 ft. The design grout strength is 5000 psi, and the reinforcement in the test pile is a single No. 8 center bar. The grout factor for the test pile, which is calculated as the pumped grout volume divided by the theoretical pile volume x 100%, is 128%.

The strain gages were attached to the center bar at locations corresponding to the following depths from the ground surface: 2 ft, 6 ft, 10 ft, 14 ft, 18 ft, 22 ft, 26 ft, and 30 ft. Two Geokon Model 4911-4 Vibrating Wire Strain Gages were included at each depth for redundancy. Gage readings were recorded with a Geokon LC-2 16 Channel data logger.

Recording was initiated with the center bar resting on the ground after the gages had been attached to the bar with steel wire ties. Once the test pile location was drilled and fully pumped with grout, the center bar was hoisted off the ground together with the data logger. The center bar was then lowered into the fluid grout until the end of the bar reached the bottom of the hole. The gages were recorded every 30 seconds during hoisting and installation, then every 15 minutes for the first 24 hours, and subsequently at one hour intervals. The strain gage data, in terms of microstrain versus time, are graphically summarized in Figure 3.

Data Interpretation

Referencing Figure 3, the strains prior to the time of pile installation represent the gage conditions at the time they were connected to the data logger and the gages were resting on the ground surface. All of the gages experienced some changes during installation. The gage at 18 ft shows a permanent offset resulting from slippage of the wire ties which occurred as the center bar was lifted off of the ground. The data show that compressive load, as indicated by an upward slope in the strain versus time plot, increases as the grout begins to cure. This corresponds to a plot time of approximately 1 to 10 hours. After 10 hours, the trend reverses and the slope of the strain versus time plot is downward. This indicates that the compression load in the pile is decreasing. The decrease in compression load may be explained by the development of internal tensile stresses due to drying shrinkage during curing.

Drying shrinkage is the reduction in the concrete volume due to the removal of water; even the removal of intercrystalline water (Neville, 1996). Mindness and Young (1981) point out that the fundamental processes underlying drying shrinkage are not yet fully understood. On the basis of the current understanding, it is rational to expect that the tensile stresses due to drying shrinkage would result in 80 to 100 microstrain as observed in this study.

Hayes and Simmonds (2002) have observed the development of tension micro-fractures in bored piles (*i.e.,* drilled shafts) during curing due to cooling shrinkage. This supports the interpretation that the strain versus time curves

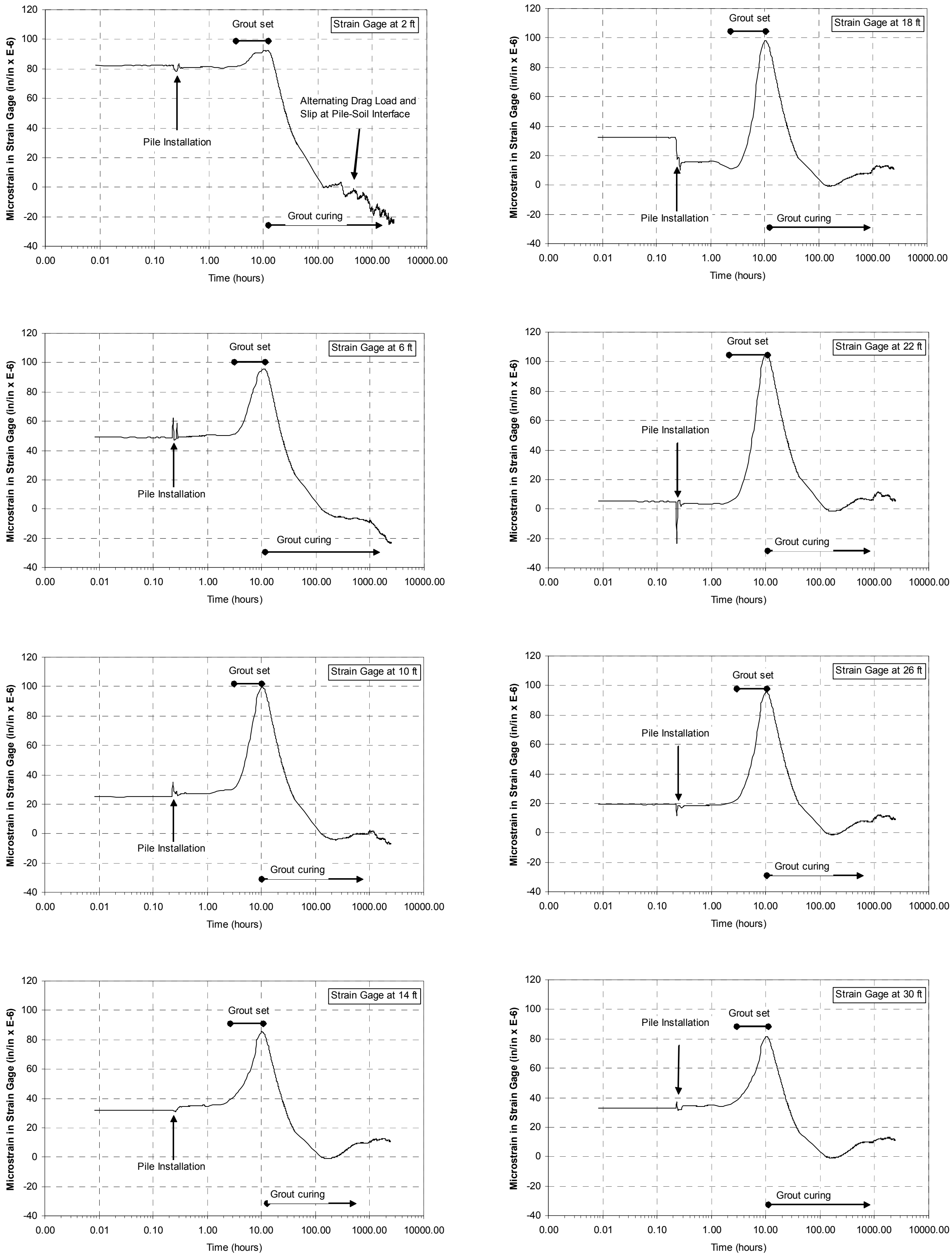

Figure 3. Strain Gage Data

reflects the development of internal tensile stresses due to shrinkage.

The data for the uppermost gage (2 ft) begins to cycle between increasing and decreasing compression load. Given that the shaft resistance at this depth is expected to be relatively small, it is believed that this cyclic behavior may be due to alternating periods of buildup of small residual stress and slip along the pile-soil interface. At approximately the same point in time, slopes of the different time versus strain curves begin to diverge. Thus, the beginning of this cyclic behavior (approximately 120 hours) should serve as a suitable reference for the onset of residual load development. Another assumption is that the uppermost gage will serve as a reference for zero external load. That is, the external load represented by the uppermost gage should be essentially zero even as the internal stresses change to maintain equilibrium. On the basis of these two assumptions, it is possible to estimate the residual load throughout the length of the pile using the relationship of strain, load, and modulus.

Because no additional loads were applied to the pile it was necessary to estimate the pile modulus as it varies with strain. Historical stress-strain data from high quality static load tests were used to estimate the secant moduli with varying levels of strain. These secant moduli are representative of a curing period of 6 to 10 days which corresponds well with the onset of residual load development. It is recognized that the modulus may experience a modest increase with time (Kim *et al.,* 2004); however, no adjustment for this potential for modulus increase was included in the data analysis.

Figure 4 presents the results of four instrumented load tests on ACIP piles and a fitted curve relating the secant modulus to microstrain. As illustrated by this figure, the secant modulus is non-linear (Fellenius, 2001), particularly at small strain levels, and significantly higher than typically estimated based on unconfined compressive strength data. Figure 4 presents the residual load distribution that is interpreted from strain gage data for 7, 30, 58, and 103 days after pile installation. The internal compressive load for the test pile was estimated based on the principles of the unified pile design using the LCPC method (Bustamante and Gianeeselli, 1982) and the site-specific cone penetration test data.

The interpreted residual load, as represented in Figure 5, was relatively small (roughly 6 tons) 7 days after installation. The residual load steadily increased with time to approximately 58 days after installation. Between 58 and 103 days after installation, the residual load experienced only a modest increase. The ultimate residual load of 55 kips was interpreted from the strain gage at a depth of 18 ft below the top of the pile.

The distribution of residual load for 103 days after installation compares favorably to the compressive load distribution anticipated based on the principles of the unified design of piles.

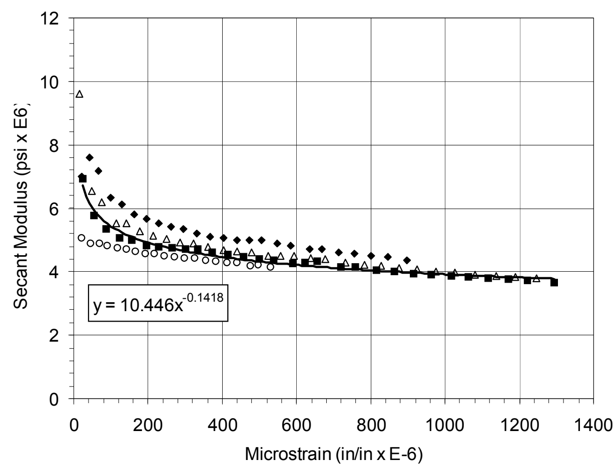

Figure 4. Secant Modulus Variation with Strain

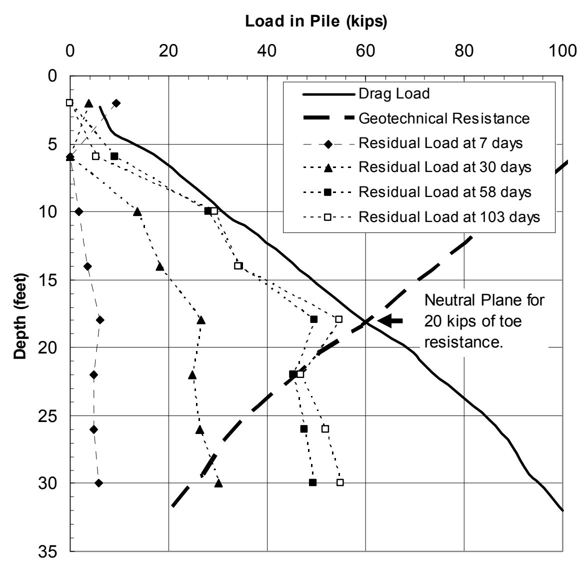

Figure 5. Residual Load Distribution Interpreted from Strain Gages

As shown in Figure 5, the residual load distribution closely matches the expected compression load in the pile due to the development of negative skin friction above the neutral plane and the opposing resistance provided by positive shaft resistance below the neutral plane and the mobilized toe resistance.

It was originally anticipated that the residual load would continue to decrease below 26 ft but this trend was not evident from the strain gage data. As will be discussed in the following section, the general shape of the residual load distribution curve from this study compares favorably to curves presented by others either inferred from load test results or long term pile monitoring. It may be that these results should have been expected for a bored or drilled pile in sand.

It may be pertinent to note that a surface load was recorded by the strain gages 40 days after installation. Review of the construction records showed that a crane was parked very near the test pile and later disassembled. It is possible that the weight of the crane performed as a sustained load on the pile and led to a higher residual load in the lower portion of the pile than expected based on the principles of the unified design of piles.

Discussion

There are two practical issues concerning the development of residual load in bored and drilled piles. The first is that consideration of residual load should be made when estimating the distribution of the geotechnical resistance from strain gage data collected during load tests. This study concludes that the residual load began to develop approximately 5 days after installation and steadily increased to 58 days after installation. For the purposes of this study, the ultimate residual load (P_{ult}) is presumed to be the maximum residual load interpreted from the strain gage data. This is believed to be rational considering that that there was very little difference between the strain gage readings taken after 58 days and taken after 103 days. The normalized distribution of residual load (P/P_{ult}) from this study along with distributions from other studies (both back-calculated and measured) at sandy sites is shown in Figure 6 where the pile length is L and normalized depth along the pile is d/L. Other studies (Fellenius, 2001) indicate that, in clay, a relatively small toe resistance contributes to the residual load but the shaft resistance is fully mobilized.

A second consideration is that the development of the residual load implies that drilled and bored piles (just as driven piles) experience negative skin friction. According to the principles of the unified design of piles (Fellenius, 1989), the combination of the negative skin friction and the sustained top load are balanced by the positive shaft resistance plus the mobilized toe resistance. A transient load that is smaller than the cumulative negative skin friction only serves to temporarily reverse the direction of the shaft resistance in the upper pile. Thus, the transient load does not influence the movement of the pile toe.

In light of the latter consideration, it may be concluded that the residual load contributes to the compressive load within the pile and should be included when evaluating the structural safety of the pile section. Also, single pile and pile group settlement should be computed using the true distribution of geotechnical resistance.

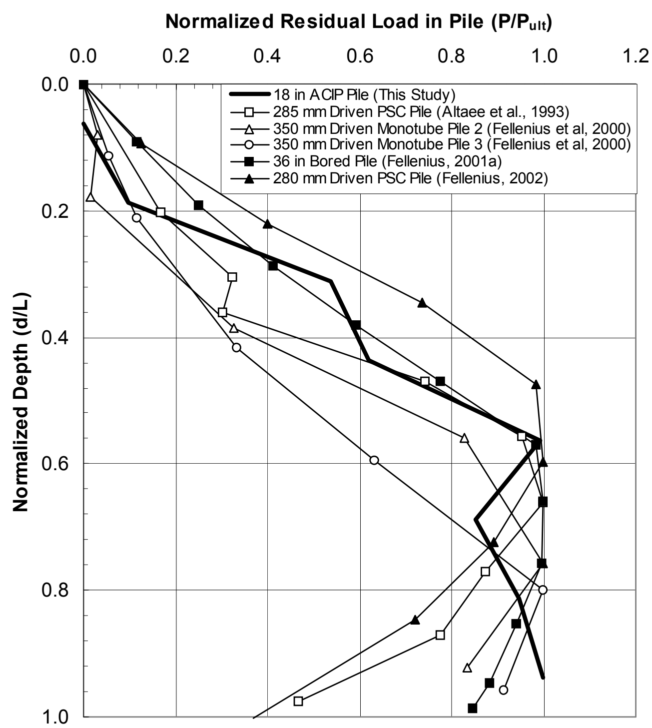

Figure 6. Normalized Distribution of Residual Load

Conclusions

This study continuously monitored strain gages in an ACIP pile for 103 days. The purposes of this study were to observe the changes in pile strain under zero top load and to then interpret the corresponding development of residual load.

The results of this study confirm that significant changes in strain occur during set and curing of the grout and the redistribution of stresses in of the surrounding soil. The interpretation made herein is that the residual load begins to develop approximately 5 days after installation. The residual load distribution approaches the magnitude and distribution expected by the unified design of piles approximately 58 days after installation. A small increase in residual load occurred between 58 days and 103 days after installation.

The presence of residual load as observed in this study has ramifications to several aspects of the design of drilled and bored piles:

1. The strain gage reading immediately prior to the application of an external top load (often called the "no load" reading) may or may not include a substantial residual load in drilled and bored piles depending on the time period between pile installation and the initial reading of the strain gage. This study on an 18 inch ACIP pile indicates that the residual load is in the range of 6 kips approximately 6 days after installation. The residual steadily increased for 58 days at which time the full shaft resistance was mobilized (either as negative skin friction or positive shaft resistance). Between 58 and 103 days after pile installation, only a small increase in residual load was observed.

2. The stress-strain conditions in drilled and bored piles are complex and careful interpretation of strain gage data is critical to accurately understanding pile behavior. The initial reading of the gage as it is received from the manufacturer or while the gage is resting flat on the ground may be considered the true zero load reading. However, the strain gages will be exposed to compressive stress during grout set and to subsequent decreases in compressive stress (and possibly tensile stress) during grout curing. These changes in internal stress can take place concurrent with the development of residual load. Both internally generated stresses and residual load may be present immediately prior to load testing.

3. The development of residual load as interpreted herein supports that the soil-pile stress conditions for drilled and bored piles will be consistent with the unified design of piles (Fellenius, 1989).

4. The residual load developed in drilled and bored piles may be significant and should be included when computing the structural safety of the pile section.

5. The development of residual load is pertinent to meaningful estimations of the single pile and pile group settlement. That is, the load-deflection response from a conventional load test is appropriate for establishing a capacity or limit state for the pile. However, the pile settlement should be considered by establishing the geotechnical resistance distribution (including any residual load) according to the unified design of piles. It is anticipated that the importance of an accurate representation of the geotechnical resistance distribution and the load distribution in the pile may be even greater for pile rafts and mats with piles as settlement reducers.

Concluding Remarks

The authors recognize that the development of residual load in piles has commonly been associated with either pile driving or consolidation induced by fill placement. Considering this, the residual load in drilled and bored piles is rarely explicitly estimated or considered significant.

It can be theorized that the reason that the residual load is rarely explicitly estimated for the design of drilled and bored piles is that there is no immediately intuitive mechanism (absent of substantial ground settlement) for its development. Also, data on long term pile behavior has been lacking, most likely due to the challenges inherent in continuously monitoring strain gages for a long time period and with the interpretation of such data. A study by Kim *et al*. (2004) monitored residual strain in ACIP piles in interbedded sand and clay, but did not attempt to differentiate between strain resulting from grout setting and curing and strain resulting from shear at the soil-pile interface. Neither did a study of ACIP pile in dense sand performed by Vipulanandan *et al*. (2007); however, a review of the published data suggests that the distribution in residual load, if adjusted by assuming zero residual load at the location of the top strain gage (near the ground surface), is consistent with the conclusions of this study. Falconio and Mandolini (2003) review data from strain monitoring of bored piles and conclude that significant residual strain develops that should influence the piles' response during loading.

The proposition that pile groups under buildings behave differently than single piles (presumably characterized by load tests) was first put forth by Terzaghi and Peck (1948). Peck *et al.* (1953) computed settlements of friction piles in clay using an “equivalent pier foundation” located at the lower third of the pile length rather than using a derivative of the response of a single pile during load testing. Meyerhoff (1976) concluded that the settlement of pile groups in both sand and clay could be estimated using the equivalent pier foundation method.

While it is established that the shear resistance developed during pile load testing is confined to a thin zone around the pile (Burland, 1973; Meyerhoff, 1976), this conclusion is not necessarily applicable to the long term conditions. That is, the load-deflection behavior of a single pile that is loaded relatively quickly to its ultimate shaft resistance (such as a test pile) has been observed to be controlled by the shear response of the soil that is immediately adjacent to the pile. However, this does not necessarily indicate that the observed shear response is representative of a soil-pile system that is in equilibrium. On the contrary, it may be expected that, under any long term top load, that the pile and soil will undergo a cycle of soil deformation, stress reorientation, and changes in residual load until the pile and nearby soil are in a state of equilibrium. The authors take the position that residual load will develop in all piles as a result of this process that progresses toward equilibrium.

The predominance of evidence for residual load in driven piles, as opposed to drilled and bored piles, appears to be related to the efficacy for measuring the residual load rather than its presence or absence. Admittedly, the presence of internal stresses and corresponding strains due to grout set and curing make interpretation of residual load less direct and more difficult for drilled and bored piles than for driven piles.

The authors have attempted to present a persuasive case for acknowledging the development of residual load in drilled and bored piles. As referenced herein, Fellenius has presented several back-analyses that illustrate the effects of residual load on the interpretation of strain measurements collected during load testing of drilled and bored piles. Hayes (personal communication, 2009) has observed that residual load is routinely present and its effects adjusted for during Osterberg Load Cell testing of drilled and bored piles. Finally, the results of this study show that residual load developed in an ACIP pile and that the corresponding geotechnical resistance distribution closely matches the expectations of the unified design of piles (Fellenius, 1989).

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