This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

DEEP FOUNDATIONS INSTITUTE
DFI

Vol. 4, No. 2 December 2010

DFI JOURNAL

The Journal of the Deep Foundations Institute

PAPERS:

TECHNICAL NOTE

Deep Foundations Institute is the Industry Association of Individuals and Organizations Dedicated to Quality and Economy in the Design and Construction of Deep Foundations.

This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

TECHNICAL NOTE

Load Testing and Interpretation of Instrumented Augered Cast-in-Place Piles

Timothy C. Siegel, P.E., D.GE, Principal Engineer, Dan Brown and Associates, Knoxville, TN, USA; tsiegel@danbrownandassociates.com

ABSTRACT

Given that piles are composed of different materials and installed using a variety of methods, *ASTM D 1143/D 1143M-07 Standard Test Methods for Deep Foundations Under Static Axial Compression* is justifiably general so that it has as many applications as reasonably possible. There are aspects of the test setup, the test procedures, and data interpretation for instrumented piles (some of which are specific to augered cast-in-place piles) that are either not discussed in detail in this ASTM standard or are not addressed at all. Specifically, the use of variable time hold times can obscure the shape of the conventional load-deflection curve, as well as, influence the interpretation of the ultimate pile capacity. Unload-reload cycles induce additional non-uniform internal axial loads within the pile which complicate interpretation of the strain gage data and increase the potential for errors. Attempts to maintain a constant top load during the hold period can lead to operational difficulties in taking measurements and are technically unnecessary. Residual load has been shown to develop in cast-in-place piles and can significantly influence the axial load distribution as interpreted from strain gage data. The stiffness of cast-in-place piles has been observed to vary with the measured strain, and a constant modulus, as provided by correlation with the unconfined compressive strength of the grout, may lead to a significantly different interpretation of the axial load distribution.

INTRODUCTION

Test procedures for conventional top-loaded axial compression testing of a single pile are presented in *ASTM D 1143/D 1143M-07 Standard Test Methods for Deep Foundations under Static Axial Compression* (ASTM International, 2007). Given that piles are composed of different materials and installed using a variety of methods, the referenced ASTM standard is general so that it is applicable to as many situations as reasonably possible. It is understandable that there are aspects of the test setup, the test procedures, and data interpretation for instrumented piles (some of which are specific to augered cast-in-place piles or ACIP piles) which are either not addressed in detail in the ASTM standard or are not discussed at all. The purpose of load testing is not limited to verification of the preliminary design, but is also to collect site-specific information on the pile geotechnical resistance on which to justify optimization of the pile length. Particularly in the latter case, it is useful to separate the resistance to the total applied top load into the two components of (1) the shaft resistance and (2) the toe resistance. An approximation of the shaft and toe resistances can be accomplished using a graphical construction (Bloomquist et al. 2007), but they can be more directly computed by the collection and subsequent interpretation of strain gage data along the length of the pile. While a correct interpretation of the load-deflection response and the strain data can lead to a greater understanding of the pile behavior, misunderstandings can lead to confusion and even a misplaced loss of confidence in ACIP piles. Such misunderstandings do occur, due in part, to the higher level of complexity involved with load testing ACIP piles. Given that the foundation design depends on accurate data and interpretation, it becomes imperative that each step in the instrumentation setup, data collection, and interpretation be executed in a rational manner that is consistent with the physics of pile and soil behavior.

PILE-SOIL MODEL

A representative conceptual pile-soil model is necessary for the successful planning, execution and interpretation of load tests on ACIP piles. In that regard, it is understood that the internal pile strain (*i.e.,* changes in the strain gage reading from the factory) is a result of the following four mechanisms: (1) disturbance due to shipping, handling, or installation, (2) internal stresses due to grout set and curing; (3) residual load as the pile-soil

This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

system progresses toward stress equilibrium and strain compatibility even in the absence of a top load, and (4) transfer of the applied top load to the soil surrounding the pile (as either shaft resistance or toe resistance).

When a top load is applied, the pile undergoes compression that begins at the top and progresses to the depth where the entire applied top load is fully distributed into the soil. The pile also shortens under the compressive load. The pile tends to move downward relative to the surrounding soil and this mobilizes positive shaft resistance (*i.e.,* the soil tends to force the pile upward). Depending on the magnitude of the movement of the pile toe, the soil immediately beneath the pile may also push upward on the pile. If the top load were removed, then the pile will tend to expand upward and the direction of the shaft resistance will reverse along a portion of the upper pile. That is, pile expansion will be resisted by negative shaft friction (also called negative skin friction or negative shaft resistance). A net effect of such an unload-reload cycle is the development of additional internal compressive loads. There was a time when the total downward pile movement minus the upward pile movement upon unloading (known as rebound) was believed to be a meaningful representation of the pile settlement. This has been shown to be erroneous given the better understanding of pile behavior and specifically the development of shaft resistance.

The data from a load test is most often used to estimate the ultimate geotechnical resistance of pile and can be used to estimate the distribution of the shaft and toe resistances. The design load may be compared to the interpreted geotechnical resistances to determine the factor-of-safety against geotechnical pile failure for the controlling load case. The duration of the applied top load is essentially always too short to conclude that the distribution of the geotechnical resistances represents a state of equilibrium. The single pile or pile group settlement should be computed using a method consistent with the unified design of piled foundations (Felllenius, 2004) which explicitly considers the relationship between pile load, soil compressibility, residual load and settlement. Similar to the accepted design methodology for shallow foundations, pile foundations should be designed based on the capacity often confirmed by load tests and the settlement based on field and laboratory data and an appropriate analytical model (Meyerhoff, 1976; Fellenius, 2004).

PILE INSTRUMENTATION

Strain Gages. The writer is most familiar with the vibrating wire type strain gage where the measurement is the frequency of vibration of a tensioned steel wire whose frequency of vibration is proportional to the mass and length of, and strain in, the vibrating wire. The gage is attached to (or embedded inside) a section of reinforcing steel at the manufacturer's laboratory and the entire assembly is embedded in the grout during installation. The following suggestions are made regarding the installation of the strain gages: Record the strain gage serial numbers and confirm that the ends of the wires are correctly marked with the serial number.

1. Use two gages at each location for redundancy. It has been suggested that the two gages at each location be placed at points opposite one another within the pile cross-section to allow adjustments for eccentricity. The drawback is that if one of the gages fails prior to or during testing, then the data from a single gage that is offset significantly from the pile center may result in greater error. There may also be limitations in the placement of gages associated with the reinforcing steel in the test pile.
2. Plan the strain gage wire lengths well in advance of the pile installation keeping in mind that it is much easier to cut (shorten) a long cable than it is to successfully splice a short cable. Wires that are too short are problematic while wires that are very long become unwieldy during placement of the reinforcing steel. Cable splices should be avoided wherever possible.
3. The sister bar should be securely fastened to the center reinforcing bar either with wire ties or welding so that its position does not slip during installation. The strain gage wires should be secured firmly along the bar while avoiding pinching. The strain gage wires may be bundled near the top of the center bar but there needs to be some exposed center bar (or an extension to the center bar) to allow for placing the lifting strap during installation. The writer's experience is that the center bar should be placed in the fluid grout first and then the shorter reinforcing cage is placed so that the center bar is threaded through the

This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

middle of the cage. The placement of the center bar separately from the reinforcing cage appears to reduce the potential for uncontrolled bending of the sister bars and stretching of the wires. The combined weights of the center bar and reinforcing cage have been observed to increase the potential bending during lifting and subsequently to increase the potential for stretching of the wires as well.

4. ***Telltales.*** A telltale can be used to directly measure pile movements at certain points along its length. While a telltale (or multiple telltales) can be used at any location along the pile length, a telltale at the pile toe can be used to collect information on which to evaluate the relationship between mobilized toe resistance (established using the strain gage data) and toe movement.

To allow the use of a telltale, a small diameter (12 to 19 mm or 0.5 to 0.75 in) metal pipe is typically attached to the center bar to be embedded in the grout. A permanent cap should be on the bottom end of the pipe and a removable cap should be on the top end. The tell tale rod should be very stiff with negligible bend. A butt welded reinforcing steel bar with a diameter slightly smaller than the pipe inner diameter can be very effective. The steel bar should make a right angle where it exits the top of the pile. Ideally, the connection at the right angle should be welded rather than created by a bend. Bends are often more flexible which is undesirable. The telltale should extend beyond the perimeter of the pile.

PILE LOADING

Experience has shown that loading the pile in equal time intervals in load increments consistent with the quick test generally provides a smooth load-deflection curve (Fellenius, 1980). Varying the time interval is undesirable and unnecessary. It may be initially perceived that a longer time interval would provide an estimation of pile settlement, but the settlement is more appropriately considered by an analytical procedure that more accurately represents the pile under design conditions. The time interval for each load increment should allow sufficient time to make at least two sets of readings for all of the dial gages, strain gages, jack pressure, and load cell. Automated load testing devices that allow nearly continuous readings are even better. Make sure that the batteries in all readout boxes and dataloggers are fully charged prior to testing. In some cases, battery failure in a device can result in loss of data. Fully charged spare or replacement batteries should be kept on hand.

All readings of pile movement should be collected directly from the pile. Experience indicates that pile movements that reference plates, the jack, or surfaces other than the pile itself can incur error in the readings. An example is where the plate on the top of the pile exhibits some warping either prior to or during loading. During loading, the plate can experience measurable bend due to either flattening or warping even further.

The target load should initially be applied and then there will likely be some observed reduction in the applied load as the pile moves downward. The load will be transferred downward along the length of the pile and distributed as the soil resistance is mobilized. The pile-soil system will progress in time toward stress equilibrium and strain compatibility. Considering that two months or more may be required to reach a balance between stress and strain (Siegel and McGillivray, 2009), it is not practical to wait until the pile-soil system is in equilibrium within the duration of the load test.

Attempts to maintain a "constant" top load should be avoided. A practical reason is that the readings from the dial gages, strain gages and telltales will continue to change with each attempt to re-establish the target load. A technical reason is that once the pile moves approximately 2.5 to 5 mm (0.1 to 0.2 inches) then the shaft resistance along the upper portion of pile will begin to decrease after its peak value. Each attempt to re-establish the target load will result in further decrease in shaft resistance in the upper pile due to remolding at the shaft-soil interface which may be offset by the mobilization of additional shaft resistance in the lower pile and/or toe resistance. The best practice is to record the applied load that exists at the end of the time interval.

The pile should be incrementally loaded until one of the following occurs: (1) the applied load equals the limit of the test system, (2) the pile experiences structural failure, or (3) the pile is no longer able to resist the target load – *i.e.,* geotechnical failure. Cycling of the load should be avoided as it induces an unrecoverable internal compressive load that varies non-uniformly through the pile length. During unloading, the internal compressive load develops as lower portion of pile tends to force the pile upward while negative skin friction develops along the upper portion of pile. The distribution and direction of the shaft resistance will vary with depth which will lead to difficulties in the interpretation of the strain gage and telltale data.

Minimal significance should be applied to the unload portion of the load test. During and after removal of the applied top load, the distribution of the internal compressive load is unknown. It is also certain that the pile movement and the stress conditions will continue to change with time. As discussed earlier, the actual pile settlement should not be determined from the load-deflection behavior during load testing.

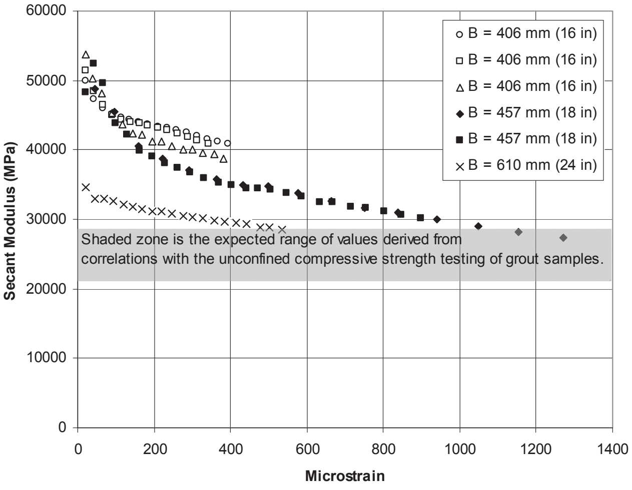
If the pile appears to have experienced a structural failure, then observations during unload can help as a confirmation. When a pile structurally fails then the readings in the lower strain gages can become inconsistent and the telltale acts as if it is wedged in the pile. During reloading, a structural damaged pile may resist a load that corresponds to the intact portion of the pile.

DATA INTERPRETATION

The two primary parts of the data interpretation are (1) selection of the ultimate pile resistance, and (2) determining the distribution of the shaft resistance. There are an abundance of proposed methods to interpret the ultimate pile resistance. Appropriate methods are calibrated for the typical ACIP pile conditions and identify the limit state or the ultimate pile resistance. The ultimate pile resistance should not make any consideration for the pile movement under design loads. Such is not the purpose of the load test nor do the conditions of the load test sufficiently represent the behavior of the pile under the actual conditions. Ideally, the ultimate pile resistance will be defined by the shape of the load-deflection curve and specifically the point at which the slope of the curve becomes very steep. One such failure interpretation is the Brinch Hansen 90% criterion (Brinch Hansen, 1963) that defines failure where one-half of the top deflection at the failure load occurs as a result of the application of 90% of the failure load.

Determination of the distribution of the shaft resistance using the strain gage data has been described by Fellenius (2001). In the initial step, the top load is divided by the product of the cross-sectional area and the measured strain (actually the difference between the zero load strain and the strain under the top load) near the top of the pile to calculate the secant or tangent moduli. It is also feasible to determine a direct correlation between strain and load. In any case, the uppermost strain gage should be sufficiently close to the ground surface so that the assumption of negligible shaft resistance is valid. The modulus of ACIP piles varies with strain and time (including curing conditions). Typically, the effect of time is ignored and the stress-strain response of the uppermost gage(s) is used to define the moduli in terms of strain only. Data from several projects were used to prepare Fig. 1 which illustrates the variation of secant modulus with strain where the nominal pile diameter (B) was between 406 and 610 mm (16 to 24 inches).

As shown in Fig. 1, the secant modulus decreases with strain. There appears to be some variation in the strain-modulus relationship for different load tests which emphasizes the importance for site-specific determination of the modulus for strain gage interpretation. For reference, the shaded zone illustrates the expected range of values that may be derived from correlations with the unconfined compressive strength testing of grout samples. This suggests that the use of such correlations will tend to under predict the modulus and lead to a significantly different interpretation. The writer's experience confirms the interpretation based on such correlations tends to over predict the shaft resistance in the upper portion of the pile and under predict the resistance in the lower portion of the pile.



Shaded zone is the expected range of values derived from correlations with the unconfined compressive strength testing of grout samples.

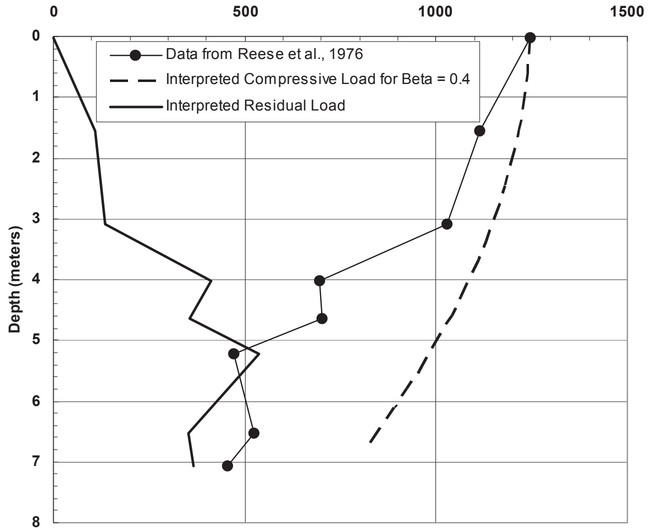
[FIG. 1] Variation in Secant Moduli with Strain for ACIP Piles

Using the strain dependent modulus, the internal compressive load is computed for each strain gage location. The difference in the internal compressive load between strain gage locations is the mobilized geotechnical resistance plus any residual load. Long term monitoring of ACIP piles indicates that the residual load developed within the typical wait time for load testing (approximately 7 to 10 days) is relatively small but that it can become significant afterwards (Siegel and McGillivray, 2009). If there is an extending wait period between pile installation and load testing, then it will become necessary to correct the load distribution for the effects of residual load development. As there is no established analytical method for determining the development of residual load over time, the writer proposes interpolation based on the results of long term observation. The limited available data indicates that the residual load begins developing approximately 5 days after pile installation and reaches near equilibrium approximately 58 days after installation. The equilibrium stress conditions are defined where the negative skin friction developed along the upper portion of pile will be resisted by the shaft resistance in the lower portion of pile and the mobilized toe resistance in accordance with the unified design of piled foundations.

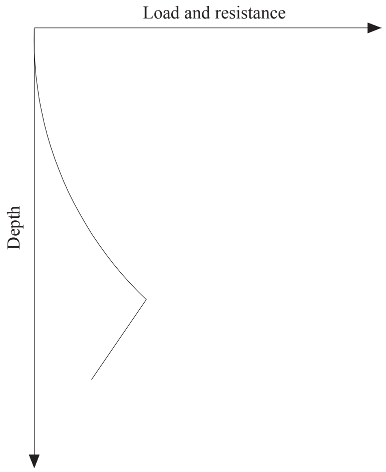
EXAMPLES

Effect of Residual Load. The writer considers the interpreted load distribution of a 760 mm (30 inch) diameter bored pile as presented by Reese et al. (1976) as an example of the effect of residual load on the interpretation of data from an instrumented cast-in-place pile (Fellenius and Altaee, 1996). This pile was installed in high plasticity clay and low plasticity silt where the water table was approximately 4.6 m (15 ft) below the ground surface. The strain data was collected using embedded Mustran strain cells and converted to axial compressive load. The following three quantities are presented in Fig. 2 versus depth: (1) the internal compressive load as originally interpreted by Reese et al. (1976) from instrumentation with the assumption of no residual load; (2) the internal compressive load computed using the β method and a constant β value of 0.4. and; (3) the interpreted residual load. The interpreted residual load is the difference between (1) and (2).

For comparison, Fig. 3 shows the typical shape of the residual load distribution proposed in several studies (Hanna and Tan, 1973; Fellenius and Altaee, 1995; Siegel and McGillivray, 2009). The writer believes there is sufficient similarity between the typical shape of residual load distribution (Fig. 3) and the interpreted residual load (Fig 2) to conclude that significant error was introduced by ignoring the presence of residual load.

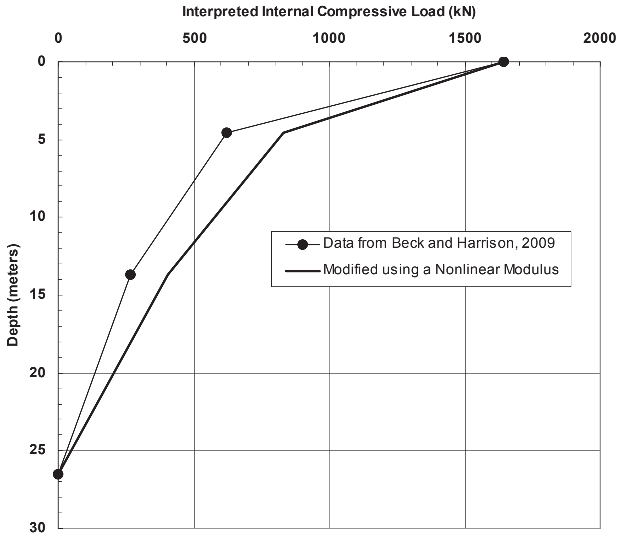


[FIG. 2] Interpreted Internal Compressive Load in Pile from Reese et al. (1978)



[FIG. 3] Typical Shape of Residual Load Distribution

Effect of Modulus. The writer considers the interpreted load distribution of a 356 mm (14 inch) diameter ACIP pile as presented by Beck and Harrison (2009) as an example of the effect of the modulus selection in the interpretation of data from an instrumented cast-in-place pile. The 26.5 m (87 ft) long test ACIP pile was installed into (from the ground surface): (1) approximately 5 m (16 ft) of existing fill, (2) 10 m (33 ft) of soft to medium stiff clay, and (3) loose to medium dense sand. Fig 4 compares the reported load distribution (based on an interpretation using a constant modulus) and a modified load distribution interpretations based on the non-linear modulus shown in Fig 1. It was assumed, for comparison purposes, that a constant modulus of 24.1 MPa (3.5 x 106 psi) was used by Beck and Harrison considering that the modulus was assigned based on the results of unconfined compression tests (Beck, 2009). As illustrated in Fig 4, the non-linear modulus results in the interpretation of higher internal compressive loads at smaller strains and the distribution shows a significantly greater portion of the top load being transfer to the clay and sand below the upper fill during load testing.



[FIG. 4] Interpreted Internal Compressive Load Distribution in Pile from Beck and Harrison (2009)

CONCLUSIONS

Considering that the procedures in *ASTM D 1143/D 1143M-07 Standard Test Methods for Deep Foundations Under Static Axial Compression* are general so that it may be applied to a wide range of pile types, there are aspects of the test setup, the test procedures, and data interpretation for instrumented piles (some of which are specific to ACIP piles) that are either not discussed in detail in this ASTM standard or are not addressed at all. The following conclusions highlight several aspects of load testing not explicitly addressed in the referenced ASTM standard with an emphasis on their application to ACIP piles.

1. The use of variable time hold times can obscure the shape of the conventional load-deflection curve, as well as, influence the interpretation of the ultimate pile capacity. It is strongly preferred to maintain a constant hold time for each load increment. Unless the pile is experiencing a plunging failure, the dial gages typically stabilize during the hold time; however, some movement of the jack ram and decrease in the applied test load are expected. Attempts to maintain a "constant" top load are unnecessary and should be avoided for practical and technical reasons.
2. Unload-reload cycles should be avoided as it induces an unrecoverable internal compressive load that varies non-uniformly through the pile length. During unloading, the internal compressive load develops as lower portion of pile tends to force the pile upward while negative skin friction develops along the upper portion of pile. The distribution and direction of the shaft resistance will vary with depth which will lead to difficulties in the interpretation of the strain gage and telltale data.
3. Residual load has been shown to develop in cast-in-place piles and can significantly influence the axial load distribution as interpreted from strain gage data. Long term monitoring of ACIP piles indicates that the residual load developed within the typical wait time for load testing (approximately 7 to 10 days) is relatively small but that it can become significant afterwards (Siegel and McGillivray, 2009). If there is an extending wait period between pile installation and load testing, then it will become necessary to correct the load distribution for the effects of residual load development.
4. The stiffness of cast-in-place piles has been observed to vary with the measured strain, and use of a constant modulus, as provided by correlation with the unconfined compressive strength of the grout, may lead to a significantly different interpretation of the axial load distribution.

REFERENCES

1. ASTM International (2007). ASTM D1143/D1143M-07 Standard Test Methods for Deep Foundations under Static Axial Compressive Load, West Conshohocken, PA, 15 pp.
2. Beck, W.K. and Harrison, P.J. (2009) Load tests on small diameter augered cast-in-place piles through fill ASCE, *Contemporary Topics in Deep Foundations*, GSP No. 185,

This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

Edited by M. Islander, D.F. Laefer and M.H. Hussein, Orlando, Florida, 430-437.

3. Beck, W.K. (2009) Personal communication.
4. Bloomquist, D., McVay, M., and Hu, Z. (2007) Final Report – Updating Florida Department of Transportation's (FDOT) Pile/Shaft Design Procedures Based on CPT & DTP Data, Department of Civil and Coastal Engineering, University of Florida, Gainesville, Florida, 199 pp.
5. Brinch Hansen, J. (1963) Discussion, Hyperbolic stress-strain response. Cohesive soils, ASCE, *Journal of the Soil Mechanics and Foundations Division*, 89(SM4), 241-242.
6. Fellenius, B.H. and Altaee, A.A. (1995) Critical depth: how it came into being and why it does not exist, *Proceedings, Institution of Civil Engineers*, Geotechnical Engineering, 113(4), 107-111.
7. Fellenius, B.H. and Altaee, A.A. (1996) Discussion Critical depth: how it came into being and why it does not exist, *Proceedings, Institution of Civil Engineers, Geotechnical Engineering*, 119, 244-245.
8. Fellenius, B.H. (1980) The analysis of results from routine pile load tests, *Ground Engineering*, 13(6), 19-31.
9. Fellenius, B.H. (2001) From strain measurements to load in an instrumented pile, *Geotechnical News Magazine*, 19(1), 35-38.
10. Fellenius, B.H. (2004) Unified design of piled foundations with emphasis on settlement, ASCE, *Current Practice and Future Trends in Deep Foundations*, GSP No. 125, Edited by J.A. DiMaggio and M.H. Hussein, Los Angeles, California, 253-275.
11. Hanna, T.H. and Tan, R.H.S. (1973) The behavior of long piles under compressive loads in sand, *Canadian Geotechnical Journal*, 10(3), 311-340.
12. Meyerhoff, G.G. (1976) Bearing capacity and settlement of pile foundations, ASCE, *Journal of the Geotechnical Engineering Division*, 102(3), 197-228.
13. Reese, L.C., Touma, F.T., and O'Neill, M.W. (1976) Behavior of drilled piers under axial loading, ASCE, *Journal of the Geotechnical Engineering Division*, 102(5), 493-510.
14. Siegel, T.C. and McGillivray, A. (2009) Interpreted residual load in an augered cast-in-place pile, *Proceedings, 34th Annual Conference on Deep Foundations*, Deep Foundations Institute, 173-182.

This paper was originally published in DFI's bi-annual journal, Volume 4, No. 2 in December 2010. DFI is an international technical association of firms and individuals involved in the deep foundations and related industry. The DFI Journal is a member publication. To join DFI and receive the journal, go to www.dfi.org for further information

DFI JOURNAL

The Journal of the Deep Foundations Institute

International Standard Serial Number (ISSN): 1937-5247

Deep Foundations Institute
326 Lafayette Avenue
Hawthorne, New Jersey 07506 USA
Tel: 973-423-4030
Fax: 973-423-4031
www.dfi.org