371. Fellenius, B.H. 2017. Best practice for performing static loading tests. Examples of test results with relevance to design. 3rd Bolivian International Conference on Deep Foundations, Santa Cruz de la Sierra, Bolivia, April 27-29, Vol. 1, pp. 63-73.

Best practice for performing static loading tests. Examples of test results with relevance to design

Fellenius, Bengt H.[1]
[1] Consulting Engineer, Sidney, BC, Canada. <bengt@fellenius.net>

ABSTRACT. When determining "capacity" from the result of a static loading test, the profession has no common definition of "capacity". Thus, a group of professionals will come up with an array of capacity values from the same test results. To improve reliability, piles are usually instrumented with strain-gages at selected depths. The analysis of the strain records depend on the axial secant stiffness of the pile, which is best determined applying the direct secant modulus method to the records from a gage level near the pile head and the indirect method, the incremental stiffness method, to records of gage levels down the pile. Examples demonstrate the necessity that a test be carried out with no unloading-reloading events included and that all increments be equal and the load levels be held constant for an equal duration not shorter than 10 minutes. If so, the analysis of the records will produce the load distribution and the pile toe load-movement response. When combined with the calculated soil settlement around the piles, the designer will be able to determine the settlement of the piled foundation. The approach is far more reliable and appropriate for a piled foundation design than one based on some perceived value of "capacity" with a settlement estimate just referenced directly to the load-movement of the test.

1. INTRODUCTION

In countries with a stagnant and code-driven piled foundation system, loading tests are rarely performed. In contrast, in countries with an advanced and adaptable foundation industry, testing is a fundamental part of the engineering process. Sometimes, the tests are performed as a part of a design effort, sometimes they are performed for proof testing during or after construction. Conventionally the tests are performed in a head-down approach and include no instrumentation down the pile. Testing an uninstrumented pile has little value for design, however. Moreover, including unloading-reloading steps in a test or uneven length of load-holding will adversely affect the strain records. The pile axial stiffness, EA, can be determined from the measured strains by applying direct secant and incremental stiffness methods, as is illustrated in this paper. However, determining the axial loads imposed by the test can be difficult even when the test is properly planned and executed.

2. CAPACITY

The dominant approach to assessing the results of a static loading test is to determine a pile capacity from the pile-head load-movement curve. The term "capacity" implies an ultimate resistance and is, in its purest form, defined as a continued movement for no increase of load once reached, i.e., plastic response after an initial "elastic". Figure 1 shows the test results of a 14 m long 400 mm diameter bored pile equipped with a telltale to the pile toe: the pile-head load plotted against the movements of the pile head and the pile toe. The profile consists of about 4 m of sand on 7 m of clay on a thick layer of dense sand. No obvious "kink" in the pile-head load-movement curve can be discerned that could have been used to characterize a capacity. As indicated by Fellenius (1975; 2017), the capacity assessed from a static loading test can be defined by several different methods and, for the curve shown, it would typically range from a low of about 1,500 kN to the 2,100-kN maximum load applied, indeed, even beyond the maximum load.

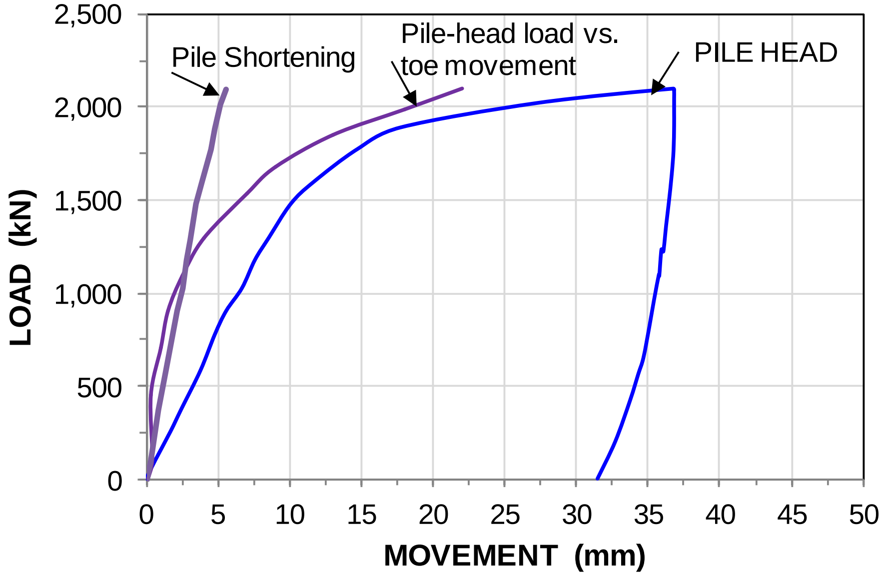

Fig. 1 Pile-head load-movement curve

What definition to employ differs widely within the geotechnical profession, as illustrated by the following example from a prediction event organized by the Universidade Federal do Rio Grande do Sul in the Araquari Experimental Testing Site, Brazil in 2015. The event comprised a 1,000-mm diameter, 24 mm long bored pile constructed in bentonite slurry a in sand deposit. The premise of the prediction was that the test be carried to a capacity defined as the load that resulted in a pile head movement determined as 10 % of the pile diameter (100-mm), a definition of "capacity" taken from the EuroCode that has its root in a misconstrued recommendation by Terzaghi (Likins et al. 2012). Moreover, the definition does not consider obvious aspects such as whether the pile is driven, bored, advanced by CFA methods, installed in open borehole, if the hole was maintained by means of slurry, the pile constructed by full-displacement method, or if the soils are clay or sand.

The task was to predict the pile-head load-movement curve for the test pile until the 10-% defined maximum load was reached. After the test results had been published, I contacted all predictors and asked them to tell me, using their own definition, what capacity the actual test curve demonstrated. Twenty-nine, about half of the total, replied and Figure 2 compiles the capacities received. The values diverged considerably. Seven accepted the organizers' assertion that the capacity was the load that gave a movement equal to 10 % of the pile diameter, whereas the others indicated values that were as low as two-thirds of the maximum—with a 21-mm movement, as opposed to the 100 mm value stipulated by the organizers. Compilations from other cases have been published that show similar diversity (Fellenius 2016b, Fellenius and Terceros 2014). It is obvious that to make use of results of static loading tests performed by others, a researcher cannot take stated "capacities" at face value, but needs to re-assess the results in order to develop a consistent data base.

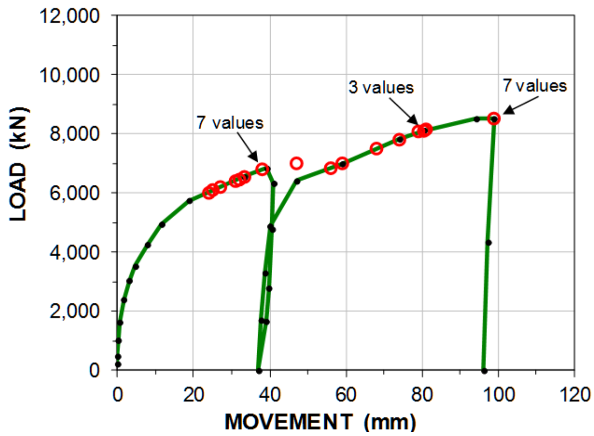

Fig. 2. Test results and capacities assessed by 29 predictors for the Araquari prediction case.

An additional phenomenon affecting the shape of the pile-head load-movement curve and the assessment of "capacity" is presence of residual force, which is an environmental axial force introduced in the pile during or after installation or construction of a pile. It is almost always present in a driven pile and, on occasion, also in a bored pile. For example, jacked-in piles have a more or less fully developed residual force. The residual force is always zero at the pile head or at the ground surface, it then increases due to accumulated negative skin friction to a maximum value at a "neutral plane" where it is in equilibrium with the below this point developing positive shaft resistance and, usually, some residual toe force.

Figure 3 compares the pile-head load-movement curves of two piles, one with fully developed residual force and one with no residual force. The piles and soil are otherwise identical in all respects. The figure includes the pile "capacities" assessed according to the in North America common offset-limit method (Davisson 1972, Fellenius 2017). Other definitions would show a similar difference between the capacity values assessed from the two curves. Obviously, presence or not of residual force will affect the evaluation of the results of a test: the capacity evaluated from the pile-head load-movement curves are very different. The fact that ultimate resistance of each element making up the piles is the same for both piles, emphasizes the uncertainty of the concept of "capacity".

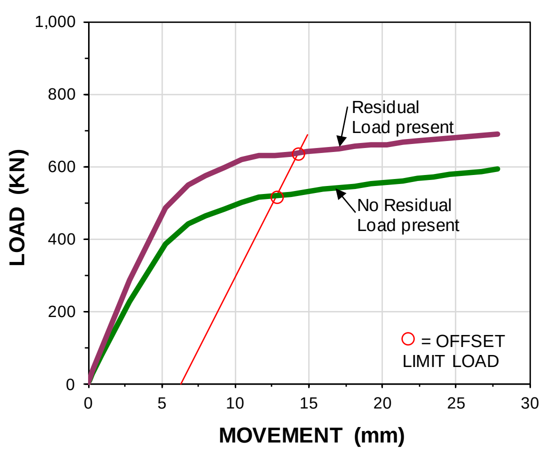

Fig. 3. The effect of presence of residual load.

3 DETERMINING THE PILE STIFFNESS, EA, AND LOAD DISTRIBUTION

It is common to instrument the pile in order to also determine the distribution of axial load down the pile. In the past, the instrumentation consisted of telltales, i.e., rods that measured shorting between two points in the pile. The simplest arrangement being a telltale to the pile toe measuring the total shortening of the pile and enabling the pile toe movement to be determined by subtracting the total pile shortening from the pile head movement, as was illustrated in Figure 1. A modern type of telltales is the Glostrext system (Hanifah and Lee 2006). The shortening caused by the applied load divided by the telltale length is the induced strain. That strain multiplied with the pile stiffness, EA, is the average load over the telltale length.

Sometimes telltale records are back-analyzed employing total stress method with a unit shaft resistance assumed constant along the pile. The average load in the pile is then located at the mid-point of the telltale distance. However, unit shaft resistance is proportional to the effective overburden stress, and it, thus, increases linearly with depth. For a linearly increasing shaft resistance, the average load over the telltale length should be plotted at the point $h/\sqrt{2} = 0.70h$ (Fellenius 2017). Plotting the average value at mid-point is a common error. It implies more shaft resistance in the upper portion of a pile and less in the lower portion. The error has contributed to the "critical depth" fallacy (Fellenius 1995; 2017).

For a telltale starting from the pile head, the applied load minus two times the difference to the average load represents the load at the telltale foot regardless of the unit shaft resistance being assumed constant or linearly increasing. Thus, a single telltale from the pile head will indicate two load values in the pile: one for the average value and one at the telltale foot. Of course, as the accuracy of telltale measured shortenings is often poor, the values are should be considered to be approximations (strain-gage measurement usually provide more representative loads). More important is that combined with the telltale-determined movement of the pile toe, the toe telltale provides the pile-toe load-movement response, a very useful record for the analysis of the pile response.

Other than the load cell that measures the applied load (relying on the pressure in the hydraulic jack is not recommended), gages that measure axial load directly are rarely used. Most instrumentation consists of vibrating wire or electric resistance gages that measure the strain induced at certain levels in the pile. Thus, the pile axial stiffness (EA) is an integral part of the measuring gage. The stiffness, however, is not a fixed entity. The two components, modulus (E) and area (A), are only known accurately for steel piles with definite shape. Premanufactured piles, such as prestressed concrete piles have constant cross section, but their modulus can vary within a wide range depending on amount of reinforcement, of course, but also on the fact that concrete, unlike steel, does not have an *a-priori* known modulus, and, furthermore, it reduces with increasing stress (or strain). For a bored pile, the actual cross section of can deviate quite widely from the nominal size depending on construction method and soil layering.

The issue of uncertain modulus can be addressed by placing a gage level at a distance below the pile head that is sufficient to ensure that the uneven stress distribution immediately below the pile head has equalized and yet not so deep that shaft resistance will interfere with the axial load in the pile. That distance is about 2 to 3 pile diameters. The records obtained from the gage level can then be used to determine directly the secant stiffness, E_sA, (N.B., modulus and cross section together) of the pile by plotting load divided by strain (Q/ϵ) versus strain (ϵ), as indicated in Figure 4. The first point in the figure is off the straight-line relation, which is probably due to initial friction in the system. However, a friction or reading correction of the 200-kN first load increment by subtracting a mere 21 kN would bring also the first point into line.

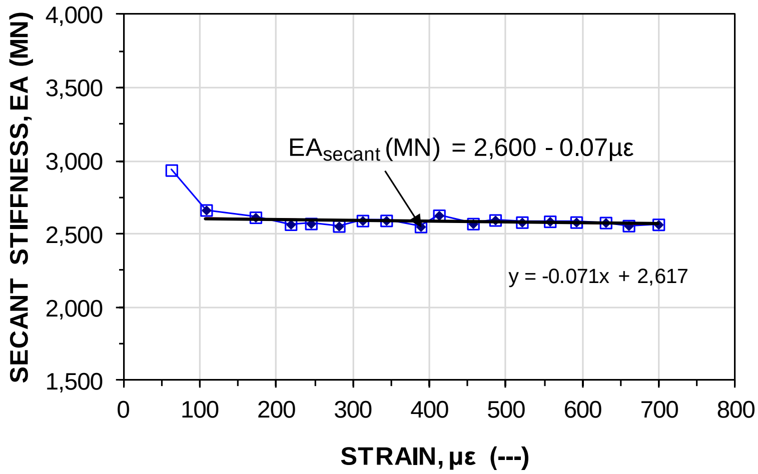

Fig. 4. Pile stiffness for a 400-mm CFA pile (after Fellenius 2012).

The direct secant modulus plot for the gage records indicates an initial pile axial stiffness (EA) of 2,600 MN reducing with increasing strain as the load increases. The decrease with increasing strain is minute for the case. The important observation is that the stiffness is obtained directly from the measurements and does not depend on a frequently uncertain value of the pile cross section and a modulus obtained from calculations or testing of a separate specimen.

The direct secant method depends very much on the accuracy of the load and strain records. As mentioned, if the pile has been unloaded and reloaded—whether this was intentional or not matters little, the strain readings will be adversely affected by the so-introduce hysteresis effect. Figure 5 shows the secant analysis of strain-gage records from a gage level in a 600-mm diameter, octagonal, 33 m long, prestressed concrete pile driven into a sandy silt. The gage level is located about 1.5 m below the pile head and 1.0 m below the ground surface. The pile was loaded in twenty-six 150-kN increments to 3,900 kN maximum load. The night before the test, an unprescribed check of the test set-up was carried out involving loading the pile to about 1,000 kN without taking records. As seen, the incident resulted in a disturbance of the initial part of the secant stiffness line.

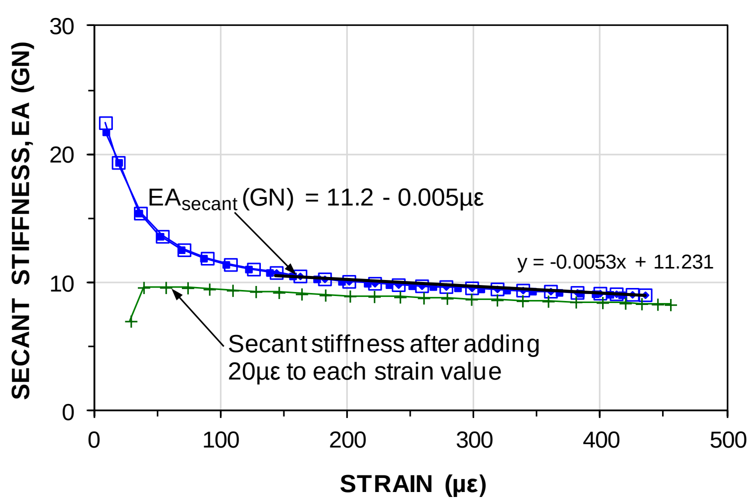

Fig. 5. Pile stiffness for a 600-mm diameter prestressed pile (after Fellenius 2012).

An initial curvature of the direct stiffness line can often be removed by adding a small strain value to all measured strains. In this case, the added value is small, only 20 μϵ. Also for this pile, the deviation of the first point of the corrected stiffness line is probably due to friction in the system and it could have been removed by adding 77 kN to the first increment of load. The point of the example is to show that a preceding full unloading-reloading cycle has affected the gage records. Indeed, unloading-reloading should never be a part of a static loading test as such events will adversely affect the records and may cause them to be unacceptable for analysis.

The direct secant method will not work where the pile is subjected to shaft resistance along the length above the gage level. For those records, the stiffness can be determined from an incremental stiffness method (tangent stiffness method). Figure 6 show head-down test results from a gage level at 11 m depth in a 1.0-m diameter bored pile constructed in a marine clay. The first measurements are affected by shaft resistance and the linear trend only develops after the shaft resistance is fully mobilized. Note, in strain-softening or strain-hardening soil, the slope of the line will be steeper or flatter, respectively, than the true stiffness line.

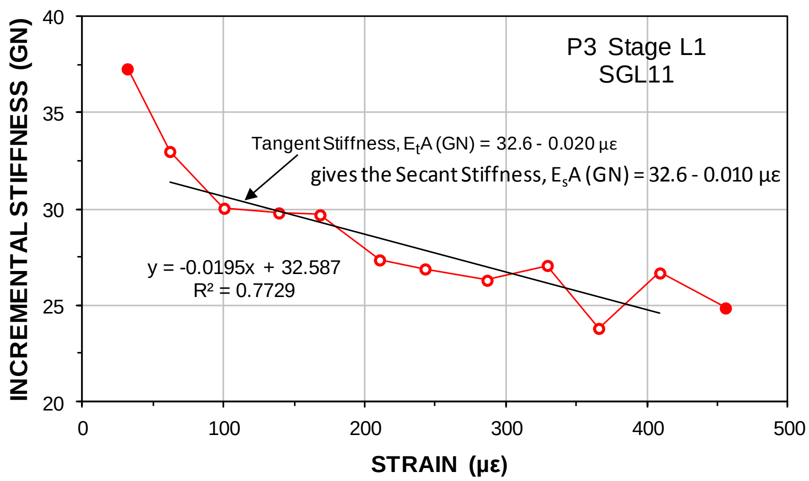

Fig. 6.Pile incremental stiffness for a gage level 12 m down in a 1,000-mm diameter bored pile. (Data from Fellenius and Tan 2010).

The loads translated from the strain records apply the secant stiffness times the strain, $E_s A \epsilon$. The y-intercept (stiffness at zero strain) is the same for the secant and incremental plots and the slope of the straight line in the incremental plot is twice that of the secant line. Thus, for the shown plot, the secant stiffness, E_sA (GN), is 32.6 - 0.010μϵ. Because the incremental method is based on differentiation, small inaccuracies in the records are exaggerated and an incremental stiffness plot is always more scattered than a direct secant plot. Thus, provided that the tests is carried far enough, that is, the applied load has imposed sufficient strain in the pile for a meaningful plot of the data, the records can be used to determine the stiffness of a test pile at the various gage levels down the pile (Fellenius 1989).

Figure 7A shows the load-movement curve from a bidirectional test on a 1.85 m diameter, 65 m long bored pile, for which accidental hydraulic leak necessitated an unloading and reloading cycle (Thurber Engineering Inc., Edmonton; personal communication 2016). Figure 7B shows the incremental stiffness curves from the initial and reloading records.

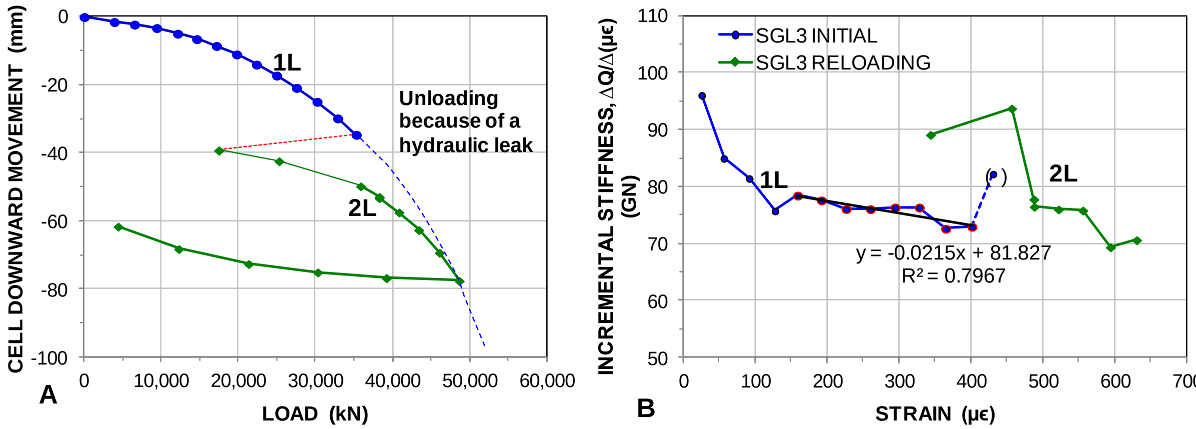

Fig. 7. Effect on the incremental stiffness curve from an unloading-reloading cycle.

It is clear from the examples illustrated in Figures 5 and 7 that using the load and strain records to determine the pile stiffness requires that the data must not be affected by unloading-reloading cycles. Many practitioners incorporate intentional unloading-reloading cycles in a static loading test and they frequently keep the applied load constant for a longer duration at one load level or other. I have many times tried to have users of such actions explain to me what they expect to gain from this extraneous imposition on the test procedure, but nobody had ever been able to tell me anything else than "*this is what we always do*" or "*this is what I think the code says I must do*". The fact is that nothing is gained by this and such deviation from the simple direct incremental procedure will instead result in that the investment in the instrumentation is wasted.

Note, translating strain measurements to load requires that the measured strains cover an acceptable range. There is little sense in investing in instrumentation if the induced strain at the maximum load are smaller than 200 με. Designing the pile and test toward achieving strains in excess of 500 με is preferable. If a calculation check shows that the strain are small, and it is important to determine the load distribution, it is much better to do the test on a smaller diameter pile and cautiously "extrapolate" the result to the larger diameter pile. For example, a pile with half the diameter will show four times larger strain for the same load. In the process, it might be realized that the original pile design is too conservative and a smaller diameter pile will suffice. Of course, the transfer of the analysis results for the smaller diameter pile to the larger diameter pile must be carried out with care and with some conservatism.

Furthermore, to obtain records that can be used for determining the pile axial stiffness and load distribution, requires that a static loading test, be it a head-down test or a bidirectional test, be carried out by applying equal increments of load and time. At each level, the load must be maintained (held constant) for an equal length of time. The load-holding time can be short or long, but an interval shorter than 10 minutes or longer than 20 minutes is impractical. The frequently applied 5-minute load-holding interval is not suitable when testing piles with strain-gage or other instrumentation down the pile. The reason is that it takes a few minutes after adding an increment for the pile to react, i.e., for the gages down the pile to register the load change. Therefore, the applied load and the strain-gage records are best combined after at least ten minutes of constant load at the pile head (or bidirectional cell). And such "waits" must be the same for all gage levels.

These days, it should not be necessary to state that the pump supplying pressure to the hydraulic jack must be able to maintain the pressure automatically. However, only too often is an investment in a sophisticated test employing instrumentation severely affected by the variation in the applied load level during the load-holding phases originating from a pump without automatic load-holding means or, even, a manually operated pump. It must be realized that the load exerted by a hydraulic jack is affected by friction between the two cylinders making up the jack and that friction is different when pressure is increased and when it is let to relax. Naturally, the load is monitored with a load cell. However, load cell or not, the load will vary up and down as the pressure pump is relaxed or engaged. A pump with an automatic pressure-holding device will minimize the effect of friction as well as maintain an even load.

The days of manual recording of the records are over and records are now obtained at every 30 seconds, or so, and stored in a data collector (data acquisition unit). However, the records must be obtained using a single data collector. Do not use one collector for the load records and pile movements and a separate one for the strain records. Marrying records using the time stamp does not work. My experience is that ever so often a line shift is made and a "divorce" occurs before the marriage is completed.

The objective of determining the stiffness is to find the distribution down the pile for the applied loads. The pile for which the load-movement results were shown in Figure 1 was instrumented with four levels of strain-gages. The pile stiffness was determined using the methods described above and the loads at each gage were determined from the measured strains. Figure 8 shows the so-obtained load distributions. In addition, it shows the calculated pile toe load plotted versus the measured pile toe movement. These results together with the load distributions are the key results from a static loading test for use toward a piled foundation design.

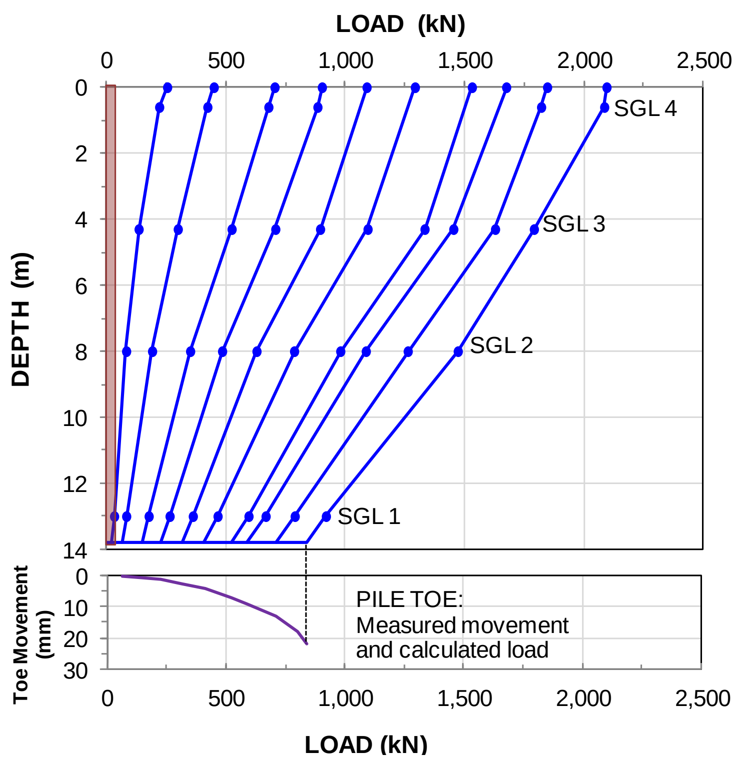

Fig. 8 Load distribution determined from strain gage records.

4. EMPLOYING THE TEST EVALUATIONS IN PILED FOUNDATION DESIGN

Figure 9 shows the results of testing and analysis of a 25 m long, strain-gage instrumented auger-cast pile constructed through sand and silty clay to bearing in a glacial till designed according to the Unified Design Method (Fellenius 2004; 2016a; 2017). The spacing between the piles was large enough for the piles to be acting as single foundation-supporting units. The distributions of load and settlement are shown for a test pile, at the site. After construction, a fill was placed over the site, which introduced soil settlement and, therefore, downdrag. Consequently, the long-term load distribution will increase downward from the applied dead load to a maximum at the force equilibrium—the neutral plane. This is where the dead load plus the drag force due to accumulated negative skin friction are equal to the positive shaft resistance and the toe resistance below. The latter depends on the magnitude of the imposed toe movement. The neutral plane is also the settlement equilibrium, the location where there is no relative movement between the pile and the soil. The figure demonstrates how the forces and soil deformation interact and that the settlement of the pile head, i.e., of the piled foundation, is governed by the settlement at the neutral plane. The location of the neutral plane is controlled by the loop from toe load that governs the force equilibrium that determines the settlement equilibrium that results in a toe movement that establishes the toe force. For the loop to close, the final toe force must be equal to the starting toe force. There is only one neutral plane location that satisfies this requirement. The final result is the pile cap settlement as indicated in the figure.

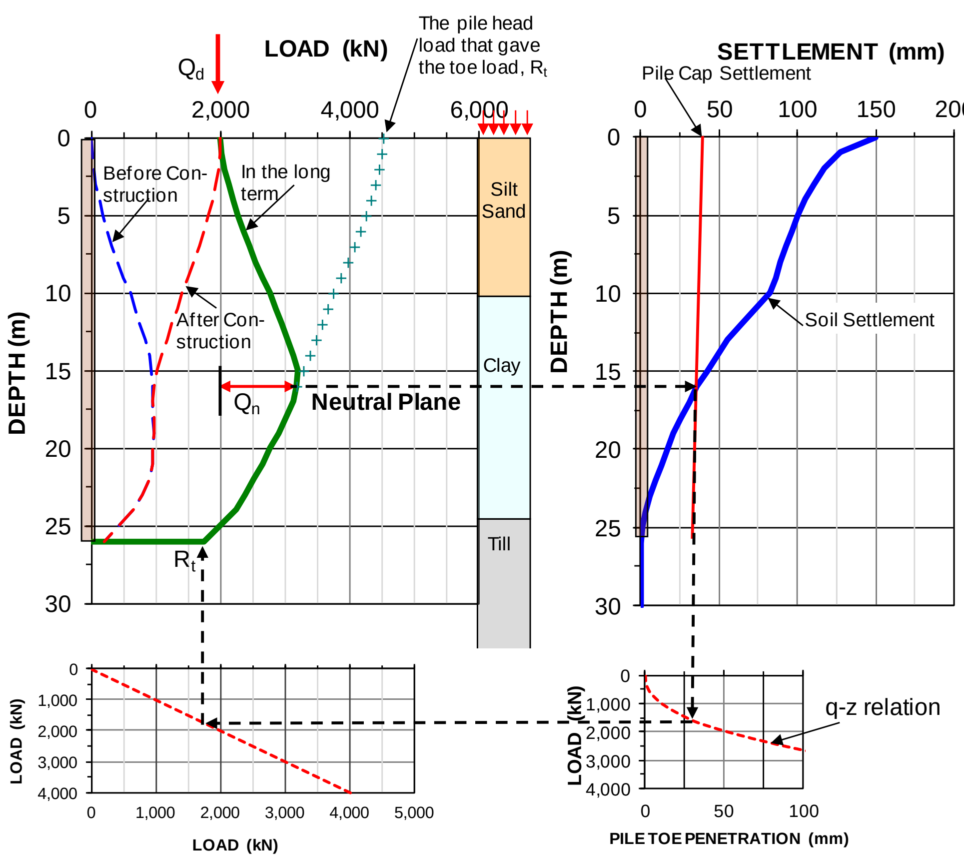

Fig. 9. The unified pile design loop for determining settlement of single piles and small pile groups, (Fellenius and Ochoa 2009).

The results of the instrumented static loading test and strain-gage analyses provide the necessary load distributions and pile toe movement to combine with the settlement analysis (which must incorporate all aspects causing the soil to settle) in order to determine the piled foundation settlement. If the settlement is too large, lowering the neutral plane will reduce it, which can be achieved by either lengthening the pile or reducing the load. The main point is that the proper testing procedure, analysis methods, and design process enable implementing a safe design within acceptable settlement. "Capacity" need not be a part of the picture.

5. CONCLUSIONS

The scatter of capacity values resulting from the various definitions in vogue in the profession and the effect of residual force on the evaluation of capacity are addressed. It is shown that including an unloading-reloading event in a static loading test will adversely affect the calculation of the pile axial stiffness. The analysis of the strain-gage records aim to determine the load distribution and pile toe-load-movement, which, when combined with the soil settlement in an interactive analysis, will show the settlement of the piled foundation. Designing for settlement makes the conventional "capacity approach" redundant and provides a more reliable design.

REFERENCES

Davisson, M.T., 1972. High capacity piles. Proc. of Lecture Series on Innovations in Foundation Construction, ASCE Illinois Section, Chicago, March 22, pp. 81-112.

Fellenius, B.H., 1975. Test loading of piles—Methods, interpretation, and proof testing. ASCE Journal of the Geotechnical Engineering Division 101(GT9) 855-869.

Fellenius, B.H. and Atlaee, A., 1995. The critical depth – How it came into being and why it does not exist. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering Journal, London, 113(2) 107 111.

Fellenius, B.H., 2004. Unified design of piled foundations with emphasis on settlement analysis. Honoring George G. Goble—Current Practice and Future Trends in Deep Foundations. Geo-Institute Geo-TRANS Conference, Los Angeles, July 27-30, 2004, Edited by J.A. DiMaggio and M.H. Hussein. ASCE Geotechnical Special Publication, GSP125, pp. 253-275.

Fellenius, B.H., 1989. Tangent modulus of piles determined from strain data. ASCE, Geotechnical Engineering Division, the 1989 Foundation Congress, F.H. Kulhawy, Editor, Vol. 1, pp. 500-510.

Fellenius, B.H. 2012. Critical assessment of pile modulus determination methods. Discussion. Canadian Geotechnical Journal, 49(5) 614-621.

Fellenius, B.H., 2015. Field Test and Predictions. Segundo Congreso Internacional de Fundaciones Profundas de Bolivia, Santa Cruz May 12-15, Lecture, 22 p.

Fellenius, B.H., 2016a. The unified design of piled foundations. The Sven Hansbo Lecture. Geotechnics for Sustainable Infrastructure Development – Geotec Hanoi 2016, edited by Phung Duc Long, Hanoi, November 23-25, pp. 1-26.

Fellenius, B.H., 2016b. Fallacies in piled foundation design. Geotechnics for Sustainable Infrastructure Development – Geotec Hanoi 2016, edited by Phung Duc Long, Hanoi, November 23-25, pp. 41-46.

Fellenius, B.H., 2017. Basics of foundation design—a textbook. Electronic Edition, www.Fellenius.net, 451 p.

Fellenius, B.H. and Ochoa, M., 2009. Testing and design of a piled foundation project. A case history. Journal of the Southeast Asian Geotechnical Society, Bangkok, 40(3) 129-137.

Fellenius, B.H. and Terceros, M.H. 2014. Response to load for four different bored piles. Proceedings of the DFI-EFFC International Conference on Piling and Deep Foundations, Stockholm, May 21-23, pp. 99-120.

Hanifah, A.A. and Lee S.K., 2006. Application of global strain extensometer (Glostrext) method for instrumented bored piles in Malaysia. Proc. of the DFI-EFFC 10th Int. Conference on Piling and Deep Foundations, May 31 - June 2, Amsterdam, 8 p.

Likins, G.E. Fellenius, B.H., and Holtz, R.D., 2012. Pile driving formulas. Pile Driver Magazine, No. 2, pp. 60-67.