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Common Mistakes in Static Loading-test Procedures and Result Analyses

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ABSTRACT. Static loading tests on piles are arranged in many different ways ranging from quick tests to slow test, from constant-rate-of-penetration to maintained load, from straight loading to cyclic loading, to mention just a few basic differences. Frequently, the testing schedule includes variations of the size of the load increments and duration of load-holding, and occasional unloading-reloading events. Unfortunately, instrumenting test piles and performing the test while still using unequal size of load increments, duration of load-holding, and adding unloading-reloading events will adversely affect the means for determine reliable results from the instrumentation records. A couple of case histories are presented to show issues arising from improper procedures involving unequal load increments, different load-holding durations, and unloading and reloading events—indeed, to demonstrate how not to do. The review has shown that an instrumented static loading test, be it a head-down test or a bidirectional test, performed, as it should, in a series of equal load increments, held constant for equal time, and incorporation no unloading-reloading event, will provide data more suitable for analysis than a test performed with unequal increments, unequal load-holding, and incorporating an unloading-reloading event. No useful information is obtained from prolonging the holding time for the maximum load.

KEYWORDS. Pile, Loading test, Strain, Extensometer, Tangent stiffness, Load cycle.

1. INTRODUCTION

A routine static loadinging test comprises measuring pile head movements imposed by a succession of load increments, often of unequal size, to the pile head. Over the years, many have added features to the test procedure. Such features are incorporating one or a few unloading-reloading events, and prolonging load-holding at one or a few load levels, sometimes by a couple of hours, sometimes for much longer time, particularly at multiples of the desired or intended working load. The features vary between geographical areas, influenced more by interaction between local communities and 'fashion' than by any particular geotechnical similarity or difference between these areas. Considering that completing a static loading test over the course of a day is considerably less costly than one that requires several days to perform, it is surprising that no study has been made about the value of one or other feature as compared to another, or to including or not including it in the test schedule. Moreover, as the pile-head load-movement of a routine test is not particularly sensitive to variations of the applied load or measured movements (although measured using dial gages graded to 0.001 inch or 0.01 mm) or to the readings of load and movement not being quite simultaneous, little attention is directed to the quality (accuracy, reliability, and precision) of the actual recording of the measurements.

Conventionally, and regrettably, the results of a static loading test are primarily used for determining pile "capacity"—sometimes by a judgment call, but most of the time by applying a definition. Moreover, records limited to load versus pile-head movement do not allow separating the pile toe response from the shaft resistance or determining the distribution of axial load in a tested pile. Sometimes, the soil profile at the test site is well established by in-situ tests and test on recovered soil samples and this information can be used to infer a load distribution. However, the relevance of that distribution is only as good as the methods for determining it from the soil profile —leaving much to be desired. Ideally, the load distribution should be determined from the test by means of instrumenting the test pile and the so-determined load-movements be correlated to the soil profile.

Indeed, instrumenting a test pile to obtain more specific information has, during the past decades, become more assured and less costly and, therefore, loading tests on instrumented piles have become more common also for routine tests. Unfortunately, the instrumented pile tests are mostly carried out using the same schedule and procedures as used for the routine tests.

The current standard of practice of performing a static loading test and analyzing results is unsatisfactory, as will be illustrated in the following case histories taken from a couple of actual full-scale tests.

2. RECORDS OF LOAD-TIME AND LOAD MOVEMENTS

2.1 Case 1

Case 1 is a reference case to the other cases presented here. The test was performed at the B.E.S.T. experimental test field in Bolivia (Fellenius 2017). The pile is a 620-mm diameter, 9.5 m long, bored pile constructed in a soil profile composed of about 5.5 m of loose sandy silt over a deposit of compact silty sand. The pile was strain-gage instrumented at three levels: 7.5 m, 5.0 m, and 2.0 m below the pile ground surface. The loads were supplied by a hydraulic jack and reaction was from four reaction piles. The applied load was determined using a separate load cell. The test was performed in 20 increments, each applied every 10 minutes—a so-called "quick test"—and a data collector was set to store all records at a about 30-second intervals. Figure 1 shows the measured loads and pile-head movements versus time. Figure 2 shows the measured load-movement curve. The records indicate no remarkable occurrence.

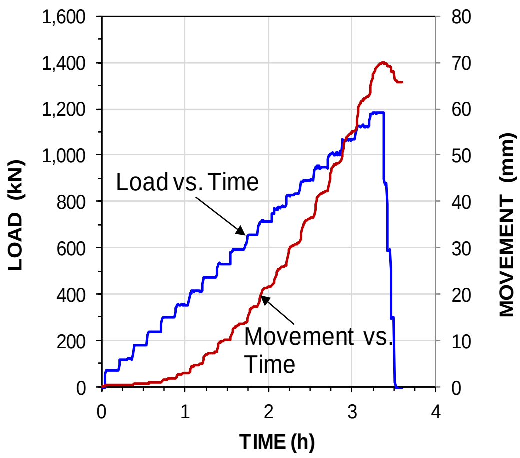

Figure 1 Case 1. Load and movement vs. time

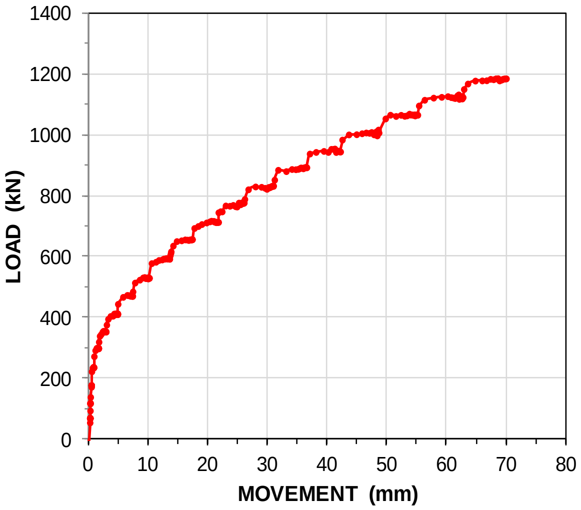

Figure 2 Case 1. Load vs. movement

The test was a part of a prediction event (Fellenius and Terceros 2017), where a large number of professionals predicted the load-movement curve of the test pile (and of three other test piles). After the test, the participants were provided with the post-prediction load-movement records and asked to assess the pile capacity from the load-movement curve per each participant's usual and preferred definition and method. Figure 3 compiles the capacities reported by 94 professional geotechnical engineers, almost all with some justified claim on expertise in pile analysis.

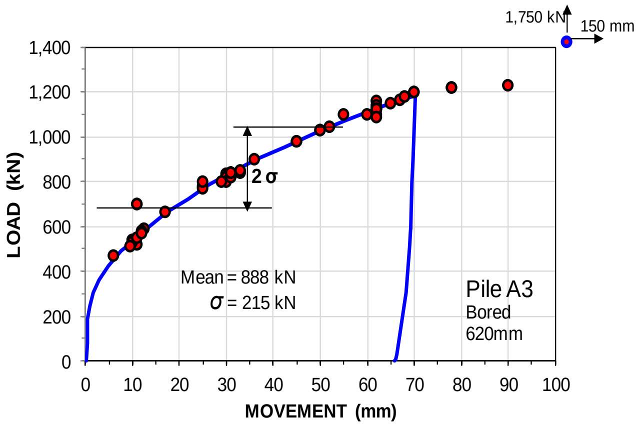

Figure 3 Case 1. Capacities assessed from actual pile-head load-movement curve as assessed by 94 participants (Fellenius 2017)

The scatter of "capacity" is huge. Some of the assessments were based on a standard or code local to the individual making the assessment, others were based on a reference to a published definition, or the individual's own method. Although the concept of "capacity" based on the ultimate resistance of the test pile, as determined from the load-movement curve of the test, would seem unambiguous, the scatter of assessments makes it clear that the profession does not employ a common definition.

"Capacity" is a one-point value derived from a complex load-movement and a tenuous and contrived concept that is applied with great disparity in the profession. Therefore, relating a "working load" to "capacity" by way of safety or resistance factors as expressed in codes and standards is not a particularly reliable design approach. The response of a piled foundation to load is better assessed by study of the full load-movement response.

2.2 Case 2

Case 2 is from a static head-down loading test on a 1,000-mm diameter, 30.4 m long pile constructed in Singapore through about 25 m of alternating layers of loose to compact sandy silt, medium sand, and soft clay above a very dense silty sand. The pile was instrumented with the Glostrext system, which measures distance between Glostrext anchor points, here spaced about 3.0 m, and the pile movements at each anchor point. The Glostrext system enables determining pile shortening between anchor points, which values are converted to strain between the anchor points. The load was applied by hydraulic jacks working against a loaded platform and were measured by a separate load cell. The pile head movement was measured by dial gages acting against a reference beam. Movements of the reference beam due to heave of the kentledge supports were measured by optical survey and the pile head movements were corrected for the movements of the reference beam. (The gradually occurring heave of the beam due to the unloading of the kentledge support was about 1.5 mm at the maximum load). The records indicate that the jack-pump supplying and adjusting the applied load was manually operated and controlled. All data were recorded and stored on a data acquisition box.

The test procedure was carried out in two phases. Phase 1 comprised four load increments applied every one hour up to the desired working load (6,154 kN), which was held for six hours, whereupon four additional load increments were applied to twice the working load, which was held for 36 hours. About 48 hours after start of test, the pile was unloaded, then, Phase 2 started by applying four increments to the working load, which was held for 16.5 hours. The pile was then given six additional increments to a 14,750-kN maximum test load, held for 16 hours, whereafter the pile was unloaded. The total test duration was 120 hours.

Figure 4 shows the assigned and measured loads and measured movements at every three minutes during the test plotted versus time, and Figure 5 shows the Case 2 assigned and measured loads versus pile-head movement.

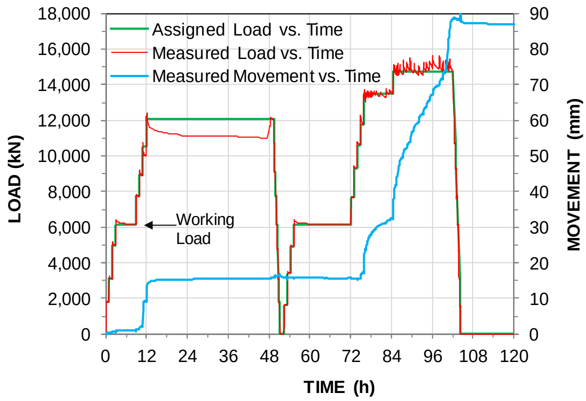

Figure 4 Case 2. Load-time and movement-time

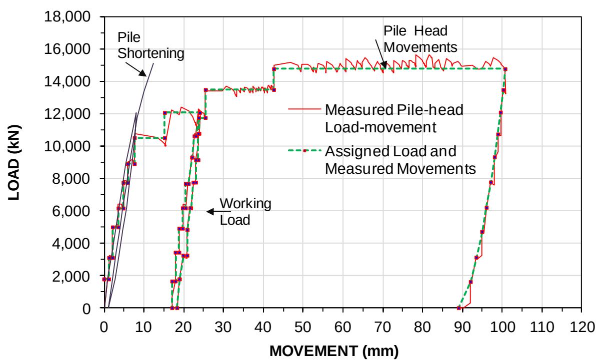

Figure 5 Case 2. Load-movement

As shown, neither the load increments, nor the load-holding times between the extra long durations were particularly equal. More obvious, the loads levels were not well maintained after the first about 12 hours of testing. From then onward and until the last hour of the 36-hour long load-holding, the loads were let to drop, only to be adjusted to proper level at the last half hour. This was not repeated during the last load-holding at the maximum load. However, the then instigated frequent adjustments of the applied load resulted in an average load level larger than the assigned load.

2.3 Case 3

Case 3 is from a static head-down loading test on a 1,200 mm diameter, 55 m long bored pile constructed to support an apartment building in Hanoi. The soil profile consisted of about 44 m of clayey sand and sandy clay followed by a gravel and cobble deposit. The pile was instrumented with eleven levels of single pairs of vibrating-strain gages at depths 8.4, 10.5, 16.0, 21.0, 25.0, 30.0, 35.1, 40.5, 44.5, 50.0, and 53.5 m. The load was applied by a bank of eight hydraulic jacks working against a loaded platform (kentledge arrangement) supported on the ground. The jack pressure was manually controlled.

The pile head movement was recorded every five minutes and all strain-gages and telltales were recorded every 3 minutes using a data logger (data collector). All movement and strain records were labeled with the assigned load, mistakenly believed then to be a measured load. However, no separate load cell to actually measure the applied load was included. Nor was the potential heave of the reference beams measured.

Before starting the test, a 50-kN "seating" load was applied to the pile head. After unloading, all gages were "zeroed" and a first test-series (Phase 1) started by applying two 4,950-kN load increments to 9,900 kN (the working load for the supported structure) followed by a two-decrement unloading. Each loading and unloading level was held for 60 minutes. After a 6-hour wait, Phase 2 started with an initial 4,950-kN load followed by eight equal increments of 2,475 kN to 24,740 kN followed by unloading using the same decrements and durations. Phase 3 started after a six-hour wait with the same first step equal to the working load and then followed by twelve 4,950-kN increments. When applying the 13th increment, raising the total load to 34,640 kN, the pile broke. The uppermost strain-gage (8.4 m below the pile head) then registered 650 microstrain.

Figure 6 shows assigned loads and loads plotted versus time. No measured pile-head load vs. time is included because, as mentioned, no load cell was included in the test to record the applied loads.

Figure 7 shows the assigned load versus measured pile-head movement. The measured load-movements do not suggest anything special, e.g., "capacity", about the test pile response.

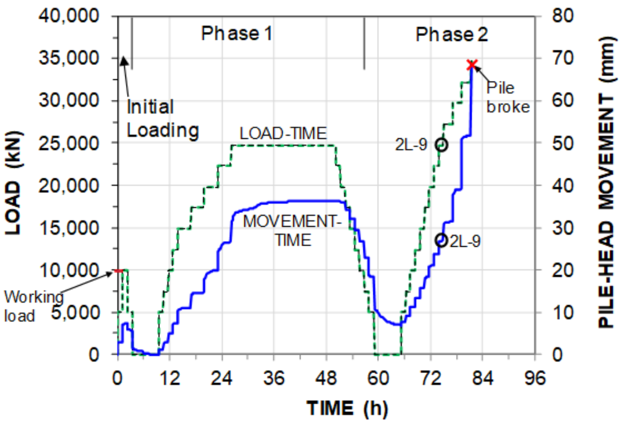

Figure 6 Case 3. Load-time

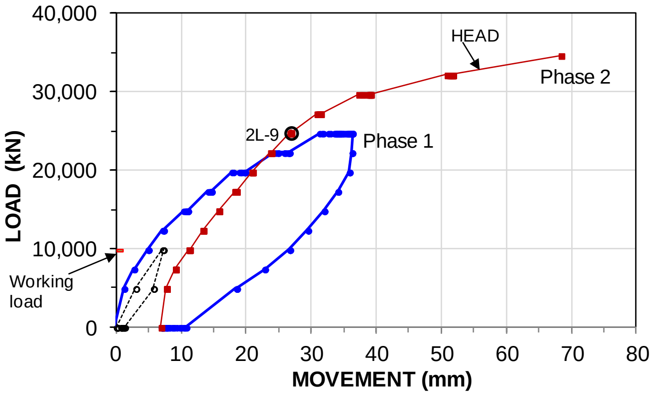

Figure 7 Case 3 measured load-movement curves

2.4 Case 4

Case 4 shows the results of a bidirectional (BD) test on a 1,400-mm diameter, 47.1 m long bored pile in Singapore. Figure 8 shows the measured upward and downward, bidirectional load-movement curves of the test.

The test was performed in a twelve equal load increments (1,700 kN) held equal length of time (1 h). The maximum BD load, 1.5 times the 1,400-kN working load, was held for about 24 hours. The soil profile consisted of an upper about 22 m thick layer of soft and loose clay and silt deposited on very dense sand and gravel. The groundwater table was at a depth of about 2 m.

In addition to measuring the BD upward and downward movement and pile head movement, telltales were installed to measure also the pile toe movement. The pile was further instrumented with 15 levels of vibrating-wire strain-gage pairs evenly distributed along the pile; eleven levels above the BD and four below. Figure 9 shows the load-time and movement-time curves of the test. The loads levels were maintained manually, which made the load-holding somewhat variable; note the drops of load occurring over night and in the late-afternoon during the long load-holding period; emphasized in the extracted detail.

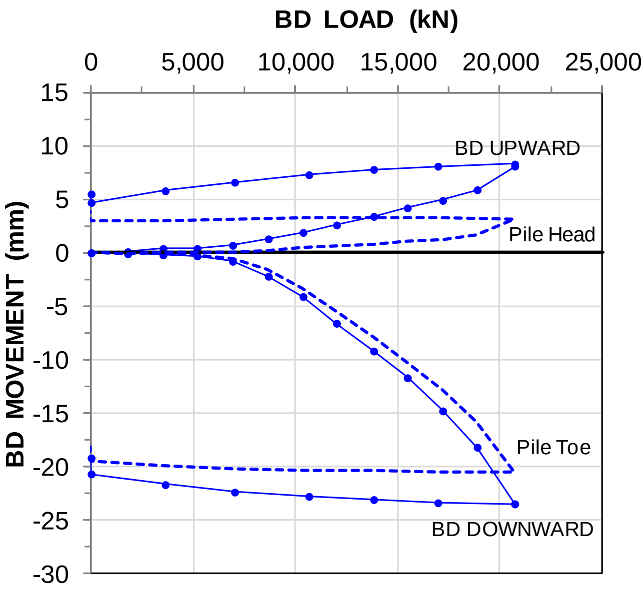

Figure 8 Case 4 Bidirectional test load-movement curves

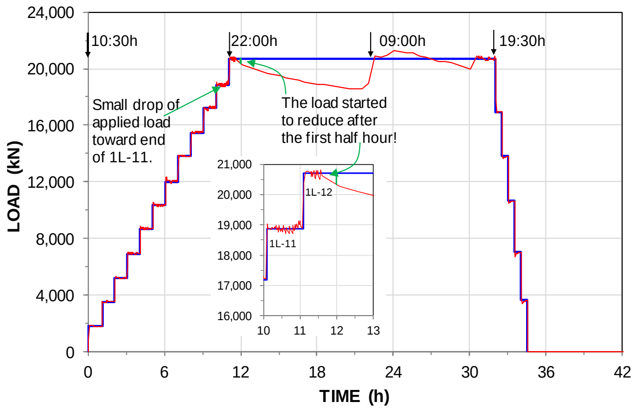

Figure 9 Case 4 load versus time

2.5 Case 5

Case 5 reports a static head-down loading test on a 500-mm diameter, 21 m long, jacked-in (pressed-in) pile in Singapore installed through about 7 m of soft and loose fill into compact sandy silt (weathered Jurong formation). The pile was instrumented at 8 levels of Glostrext anchor system (see Case 2) providing strain records at 7 depths. The pile head movement was measured by dial gages acting against a reference beam. Movements of the pile head were measured by optical survey, which eliminated the influence of reference beam movements due to heave of the kentledge support. (A reference beam was used and its gradually occurring heave was measured. It was about 2 mm at the maximum test load). The jack-pump supplying and adjusting the applied load was manually operated. All data were recorded and stored on a data collector.

The test pile was a 500-mm O.D. spun pile with a 110-mm wall manufactured as a pretensioned reinforcement bars and poststressed concrete pile ("prestressed pile"), using high quality concrete. The jacked-in force is shown in Figure 10 together with the N-indices from an adjacent borehole. The jacking force increased proportionally with the depth, but for the last about 2 m penetration, when the pile toe encountered the less weathered, larger resistance soil. The installation terminated when the jacking force was equal to twice the intended pile working load.

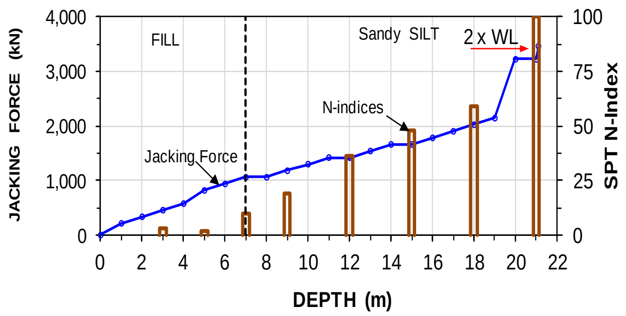

Figure 10 Case 5 Distribution of jacking force and N-indices

The test procedure comprised three phases. Load increments were added every about half hour. In Phase 1, the loading went to the desired working load (1,770 kN), which was held for 24 hours, whereupon the test pile was unloaded and, in Phase 2, re-loaded to twice the working load, which was also held for 24 hours, whereupon the pile was unloaded. In Phase 3, the pile was re-loaded to three times the working load held for about 40 hours. The total test duration was 104 hours.

Plotted versus time, Figure 11 shows the assigned loads, and measured loads and movements every three minutes during the test. As shown, during the long load-holding durations, the applied loads were not maintained, particularly not during the night. The figure also shows the measured movements of the pile head and the pile toe versus time.

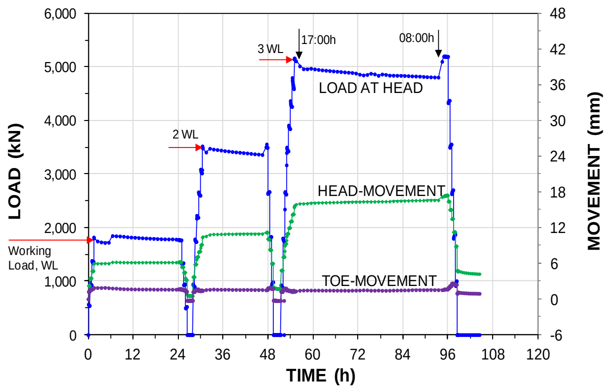

Figure 11 Case 5 Load-time and movement-time

Figure 12 shows the measured load-movement of the pile head and pile toe. The rebound of the pile during the night due to reduction of the applied load is clearly noticeable in diagram. The pile toe movement is the difference between the movement of the pile head and the pile shortening (compression) measured by the Glostrext system. After the about 1 to 2 mm movement for the initial load of Phase 1, the pile toe hardly moved. It is likely that this movement is due to a variation of the level of the reference beam(s) and an up to about 2-mm error has affected the all movement records.

Figure 13 opens out the response of the pile toe (calculated from subtracting the measured pile compression from the pile head movement measured separately) and indicates that the toe movement measured for the very first load increment is probably not due to an actual toe movement, but to an imprecision of the initial pile head movement and small continuing imprecisions. This demonstrates the value of including two independent systems of measuring pile head movement, notwithstanding that, for the subject case, the movement correction would amount to a mere about 1.3 mm.

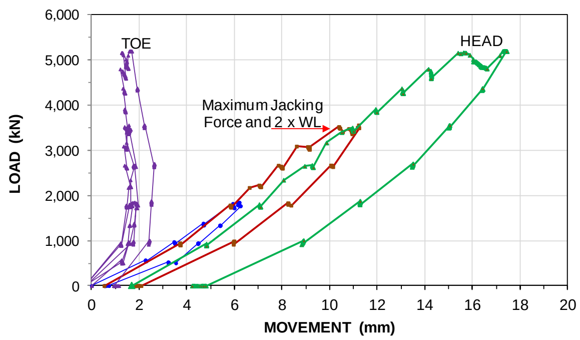

Figure 12 Case 5 Load--movement of the pile head and pile toe

It should be noted that the jacked-in installation procedure is in effect a loading test and a following static loading test is but a re-loading of the test pile. If this fact would be recognized, the static loading test might be found redundant. Moreover, as every jacked-in pile at the site is so-tested, the design could be made less conservative.

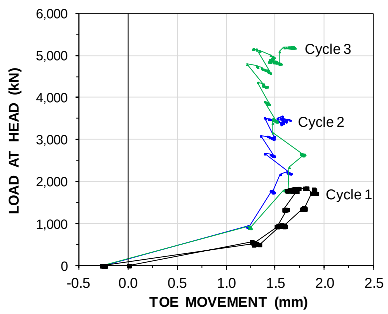

Figure 13 Case 5 Applied load versus pile toe movement

3. LOAD-STRAIN AND LOAD DISTRIBUTION

3.1 Case 1

Figure 14 shows the applied load versus the average measured strain in the Case 1 pile, gage levels, SGL1, SGL2, and SGL3 at depths of 7.0, 5.0, and 2.0 m in the 9.5 m long pile (pile head at ground surface). A first step of evaluating strain-gage records is to convert the measured strain to axial load equal to the strain times the pile stiffness: $Q = EA\varepsilon$, where EA is the stiffness, E is the Young modulus of the pile material, and A is the pile cross sectional area. Other than for a steel H- pile, pipe pile (not concrete-filled), E is not usually a well-known parameter. And, other than for a pre-manufactured shape, e.g., a prestressed, precast concrete pile (spun pile), the area, A, is not well known. Fortuitously, there is no need to know the modulus, E, and the pile area, A, separately, because the actual load-strain test records can be used to determine the pile stiffness (Fellenius 1989; 2019). The conditions for this statement to be true is either that no pile shaft resistance acts between the pile head and the gage level or that the shaft resistance has reached a plastic state. The first condition is usually true close to the pile head. Thus, strain records measured at a gage level near the pile head can be directly used to calibrate the pile stiffness of a test. Of course, the applied load needs to be known. In the zone nearest the pile head, the strains are not evenly distributed across the pile section; a minimum distance is required to develop a uniform stress plane, over which the average of the gage pair records does represent the strain at the gage location. Therefore, the "calibration" gage level needs to be located about one to two pile diameters below the pile head.

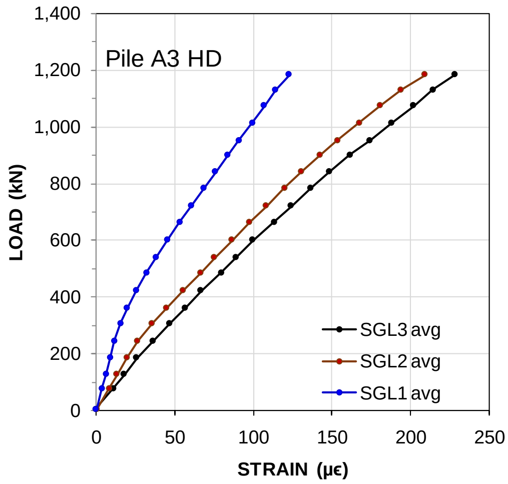

Figure 14 Case 1. Average load versus measured strain

While the slope of the load-strain curve can be determined by linear regression to yield the pile stiffness, this slope is not the best approximation of the apparent stiffness of a pile, because it reduces with the reduced mobilization of shaft resistance between the pile head and gage level and, also, because the stiffness of a pile, notably, a concrete pile, is not constant, but reduces with the increase of imposed strain. Therefore, the better way to work out the stiffness from a gage level unaffected by shaft resistance (i.e., near the pile head) is to use the secant plot: load divided by strain (Q/ε) plotted versus strain (ε) as in Figure 15, showing the "direct secant method" (Fellenius 1989; 2019). N.B., both load and strain records are always affected by some error and imprecision. In the case of a field test, usually, the strain-gage "zero" may be uncertain or some residual strain may be present. However, as the load and strain increase, the relative effect of this disappears and a straight line appears which slope can be determined by linear regression. (If the discrepancy in the zero value and/or a residual force load are small, then the slope of the plot can be adjusted to show the slope range to include also the beginning of the plot). If the zero-value discrepancy is more than a few microstrain, the direct method is not suitable.

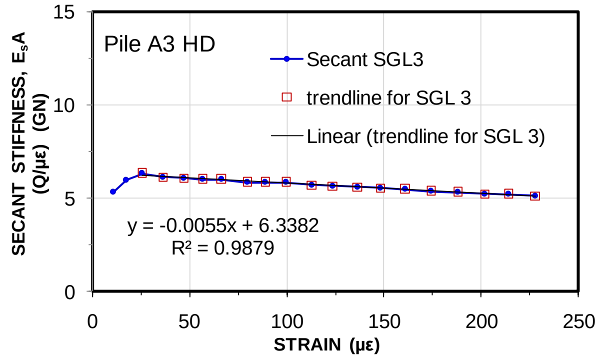

Figure 15 Case 1. Direct secant method evaluation of the pile stiffness

The linear regression equation obtained shows the stiffness at zero strain and how the stiffness reduced with increasing strain. For the SGL3 gage level, the stiffness was 6.34 GN at low strain and reduced to about 4.30 GN at 200 µε. Considering the nominal cross section of the Case 1 pile, 0.302 m^2, this correlates to E-moduli of 21 GPa and 14 GPa, respectively, which are unrealistically low values. Applying a more realistic E-modulus of, say, 25 GPa to the 6.34-GN stiffness results in a 570-mm pile diameter as opposed to the nominal 620-mm diameter, an entirely possible deviation of the pile size, notwithstanding that a bored pile diameter is usually larger than the nominal. Obviously, evaluating the test results from an assumed E modulus and the nominal diameter could result in a considerable error in the resulting load value. Employing the stiffness (EA) from the records avoids this inaccuracy.

The gage records from further down the pile, however, cannot be evaluated using the direct method because of the strain records are affected by the shaft resistance above the gage level. However, on the condition that the shaft resistance is of about the same magnitude for consecutive load increments, i.e., plastic shear response has developed (neither strain-hardening nor strain-softening occurs), a series of change of load from one increment to the next divided by change of strain plotted versus strain will be a straight line as shown in Figure 16. Linear regression of the straight portion of the line establishes the relation for the "incremental stiffness" of "tangent stiffness" of the pile (Fellenius 1989; 2019).

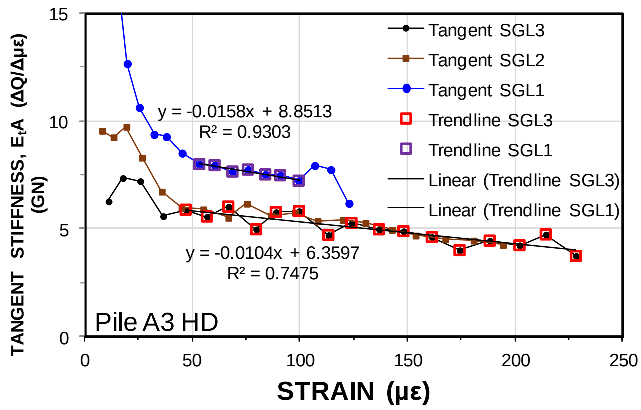

Figure 16 Case 1 tangent method evaluation of the pile stiffness

For Case 1 pile, the tangent stiffness (GN) of SGL2 and SGL3 is 6.36 - 0.010ε (the strain to be input in units of µε, 10^{-6}), which translates to a secant stiffness of 6.36 - 0.005ε, essentially the same as that determined by the direct secant method applied to SGL3. For SGL1, the low-strain E-modulus is 8.85 GN, which correlates to an E modulus of 29 GPa, which is a plausible value. However, it is not likely that the concrete modulus would be significantly different at 7 m depth as opposed to at 2 and 5 m depths. The reason must therefore be that the pile cross section is wider at 7 m depth. The relative difference in width is the square root of the stiffness ratio, i.e., the pile cross diameter is about 20 % larger at 7 m depth as opposed to higher up the pile, which would mean about 680 mm as opposed to the reduced value of 570 mm for Gage Levels SGL3 and SGL2. As mentioned, for determining the axial load in the pile, the actual E-moduli and pile areas are irrelevant, only the stiffness matters. However, the difference between the gage levels is still important, because it translates to a corresponding difference in circumference, which is about 10 % for the Case 1 pile, and corresponding uncertainty in an evaluated unit shaft resistance distribution estimated using the nominal pile section.

The stiffness evaluation allows the transfer of all strain values to axial load in the pile. Figure 17 shows the load plotted as load distributions for the end of each load level. The figure also includes the pile-head load-movement curve.

The next step of the analysis of the test results was choosing a "target point" on the curve. By means of an effective stress analysis, this point was then used in determining the soil parameters, notably, the beta-coefficient that matched the load for the chosen gage depth.

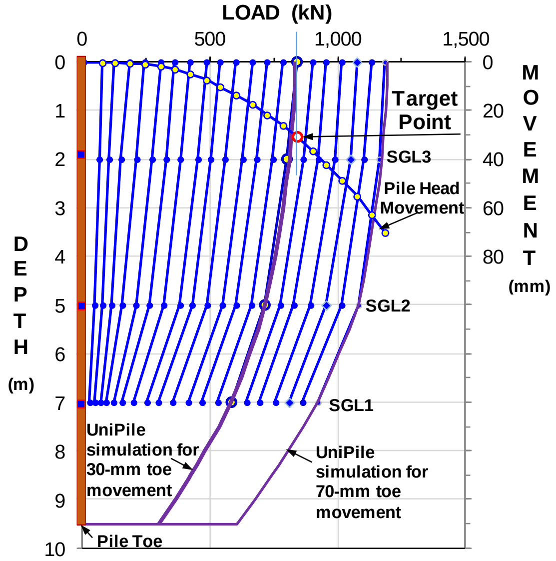

Figure 17 Case 1. Load distributions determined from the strain values and pile stiffness and simulated distributions for 30 and 70 mm pile toe movements

The toe resistances were determined by extrapolation of the load distribution curves from SGL1, that is, applying the same beta-coefficient in the layer between SGL2 and SGL1 to the soil between SGL1 and the pile toe. The figure also shows the pile-head load-movement curve with the "target point" and the load distribution for that "target load" as fitted to the strain gage determined loads in an effective stress simulation using the UniPile5 software (Goudreault and Fellenius 2014).

Figure 18 shows the results of fitting a UniPile5 simulation of the soil response to the measured values. The back-analysis simulation of load-movement curve employs so-called t-z/q-z functions to the target distribution loads, letting the t-z/q-z curves pivot around each target point. The iteration process started with the pile toe and SGL1 and proceeded up the pile.

The process of selecting the particular t-z/q-z functions that were used in the final fit is not important for the demonstration of the procedure. For details on the functions, see Fellenius (2019) and references therein.

The analysis procedure has established the pile response to load in terms of load and shaft shear distribution and pile-toe load-displacement response. The results will now enable, if desired, calculations pertaining to for a slightly longer/shorter, wider/slender pile and applying the analysis to the calculation of settlement for an applied load for a foundation supported on just a few piles or a wide group of similar piles at the site, as well as correlation to other test piles at the site or elsewhere, similarly analyzed. Note, settlement of a piled foundation is not the same as the movement of the test pile, but the latter is of value in determining the former.

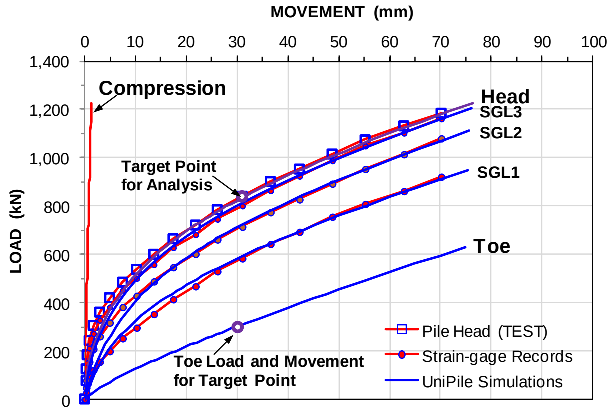

Figure 18 Case 1. Pile-head and strain-gage load-movement curves back-calculated and fitted from the target-points employing t-z/q-z functions

The analysis procedure has established the pile response to load in terms of load and shaft shear distribution and pile-toe load-displacement response. The results will now enable, if desired, calculations pertaining to for a slightly longer/shorter, wider/slender pile and applying the analysis to the calculation of settlement for an applied load for a foundation supported on just a few piles or a wide group of similar piles at the site, as well as correlation to other test piles at the site or elsewhere, similarly analyzed. Note, settlement of a piled foundation is not the same as the movement of the test pile, but the latter is of value in determining the former.

Note, the ambiguous and diffuse issue of "capacity", although amusing (see Figure 3), is only relevant in order to satisfy code requirements, it has no bearing—pun intended—on the foundation design and safe and serviceable response of the piled foundation to load.

3.2 Case 2

The analysis of Case 2 started with applying the direct secant method to the strain measurements at the two uppermost gage levels, SGL10 and SGL9 (at 1.9 and 4.9 m depths, respectively). The results are plotted in Figure 19 separated on values before and after the unloading/reloading event—Phases 1 and 2. Phase 1 records suggest a 31-GN average secant stiffness. The values from Phase 2 are obviously not useful. They are adversely affected by the unloading/reloading and the uneven increment sizes and load-holding durations, which resulted in erratic data not suitable for determining any strain-dependency.

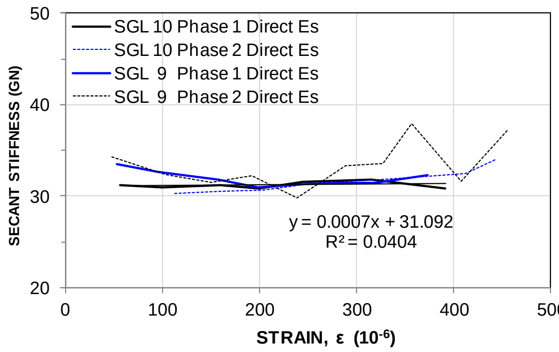

Figure 19 Case 2. Direct secant plot, SGL9 and SGL10

The tangent method was applied to the strain records, including the records of the SGL10 and SGL9 pairs are presented in Figure 20 for the individual gages at each level,, e.g., SG 10-1 represents gage No. 10 at SGL10. Similar to the direct secant method, the Phase-2 tangent method does not show anything useful for the records for. Moreover, Phase 1 records are not too distinct, either. The best one can state is that the records tend to agree with the direct secant method that the pile stiffness is close to about 31 GN.

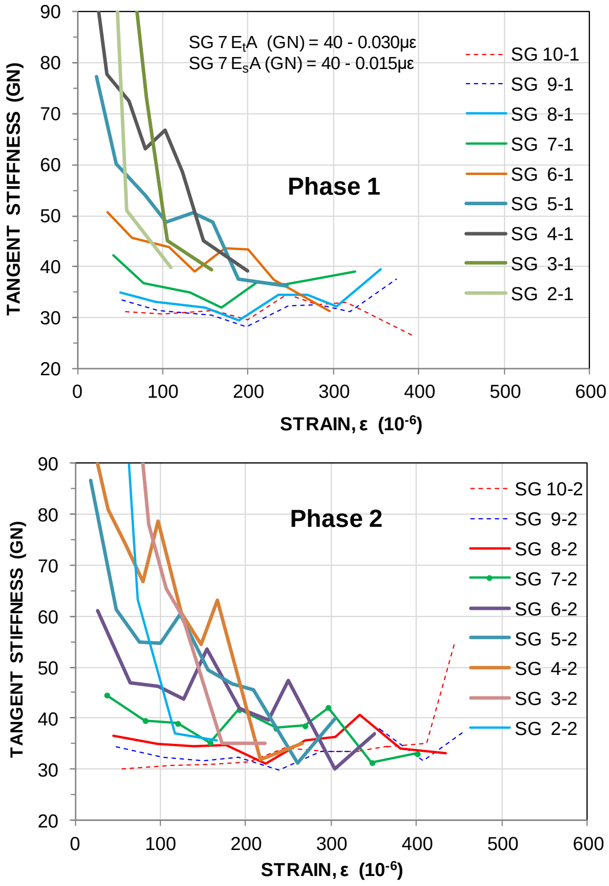

Figure 20 Case 2. Tangent modulus plot

We have applied a 31-GN stiffness to the strain measurements and converted all strains to load. Although the report lists measurements of strain for every 3 minutes of each load increment, the report does not provide any strain measurements at the start of the test—all records list zero strain before adding the first increment.

Ideally, the all strains that may or may not have occurred since lowering the gages into the ground should be measured to establish the strains present in the piles before the start of a test. However, for practical reasons, such strain records are rarely taken. However, the change of strain because of "seating everything" before starting the test (an undesirable action), or that from an accidental start-up with an increment or two that caused an unscheduled unloading and test re-start, etc. must be recorded, reported, and considered in the evaluation.

More disappointing is the fact that after unloading from the maximum load of Phase 1, no records were included of strain from between that time and the start of Phase 2. Moreover, before starting Phase 2, a 518 kN load was applied to the pile and the then recorded readings of strain were patently considered to be the "zero" condition at the "start" of Phase 2. Unfortunately, this meant that all the Phase 2 strain-gage readings are correlated to change from this unknown "zero". In contrast, the records of pile head movement and accumulated anchor movements were continued from Phase 1.

Figure 21 shows the load distributions calculated from converting all reported strains to load. Phase 2 distributions are only shown for loads larger than the maximum of Phase 1. The distributions are calculated from the strain values provided in the report. As indicated by the fact that the loads within the upper 10 m depth are larger than the applied load, the Phase 2 readings are affected by the unknown "zero" reference. This is a minor misrepresentation, however, the larger discrepancy between the shown distribution, comes from the fact that the pressed-in installation of the pile built in significant residual force in the pile, notably at the pile toe. The effect of full residual force would be that the true 2L9 distribution of shaft resistance along, at least, the upper about 25 m length of the pile could be about half only of the back-calculated distribution and the true toe resistance could be about 50 % (2,000 kN) larger than that calculated from the strain records.

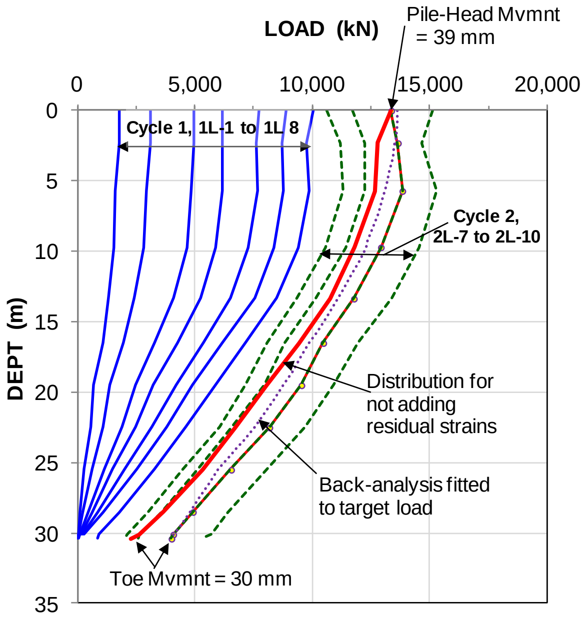

Figure 21 Case 2. Load distributions determined from the strain values and pile stiffness and simulated distributions for a 30-mm pile toe movement

Establishing agreement between the back-calculated distribution for the pile head and the strain-gage levels is a straight-forward approach. As for Case 1, the next step is a bit more laborious; achieving, by trial-and-error, the fit between all strain-gage loads and movements of the gage levels. Here very much assisted by the fact that the gage movement is a part of the Glostrext system. The results are presented in Figure 22. The fits are neat, but as indicated in the load distribution, the loads determined from the strain values are not very well established. This is because of the unknown residual strain in the pile, but also due to the uneven magnitude of the load increments and load-holding duration, and, to a larger extent, the unloading/reloading event. Much the pity, the Glostrext system of combining strain and movement is excellent for the purpose of evaluating the pile load-movement response, but it cannot here be applied to its full potential.

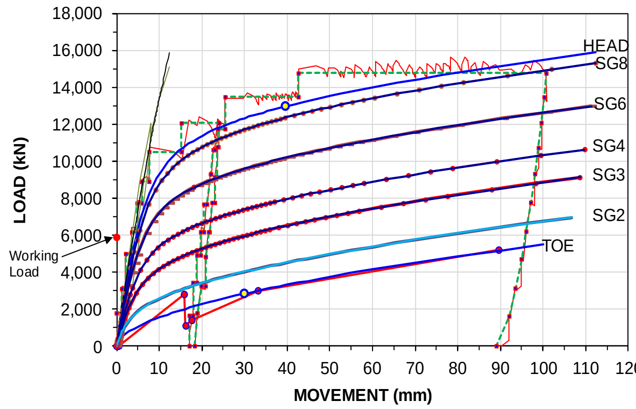

Figure 22 Case 2. Pile-head and strain-gage load-movement curves

3.3 Case 3

The results of the secant-method and tangent stiffness evaluation for Case 3 are shown in Figure 23. The unloading/reloading between Phases 1 and 2 has made both stiffness methods unfeasible for Phase 2 records. The direct-secant method applied to Phase 1 records for the uppermost gage level, SGL11, show an approximate linear-regression relation: $E_sA = 50 - 0.01\varepsilon$ (the strain to be input in units of $\mu\varepsilon$, 10-6). A 50-GN stiffness correlates to an E-modulus of about 44 GPa for the nominal pile section (1,200 mm; nominal area is 1.131 m^2), which is very large and, if true, would imply that the actual pile is considerably wider than the nominal value.

The secant stiffness of Phase 2 is about 10 % larger, the increase is consistent with reloading. It is likely that the seating and initial loading will have had a small similar increasing effect on the stiffness.

The tangent-stiffness method applied to the same gage records suggests no useable stiffness evaluation. The results of the tangent-stiffness method applied to the other Phase 2 records show an even worse scatter. The differentiation of the tangent stiffness method makes it very sensitive to errors in load readings.

Figure 24 shows the load distributions calculated from the strain records of Phase 2, applying the stiffness relation, $E_sA = 50 - 0.01\varepsilon$. The strain records measured for the 2L-9 load, 24,740 kN; were chosen for a fit of beta-coefficient in an effective-stress simulation using UniPile5. The calculations made no use of the values of applied load, only the strain records. The simulation gave the effective stress beta-coefficients shown for the three separate soil layers. The load distribution suggests that the shaft resistance is overestimated along the upper length of the pile and underestimated in the middle length. The toe resistance is clearly underestimated.

The figure also includes the load distributions provided in the test report as based on a stiffness equal to 38 GN for all measured strains (correlates to a 29-GPa nominal E-modulus) and extended to Load 2L-9. It is likely that the stiffness was taken by correlation to concrete strength or other. Evidently, the report accepted the implied huge shaft resistance between the pile head and the uppermost gage level, SGL11. However, even if one would assume a larger pile diameter potentially caused by use of a temporary casing for the construction, the main party of the discrepancy is more likely due to bending and friction in the bank of eight jacks used for generating the load may have exacerbated the overestimation of the applied load.

The strain-determined load distributions show that either is the applied load incorrect or the records underestimate the axial loads in the pile. Or both could be wrong. Both cannot be right, however.

It is quite possible that the pile shortening determined by the telltale measurement is affected by friction along the telltale—a very common case for long telltale rods. However, the telltale-measured shortenings agree with measured strains, thus serving as a duplicate measurement. It is possible that strain-gage calibration coefficient applied to the measured frequencies of the vibrating wire gages is not the correct one for the gages, as well as the agreement between the pile shortening and the strain values are coincidental. But, accepting the measured strain values, must mean that the applied loads are incorrect—too large, which indeed would be a troublesome conclusion.

The case demonstrates that not using a separate load cell to determine load, letting the jack pressure serve as aid toward applying the load and as a back-up measurement of the applied load is inadvisable. It is an omission, as it has been known for a long time that the jack pressure normally overestimates the load actually applied to the test pile (Fellenius 1984), although an error larger than 10 to 15 % is unusual.

Similar to Case 2, Case 3 illustrates the adverse effect on the evaluation of the test records, notably the strain-gage data, as caused by unequal load-holding duration and including unloading-reloading in the test. Indeed, it also make the importance obvious of always employing a separate load cell for determining the load applied to the pile head.

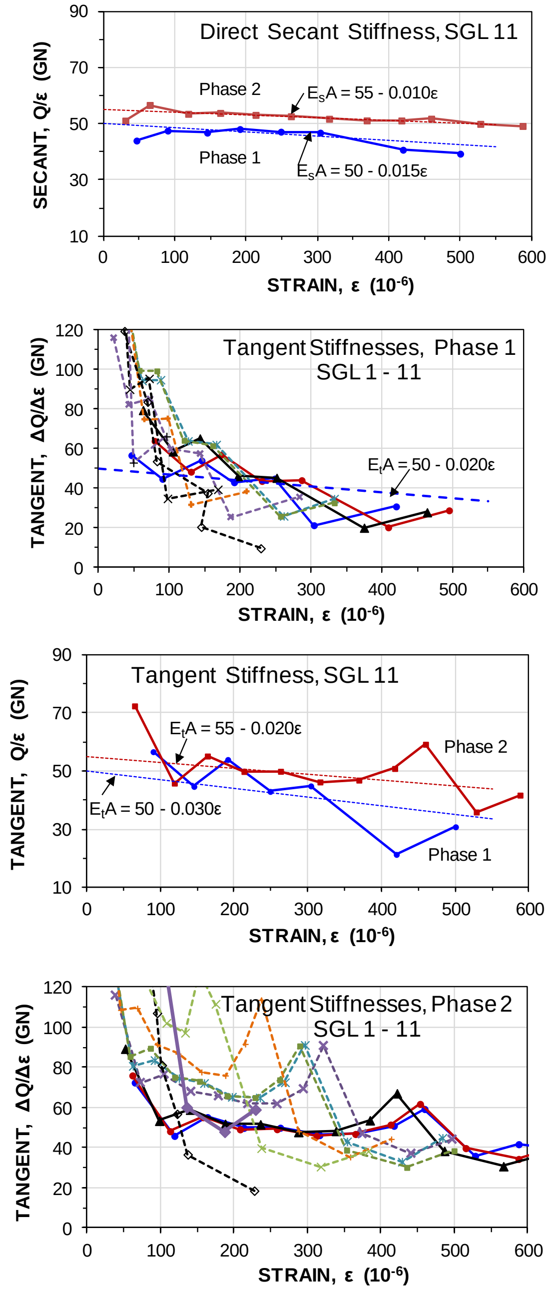

Figure 23 Case 3 direct secant and tangent stiffness plots

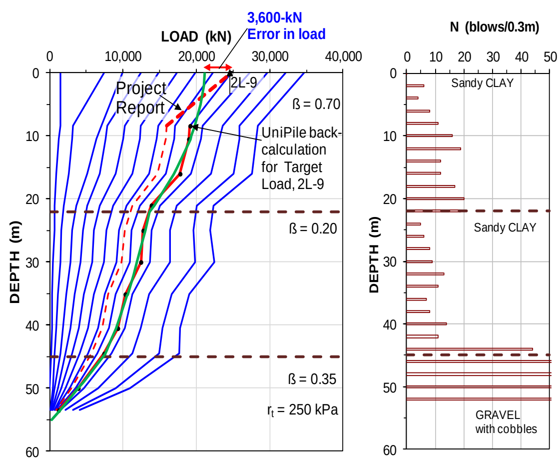

Figure 24 Case 3 load distributions with soil profile

3.4 Case 4

Case 4 test pile was equipped with 15 strain-gage levels, each comprising two pairs of Geokon vibrating wire gages. All gages functioned remarkably well albeit with slight erratics in the gage pairs below the BD level. As a representative demonstration of the consistency of the strain records, Figure 25 shows the records from Gage Level SGL8 (Pair H1 and H2 and Pair H3 and H4) 28.15 m below ground level. (Note that an unscheduled drop of the load during the 11th (next to last) load level is visible as a kink in the plot (See also Figure 8). The "kink" would seem small. However, the change of strain due to the drop of load represents about a quarter of the change of strain between the load levels, so it is significant.

The tangent-stiffness relation of the Case 4 strain-gage records are shown in Figure 26. Based on the response from the gage levels nearest the BD, the pile secant stiffness, E_sA is 42 - 0.005ε, which correlates to a nominal E-modulus of 26 GPa, a realistic value and more so than that determined for Case 3, for example. The shaft-shear-movement response of the soil surrounding the test pile, the t-z function, appears to be somewhat strain-hardening, which means that the tangent-stiffness method will deliver a stiffness response that is somewhat scattered and slightly larger than the actual.

Figure 27 shows the so-determined load distributions. The next to last applied load (1L-11) was chosen as Target Load and the corresponding distribution is identified in the figure.

The stiffness relation was applied to all strain-gage records.

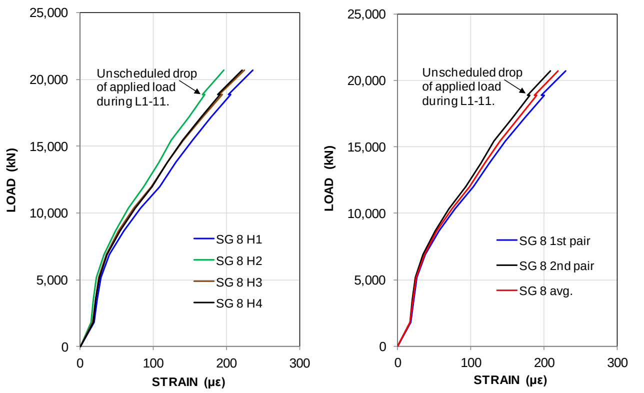

Figure 25 Case 4 Strain records at SGL8: single-gage, pair averages, and average of all gages

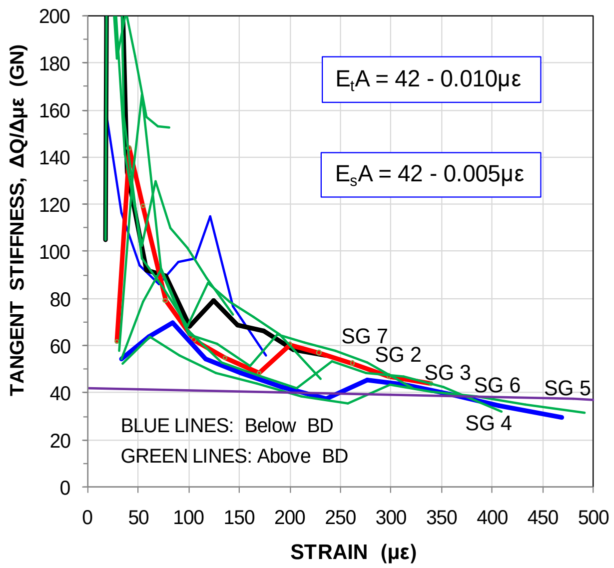

Figure 26 Case 4 Tangent-stiffness relations

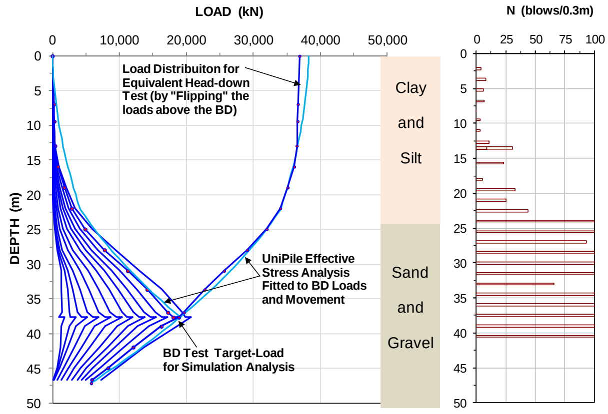

Figure 27 Case 4 Load distributions and soil profile

The BD load for the start of the upward distribution is adjusted to the buoyant weight of the pile. The equivalent head-down distribution is obtained by adding the "flipped" shaft distribution to the BD load. The simulated load distribution was obtained by fitting the distribution of axial loads for the target load as determined from the strain-gage records in an effective stress calculation using UniPile5.

The records do not include changes of strain due to pile construction and build-up of residual force in the test pile. For example, while it is possible that the shaft resistance was not fully mobilized between the ground surface and 13 m depth, it is unlikely zero, despite the strain records not indicating any shaft resistance to have mobilized in this zone. Our simulation has therefore allowed for some shaft resistance within this depth range. Moreover, residual load would have affected the distribution of shaft resistance further done along the pile. No adjustment for potential residual load is included, however.

The next step in the analysis was to fit simulated load-movement curves to the measured and not just for the BD load versus movements measured at the BD and at the pile head, but also for the load-movements at strain-gage levels, as shown in Figure 28. (The movement at the gage levels was not measured, but was estimated from accumulation of the measured strain and toe-telltale records).

The matching of not just load, but also movement at several depths in the pile add confidence the load distribution and soil response (i.e., beta-coefficients) represent the measured pile response. However, the fitting offers but little more than a confirmation and fine-tuning of the load distribution. The key advantage of a bidirectional test (BD test), such as Case 4, over a conventional head-down test is that the true load is known at two locations in the pile: the BD depth and at the pile head—the pile-head load is zero, but zero load is also a load. Thus, the load at the pile head for the equivalent head-down test and the load at the BD are two known facts, giving little leeway for variation in-between—provided that the soil profile is known. N.B., the BD load is not affected by residual force. Therefore, the bidirectional test can establish if, and to what extent, the strain-gage determined loads are affected by residual force in the pile.

For example, if no strain-gage records had been available for the analysis, a simulation of distribution for the target load down the pile would have been limited to fitting a distribution to the two known loads: the BD load and the pile head load—the latter being equal to twice the BD load minus the buoyancy-adjustment load. Figure 29 shows a resulting distribution determined from assuming uniform soil condition along the full length of the pile a beta-coefficient equal to 0.4 in the clay and silt layer and 0.8 in the sand and gravel layer. The figure also includes the distribution determined from the strain gages. The difference is not large. In fact, it may even be exaggerated due to the fact that the loads determined from the strain-gage records might include the influence of residual force in the pile (as well as the effect of the absence of knowledge of the strain at the start of the test). This demonstrates that the knowledge of the load down the pile, as provided by the BD test, is a key aspect of a back-analysis.

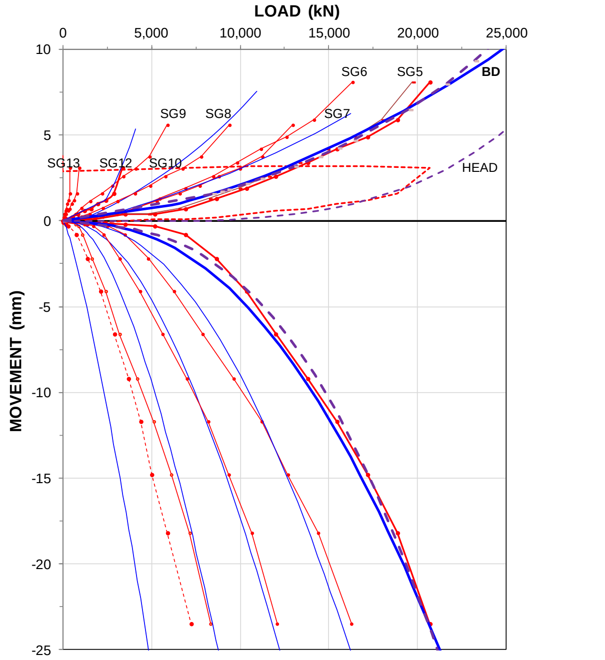

Figure 28 Case 4. Simulated load-movement curves fitted to curves calculated from measured strain and movement

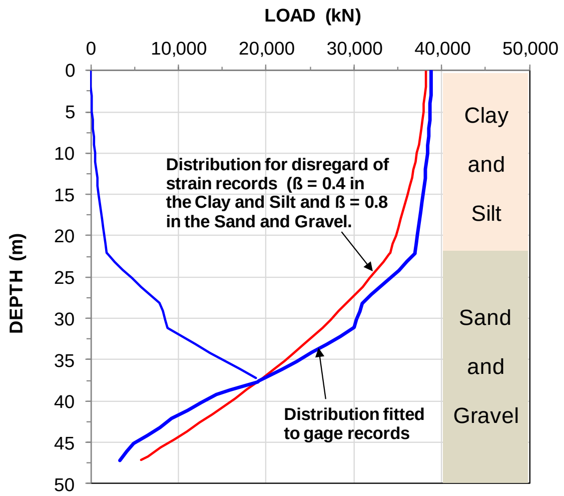

Figure 29 Case 4. Load distribution from fitting to BD load and strain-gage records compared to distribution from fitting to BD load and soil profile

The test report includes the evaluation of the unit shaft resistance from differentiation of the maximum BD load (1L-12) and as determined a the strain-gage levels. Figure 30 shows this distribution and the unit shaft resistance distribution at the next to maximum BD load (1L-11) from multiplying the beta-coefficients with effective stress (the difference in shaft resistance between 1L-11 and 1L-12 is small). The variation of the unit shaft resistance indicated by the differentiation of the strain-gage load is due mainly to variations in the strain measurement, not to a similar variation of soil shear strength along the pile. Moreover, this plot and the comparison emphasize the conclusion from previous figure, that a fit to the BD load and with adjustment to the soil profile establishes a reasonable shaft and toe resistance response. The gage records in a bidirectional test provide a refinement of the load distribution between the pile head and the BD level. Therefore, it is better to have just a few gage levels, but to have two gage-pairs at each gage level (as was used in the subject case) and add telltales to measure also the pile movement at the gage levels (using anchors as opposed to solid rods for optimum accuracy).

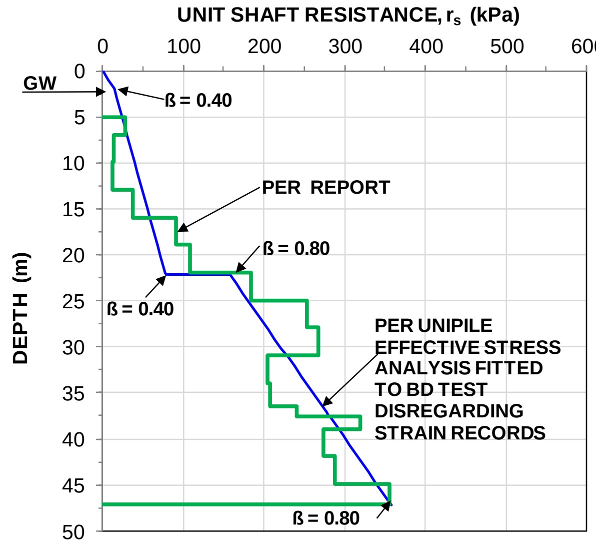

Figure 30 Case 4. Distribution of unit shaft resistance in the report and from fitting between the pile head and BD loads at the next to maximum BD load.

It is common to use the BD test to calculate the load-movement curve for an equivalent head-down loading test. The method includes adding the loads for equal upward and downward movements and plot these loads against the equal movement plus the calculated shortening of the pile due to the transfer of the downward load to the BD. The result of this construction is shown in Figure 31, plotted from the report. The figure also plots the head-down loading curve as calculated in the Uni Pile simulation that was fitted to the BD test (c.f., Figure 28). The simulation includes the effect of the fact that in a head-down test, the resistance of the upper, usually softer, soil layers, are engaged first and the lower usually stronger, soil layers are engaged last. In a BD test, the opposite occurs. Therefore, the direct calculation of the equivalent head-down test always returns a stiffer curve than that of a simulation of the test that considered the reverse stiffness engagement. If the equivalent curve is used to assess a pile, say by "capacity" based on a movement criterion, the "capacity" would turn out to be higher than that for an equivalent pile actually tested in a head-down test.

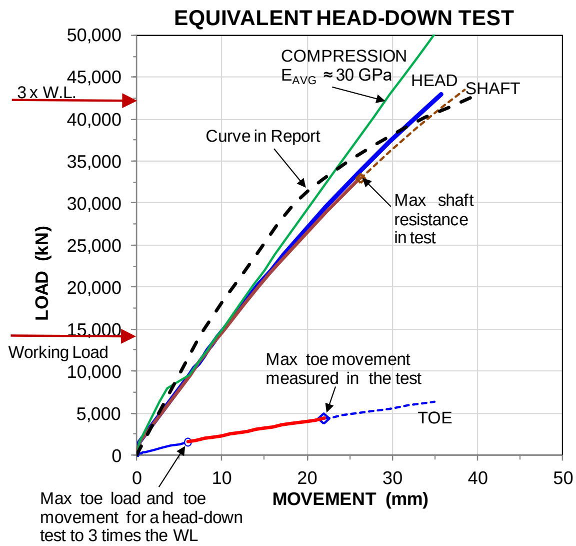

Figure 31 Case 4. The Equivalent Head-down Load-movement Curves determined directly from the test records and from the UniPile5 simulation

The more important message in the figure, however, is presented by the toe load-movement. Had the pile been subjected to a head-down test to three times the working load (WL), the pile toe would have moved just 5 or 6 mm, whereas in the BD test, the pile toe moved 21 mm. In a test to only twice the WL, the toe would not have been engaged at all. Inasmuch the response of the pile toe is critical for assessing a pile as constructed or for obtaining information to apply to a design of a piled foundation, this shows an important advantage of a BD test over a head-down test.

3.5 Case 5

The strains determined from the measurements of compression within the eight Glostrext anchor levels used to determine the axial stiffness of the pile. Table 1 shows the relevant geometric values. The Glostrext system is used to determine the average strain between the anchor points as measured shortening divided by the distance between the anchor points. That average, when converted to axial load, is usually thought appropriate to plot at mid-point between the two anchors.

Table 1 Anchor locations, depths to anchors, distance between anchors, and Gage Point depths

Anchor (#)	Depth to Anchor (m)	Distance between Anchors (m)	Depth to Mid of Anchors (m)
8	1.10	3	2.60
7	4.10	4	6.10
6	8.10	3	9.60
5	11.10	3	12.60
4	14.10	3	15.60
3	17.10	3	18.60
2	20.10	1	20.60
1	*21.10*	---	---

Anchors 8 and 7 were close to the pile head and could therefore be used in determine the secant stiffness directly. All levels were used for determining the tangent stiffness (for conversion to secant stiffness).

As shown in Figure 32, the resulting plots of the secant stiffness are not particularly in agreement with each other and the tangent stiffness only shows values for Anchor 8-7 (Gage Point between Anchors 8 and 7). The reason for poor relations lies in the imposed significant variation of the applied load and the varying load-holding duration and the loading-reloading sequences. Thus, despite the very precise measurements of strain and the constant pile cross section, the testing procedure has prevented using the strain records for determining the pile stiffness.

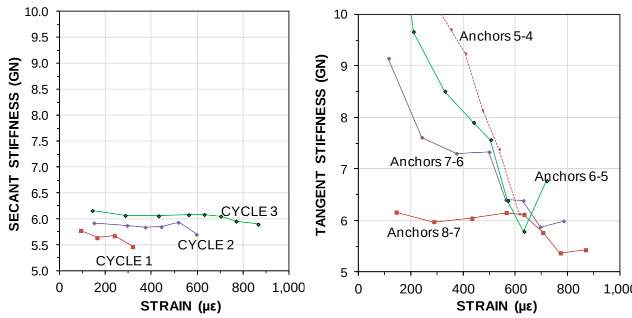

Figure 32 Case 5. Secant and tangent stiffness relations

An approximate relation between the measured strain and induced load was established by assuming that the load determined at Anchors 8-7 would be only slightly smaller than the applied load. This suggested a pile stiffness, EA, of 5.8 GN (also the stiffness presented in the test report). Dividing the stiffness with the pile cross section area indicates E-moduli of 44 GN, which is high for virgin loading of the pile (area stated to be 0.1303 m^2. The nominal values for the 500-mm and 280-mm outside and inside diameters are 0.1347 m^2). However, as mentioned, the loading was in reloading for which a higher E-modulus can be expected.

Figure 33 shows the load distribution calculated from the strain records applying the 5.8-GN stiffness. The figure includes the distribution presented in the report for the target load as developed by differentiation between the strain records at the seven levels and a distribution for the same values fitted in a UniPile5 simulation. The slight difference is due to the errors introduced by differentiation.

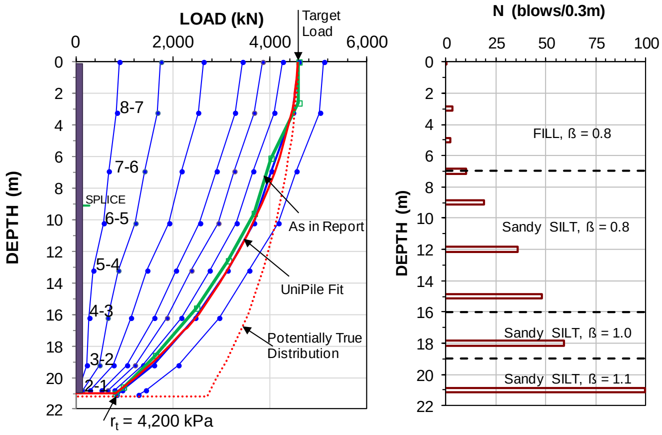

Figure 33 Case 5. Load distributions

The jacking-in installation left the pile with a locked-in distribution of axial load, i.e., residual force. Because the back-calculated axial load in the lower length of the pile diminished from one gage level to the next, it is obvious that, before the start of the test, the entire length of the pile was subject to negative direction shear forces. This means that the actual pile shaft resistance was smaller than the back-calculated value, perhaps only half of it and the pile toe resistance was, correspondingly, much larger than the back-calculated value. The dotted curve in the figure shows the potentially true load distribution after adjustment for residual force.

4. COMMENTS AND CONCLUSIONS

The review of the five test records has unearth several important issues of general validity.

The Load-time, and Movement-time records shown for the five case histories comprised lengths of testing time from less than a working day through more than four days and nights—3.5 hours to 100 hours. One wonders what could possibly have been gained—bought—by the investment of time beyond the few hours spent on the Case 1 short-duration test.

The fact is that nobody has evaluated results of static loading tests in terms of size of load increments, duration of load-holding, and effect of unloading reloading, such as were carried out for Cases 2 through 5. And, nobody has examined whether or not anything was gained by the extra time and effort spent on the loading tests that could have offset the loss of accuracy and analysis quality caused by the prolonged work. For tests employing unequal load increments, unequal load-holding, and unloading-reloading events, the attitude appears to be: *we did it last time, so why not keep on doing it*?

However, whether or not the three mentioned aspects are included in a test is not irrelevant—neither to costs nor to technical value of the test results, emphasized as follows.

The frequent use of the long test time is a residue from the times when the profession thought this would allow estimating the settlement of the foundation supported by the piles—indeed, the foundation assessment common before assessing "capacity" became fashionable. Thus, the measurements from unloading-reloading events was thought to infer the pile toe movement of the test pile. N.B., this was long before the importance of residual force in a test pile was realized. Until a few years ago, the state of practice did not have reliable means of estimating settlement of a piled foundation. We do now, however, and, while testing a single pile and recording its response to load will be useful in determining the expected settlement of a group of piles, this is true only if a proper test schedule is implemented.

In case of a routine static test, where only the applied load and the pile head movement are recorded, not much is learnt from the test other than "capacity", determined from some favored method—or that capacity was not reached with nothing else learnt. However, the five cases quoted are tests of instrumented piles measuring what happened in the test pile. To learn from those records, requires paying attention to how the tests were scheduled as well as, of course, on evaluating the strain measurements.

This means first of all that all load increments must be equal in magnitude, all load-holding durations be the same, and not even a single unloading-reloading event be included. N.B., one or two unloading-reloading events do not mean that a test is a "cyclic test". A cyclic test comprises a series of cycles between specific load levels. Such tests, expensive and time-consuming as they are, can have value for specific studies, say, seismic effects, but they do not provide the information usually desired or expected from a static loading test.

The results of the instrumentation records of the five cases manifest that the most assured way of spoiling a test and losing much, if not all, of the value of the instrumentation is to incorporate an unloading-reloading event in the test schedule. Of course, accuracy of readings, method of back-analysis of the records, correlation to the soil profile, etc. are also important, but these aspects are actually secondary to the three main rules of scheduling a static loading test: equal increment, equal time, and no unloading-reloading. Yet, as shown, even letting an applied load drop temporarily from an assigned constant load will have an adverse effect on the analysis of the strain-gage records. It is imperative that a device for automatic load (pressure) holding be employed in a test.

A head-down test must always measure load using a separate load cell passively recording the applied load. The hydraulic pump jack can be used in guiding the pressure in the hydraulic jack to reach the intended load, but the jack pressure must not be used as a measure of the load. It goes without saying that also the jack pressure should be recorded for later verification of the procedure.

A bidirectional test can make do with fewer strain-gages levels than a head-down test. Both test types will benefit from measurement of movement at the gage-levels (at least a few of them) by means of telltales. N.B., the anchor system is more reliable than a rod system.

All records of loads, movements, and strains must be by means of a data collector. However, it is necessary to have a single data collector, not two. Relying on "marrying" the record sets are after the test via the time-stamps of each set is not recommended.

Because movements of the reference beam can occur due to the unloading of the stress below the supports of the loaded platform or due to movement of the ground caused by the pull on anchor piles, it is necessary to also monitor the beam movements. Particularly so, in case of a head-down test.

Although not demonstrated in the here presented case histories, it should be recognized that the axial loads determined for an instrumented pile are affected by the presence and movement of reaction piles as well as the interference effect of any adjacent passive piles. N.B., single test piles and test pile inside a group of piles will not respond the same way.

A bidirectional test has the advantage over a head-down test of providing considerably more information of the response of the lower length of the pile, notably of the pile toe than does a head-down test. It has also the additional advantage of providing data more suitable for analysis of the soil response than does a head-down test, all other aspects equal. Moreover, it does not require a large number of gage levels, as frequently are used.

Converting results of a bidirectional test to an equivalent head-down test requires considering the fact that the head-down test engages the upper, usually softer soils, first, whereas the bidirectional test engages the lower, usually stiffer soils first.

The construction of a jacked-in (pressed-in) pile is by itself a static loading test. Therefore, the need for performing a static loading test on a jacked-in (pressed-in) pile can be questioned. However, when done, it must be recognized the test is performed in reloading and that the evaluated pile stiffness is larger than for a pile tested under virgin conditions.

A reloaded pile is a pile subjected to residual force and, unless so recognized, the evaluated shaft resistance distribution will be overestimated and the pile toe response underestimated.

The review has shown that an instrumented static loading test, be it a head-down test or a bidirectional test, performed, as it should, in a series of equal load increments, held constant for equal time, and incorporation no unloading-reloading event will provide data more suitable for analysis than a test performed with unequal increments, unequal load-holding, and incorporating an unloading-reloading event. The number of the equal magnitude increments should ideally be at least 12, preferably about 20. The load-holding duration can be any time length desired, as long as it is equal for all increments. However, even with many gages to records, it is only in very special cases, say a very long test pile, that each load-holding duration needs to be longer than 15 minutes. No useful information is obtained from prolonging the holding time for the maximum load. It is instead much preferable to make use of the margin established in preparing the test for increasing the applied load with one or more additional load increment. This is particularly simple to do in the bidirectional test.

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