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Fellenius, B.H., 2021. Results of an instrumented static loading test. Application to design and compilation of an international survey. Journal of the Deep Foundation Institute, 15(1) 71-87.

Research Paper

Results of an Instrumented Static Loading Test. Application to Design and Compilation of an International Survey

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Abstract: Results of a static loading test were used together with soil exploration records in a survey comprising analysis of the test records and estimating settlement of piled foundation to support a pipe rack. The test pile was a strain-gage instrumented, 400-mm diameter, precast, prestressed concrete pile driven into a clay and silt deposit to 25 m embedment. Two main issues were expected to be addressed by the survey participants: First, realization that the strain records were affected by presence of residual force in the pile and, second, calculation of the settlement of the piled foundation expected from the foundation load. A total of 52 submissions were received from 20 different countries. Only 12 of the submissions realized the presence of residual force. Most submissions reported a calculated settlement of the piled foundations ranging from 10 mm through 50 mm; however, 11 reported values between 60 and 200 mm. Surprisingly, only 20 submissions reported ground surface settlement close to the 200-mm value resulting from textbook analysis based on the available information. The subsequent construction of the piled foundations coincided with placing a fill across the site and lowering of the groundwater table, thus, causing a general subsidence.

Keywords: static loading test, strain-gage instrumentation, load distribution, general subsidence, pile settlement

Introduction

Ways of interpretation and application of results of a static pile loading test differ in the engineering practice. This is partly due to the influence of the particular code or standard having jurisdiction in the individual cases, but, also, I suspect, to differences in engineering culture. To study the differences more closely, I selected a test record from my files and disseminated the raw test data to friends and colleagues around the world asking them to evaluate the data and to apply them to the design of the project piled foundations.

The test pile was a strain-gage instrumented circular, precast, prestressed concrete pile driven into a clay and silt deposit to support a pipe-rack structure. Nominal pile diameter was 400 mm (16 inch—406 mm). The amount of reinforcement (number and size of strands) was not reported. Neither was the specified concrete strength, but it was probably 50 MPa. The test pile was driven to 25 m embedment depth about a month before the test. The piles for the intended project were to be the same as the test pile and were mainly single piles placed in rows (bents) of two to three piles at wide spacing with a few small pile groups comprising four or six piles at 3-diameter center-to-center (c/c) spacing. The loads were unfactored; 700 kN sustained and 100 kN transient (dead and live). The construction project required raising the site by a surcharge estimated to add a 15 kPa stress.

The soil information consisted of a soil profile description supported with a CPTU sounding log. The test data consisted of records of strains measured in at pairs of vibrating wire (VW) strain-gages cast in the test pile at grade and at every 5-meter depth down the pile. The pile compression was measured by a telltale from the pile head to the pile toe.

Unfortunately, the engineers responsible for the test omitted (neglected) to obtain measurements of strain prior to the test; neither securing records during the hydration of the concrete nor even at end of driving or just prior to the static test. The loading test was carried out by jacking against a kentledge (loaded platform). The test set-up included no independent check on reference beam movement. In other words, the test record quality is like those obtained in most routine tests.

The survey invited the participants to do the following:

1) convert the measured strains to load,
2) plot the load-movement and load-distribution curves,
3) calculate the long-term settlement of the ground and the piled foundation,
4) give an opinion on whether or not the static loading test records of strain and movement represent the true response to the applied loads, and
5) give an opinion on whether or not a pile, same as the test pile, would be acceptable as a foundation pile for the

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Print ISSN: 1937-5247 Online ISSN: 1937-5255
Published by Deep Foundations Institute
Received 09 September 2020; received in revised form 27 March 2021; accepted 14 April 2021

mentioned loads or if there was reason for change—considering the fact that the depth to a bearing layer would in most places be quite different, indeed deeper, than that at the test pile location, and that the permissible long-term settlement is 30 mm.

The following presents the test records as disseminated, followed by a full analysis with a discussion of the strain values in regard to the demonstrated presence of residual force in the test pile. The responses received from the participants in regard to Items (1) through (5) are then compiled and discussed.

Disseminated Records

Table 1 includes the basic soil parameters disseminated to the survey participants. The soil consolidation compressibility is expressed in Janbu modulus numbers for virgin compression (m) and for recompression (m_r) obtained from oedometer tests (m = ln10/CR, where CR is the Compression Ratio). The even 5-m thickness of the soil layers is estimated, and densities are average values. Density of the silty sand may be shy of actual, although it would indicate ≈30+ % water content, which is far too large for a sand albeit silty. Soil type interpretation from the CPTU suggests the upper 5 m to be sandy silt. The groundwater table at the test location was at 1.0 m depth and the pore pressure distribution was hydrostatic. Water mining in the lower sand subsequent to the pile installation will lower the water table to 2 m depth; a permanent feature of the site. The disseminated information included that of placing the fill across the site.

The disseminated records of the static loading test are presented in Table 2. The jack-imposed loads were monitored by a load cell. The strains are the strain-pair averages.

Figure 1 shows the CPTU diagrams from a sounding pushed close to the test pile. The test pile was intentionally driven to terminate at a minimal embedment into the lower sand layer at 25 m depth. No pile driving records are available.

Table 1. Basic soil parameters

Depth (m)	Soil (Type)	Density ϱ_t (kg/m³)	Compressibility: Precons. Margin, Δσ' (kPa)	Compressibility: Virgin m (–)	Compressibility: Recompr. m_r (–)
0-5	Silty Sand	1,900	0	180	
5-10	Clay	1,600	0	15	
10-15	Clay	1,600	0	20	
15-20	Clay	1,600	10	30	300
20-25	Clay	1,600	30	30	300
25-30+	Sand	2,100	100	300	300

Settlement Analysis

The adding of fill and lowering of the groundwater table will cause the soil to settle. Immediate settlement in the soil from the fill placement can be expected to develop as the load is placed on the piles and, therefore, be disregarded. The settlement will probably occur quickly in the upper silty sand (sandy silt) layer and in the lower sand but take considerable time to develop in the 20 m thick clay layer in between. However, no information on the coefficient of consolidation is available—the consolidation settlement may well take 20 to 30 years to develop and might result in downdrag for the piled foundations. Neither is any information on secondary

Table 2. Records of applied load and measured strains and movements

Increment (#)	Load (kN)	Mvmnt Head (mm)	Compr. (mm)	SG-6 (με) depth 0	SG-5 (με) depth 5	SG-4 (με) depth 10	SG-3 (με) depth 15	SG-2 (με) depth 20	SG-1 (με) depth 25
L0	0	0.00	0.00	0	0	0	0	0	0
L1	200	0.67	0.67	51	45	20	9	4	0
L2	400	1.41	1.41	103	89	52	26	10	1
L3	600	2.37	2.12	161	134	85	43	18	2
L4	800	3.53	3.00	209	173	125	73	27	6
L5	1,000	4.54	3.74	267	225	166	105	39	13
L6	1,200	6.62	4.63	315	277	212	137	59	23
L7	1,400	15.67	5.67	376	330	264	166	69	42
L8	1,600	36.54	7.04	419	383	318	218	117	75
L9	1,800	62.74	8.24	480	436	371	270	169	130
L10	2,000	94.65	9.65	528	489	424	324	221	180
L11	2,200	127.00	11.40	586	542	477	377	273	237

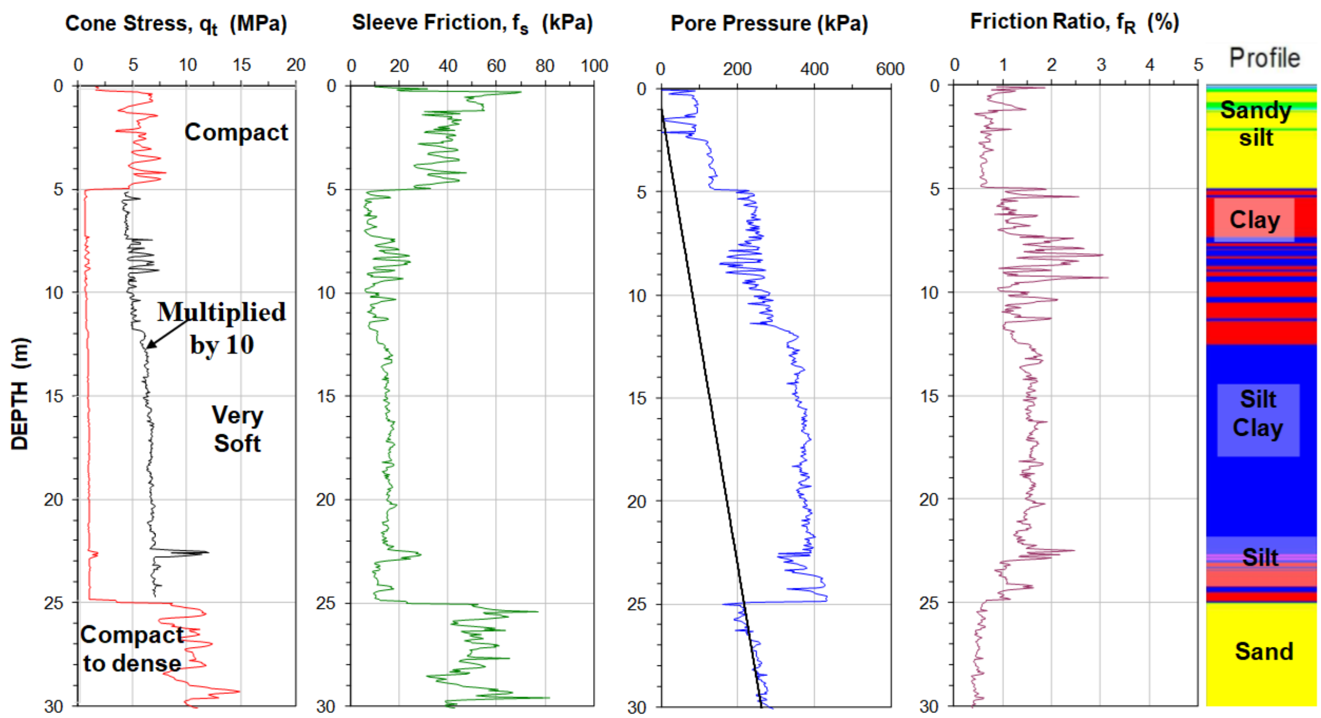

Figure 1. Records of a CPTU sounding pushed at the test pile location

coefficient of compression available, but it is assumed insignificant in relation to the consolidation settlement.

For single piles, because the pile spacing is wide (larger than 10 diameters) and the distance between the pile-bents is wide, the load on the piles will only develop load-transfer movement, not cause settlement below the pile toe level. For the narrow groups, some settlement will develop below the pile toe level.

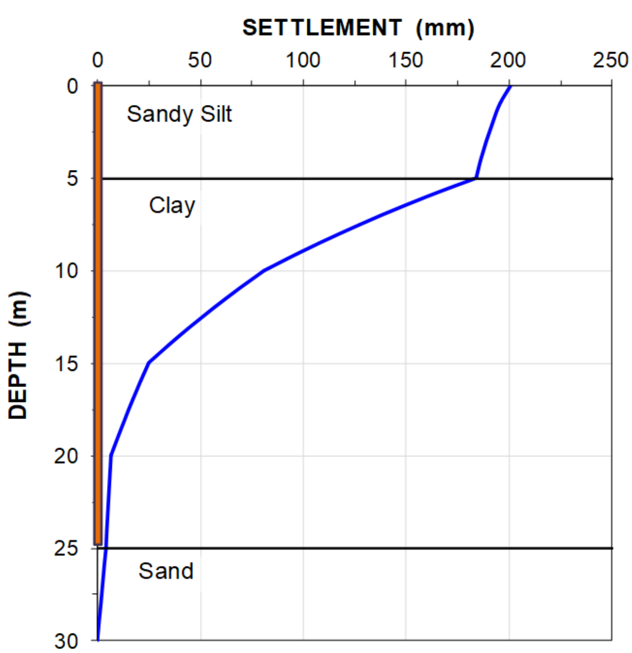

Figure 2. Distribution of soil settlement

Figure 2 shows the distribution of soil settlement due to the increase of effective stress calculated using the parameters in Table 1. The calculation shows that the area surrounding the piled foundations will settle 200 mm. As mentioned, initial compression can be assumed unimportant as it will occur during the construction of the pipe-rack. The settlement of the piled foundation is equal to the settlement of the soil at the depth of the force and settlement equilibrium, the neutral plane (N.P.), determined from analysis of the long-term pile response to load and the pile load-transfer movements available in the results of the static loading test. The analysis proceeds from back-calculation of the test records and then projecting the results to the long-term conditions, as demonstrated in Section 4.

Analysis of the Strain-Gage Records

Pile Axial Stiffness, EA

The analysis method employed in the analysis of the test records and used in applying the test results to a foundation design follows the procedures described in Fellenius (1989; 2021). The first step of analysis comprises converting the strain, ε, measured in the loading test on the instrumented pile, to axial force. This requires knowledge of the pile modulus, E, and cross-sectional area, A. The pile axial force, Q, is equal to $EA\varepsilon$ and the EA-value is best determined from the strain records. The most common method is to plot the load-strain records and determine the slope, EA, of the load-strain curve. The curves will be affected by the shaft resistance, of course, but for the uppermost gage level (SG-6). If the shaft resistance is not plastic, but strain-hardening or strain-softening, the slopes of the SG-1 through SG-5 curves would trend to be steeper or less steep, respectively, than the gage level nearest the pile head (where there is no shaft resistance). Any length of pile in a layer showing a non-plastic response would have caused the records of strain-gage levels below the layer to become less representative of a true axial pile stiffness.

Figure 3 shows the plots of applied load versus measured strain for the six gage levels. The curves show approximately parallel slopes beyond about 1,500 kN of load, which suggests that the shaft resistance response is plastic, i.e., neither strain-hardening nor strain-softening. The slopes indicate an average pile stiffness, *EA*, of 3.8 GN/m.

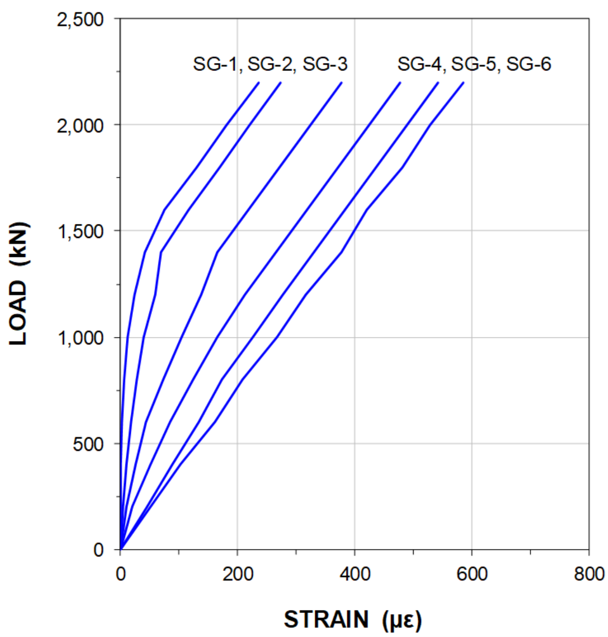

Figure 3. Load-strain measured at the strain gage levels

The uppermost gage level, SG-6 is unaffected by any shear force between the gage level and the pile head (where the load is applied). Therefore, in analyzing the strain measured at SG-6, the "direct secant method" can be used, which consists of plotting all loads, Q, divided by the strain, ε, versus strain. The method determines the stiffness from a plot of load/strain versus strain and this plot is more precise than the load versus strain plot. However, the method also requires that the strain records are referenced ("zeroed") to a true no-force condition.

Gage levels that are affected by shaft resistance are not suitable for analysis by the "direct secant method" and must be analyzed using the "tangent method". It consists of plotting all load increments, ΔQ, divided by the so-induced change of strain, $\Delta\varepsilon$, versus strain, ε. Because the tangent method relies on differentiation, it is independent of the accuracy of the "zero" reading. However, it is very sensitive to a non-plastic (i.e., strain-hardening or strain-softening, response of shaft resistance) and it also depends very much on the accuracy of the each of the values of load and strain.

Figure 4 shows the plots of the direct secant (SG-6) and tangent methods (SG-1 to SG-6). The agreement between the two methods for the strain records indicates that the records are referenced to a true zero value (secant method) and that the shaft resistance along the pile is essentially unaffected by strain-hardening or strain-softening (tangent method). The plots show the pile axial stiffness, *EA*, to be 3.77 GN/m with no reduction with increasing strain. Back-calculating using the nominal 16-inch diameter, the *E*-modulus is 4,200 ksi (29 GPa) and, using the 400 mm nominal diameter, it is 30 GPa. However, the actual area and modulus do not matter, it is the *EA* that matters for the evaluation. The strains were converted to axial force by using *EA* = 3.8 GN.

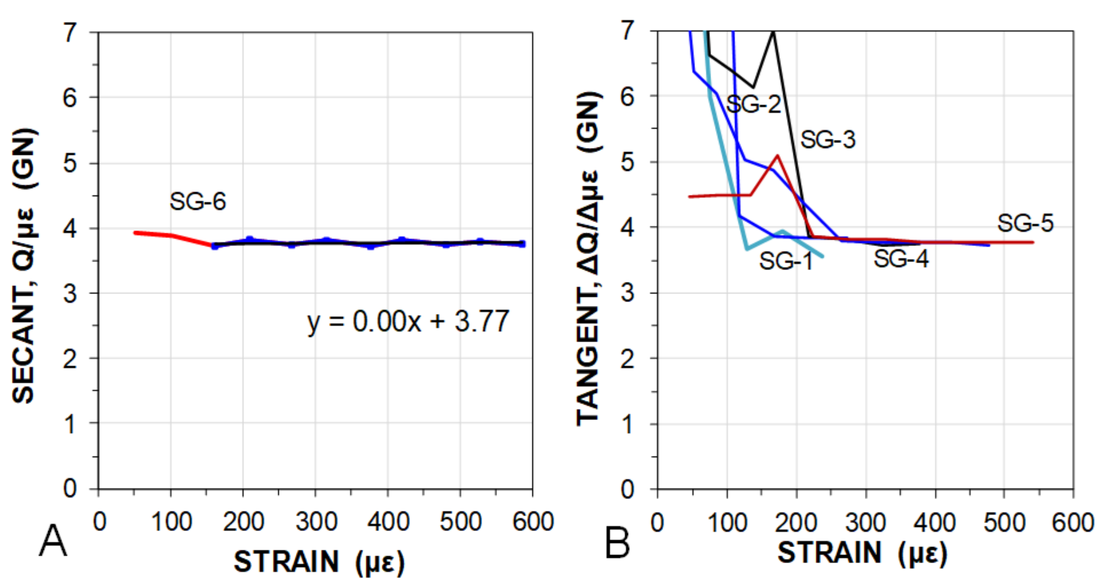

Figure 4. The direct secant and tangent stiffness plots

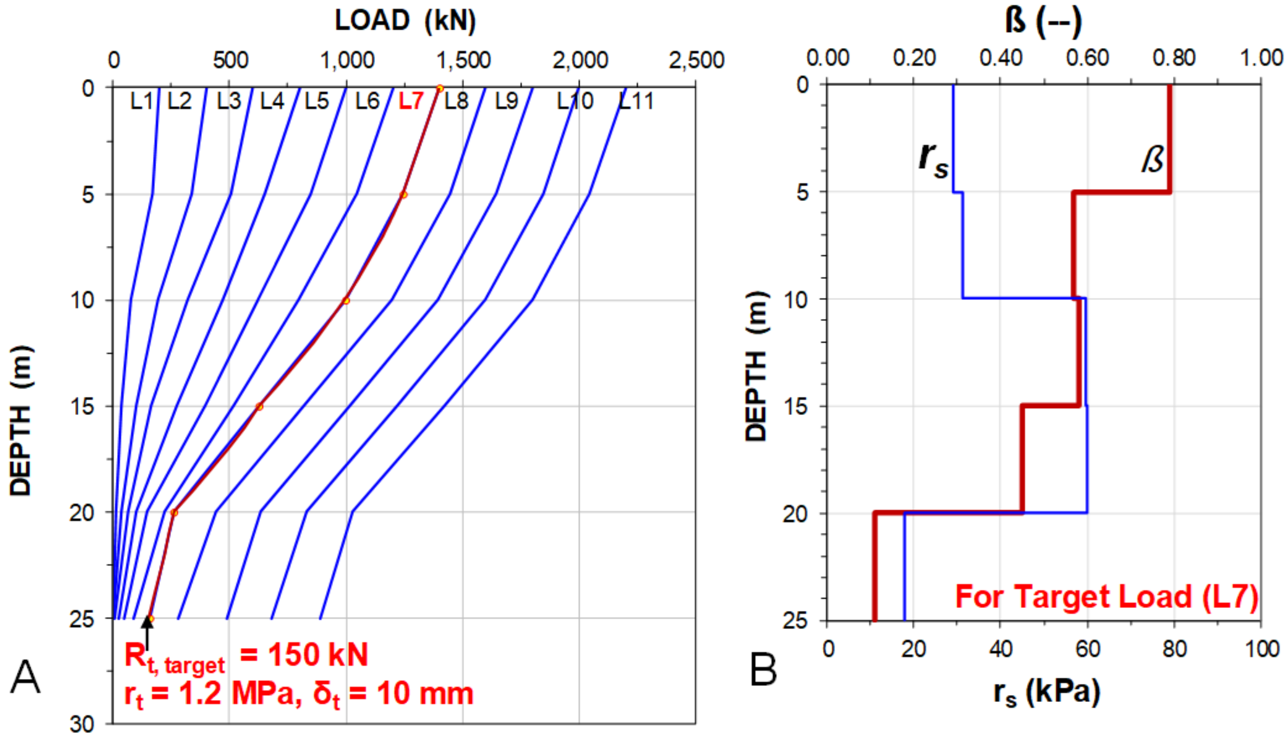

Figure 5. (A) Load distributions measured in the static loading test, (B) Distributions of ß-coefficient and unit shaft resistance

Determining Axial Force Distributions from Measured Strain

Figure 5A shows the distributions of axial force load for each value of load applied to the pile head. The records of Load Level 7 (L7) were chosen as Target for an effective stress back-analysis that gave the distribution of an effective stress beta-coefficient for each soil layer and the pile toe resistance, R_t, mobilized for the L7 applied load. Load Level L7 was chosen as Target because it had resulted in an appreciable toe movement. Probably L6, or L8 and, maybe, L9 could have been chosen equally well. However, the large movements imposed by L10 and L11 would make those records less suitable for the later load-movement analysis. Figure 5B shows the distribution of *ß*-coefficients and the correlated unit shaft resistance, r_s, for the Target Load. As noted in the figure, the pile-toe movement, δ_t, measured for the L7-load was 10 mm.

For comparison, Figure 6 shows four load distributions calculated from the CPTU records employing the methods by Eslami-Fellenius, Schmertmann-Nottingham, deRuiter-Beringen (Dutch), and Bustamante-Gianselli (LCPC) (Fellenius 2021). The methods are ostensibly calibrated to "capacity" evaluated from static loading tests and the "capacities" used in establishing the methods depend on the particular definition of "capacity" applied. Quite simply, therefore, a "capacity" determined from CPT-records is notoriously imprecise. Note also that although the CPT-determined total "capacities" are close to the assigned 1,400-kN Target Load, the latter does not represent an ultimate resistance.

The next step of the analysis was to determine the force-movement response of the pile elements and the pile

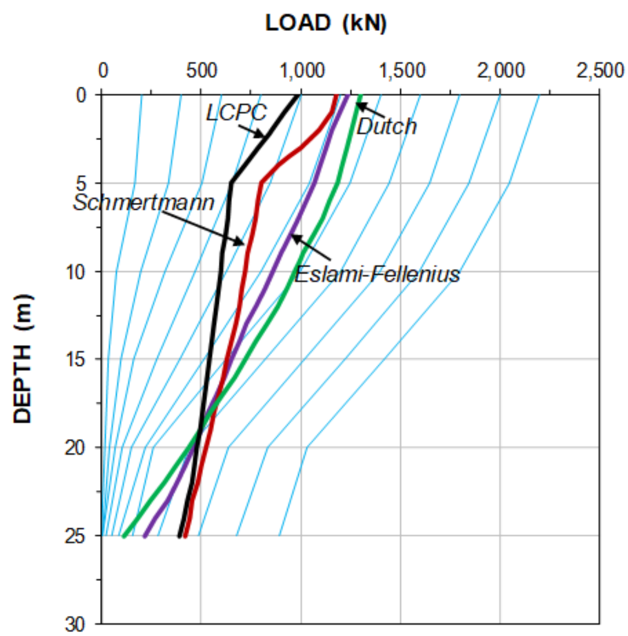

Figure 6. Load distribution at CPT-determined "capacity"

toe. Obtaining a match between a simulated pile toe response (q-z function) was the first step and it consisted of varying the function coefficient to let the q-z function pivot around the toe resistance for the Target Load. As is typical for a pile-toe response, a Gwizdala type function showed to be the one that best simulated the measured toe force (SG-1) and telltale-measured toe movement for the applied test loads. The next step was to simulate, by a similar procedure, the response of the next-up gage level, Level SG-2, etc. Figure 7 shows the two t-z functions and the q-z function that gave a good match of simulated to measured response (c.f., Figure 8). For each gage level, the 100-% value is the load that was measured for the 1,400-kN Target Load, L7.

Figure 8 shows the loads plotted versus the pile head movement (the red circles). The solid blue curves are the load-movement curves as back-calculated using UniPile5 (Goudreault and Fellenius 2014) employing the t-z and q-z relations. The solid red line with the open circles shows the pile-toe force versus pile-toe movement.

Once the test results and the pile and soil parameters have been used to back-calculate (calibrate) the pile response to load, it is simple to add the effect of the fill and the lowering of the groundwater table to simulate the pile response to load for the long-term conditions. The simulation applies the effective stress parameters, i.e., the beta-coefficients and the t-z and q-z functions obtained from the back-analysis of the test records. Figure 9 shows the long-term pile-head load-movement curve and, for comparison, also the test curve. The analysis has assumed that the pile-toe movement in the long-term is the same as during the test. Thus, the increase of pile stiffness for the long-term curve is entirely due to the increase of effective stress along the pile shaft.

The most common approach in engineering practice when assessing a pile foundation is to determine a "capacity" from the pile test load-movement curve, apply a factor of safety or a resistance factor, and let that determine whether or not a desired load ($Q_{dead} + Q_{live}$) is acceptable. As the site in this case shows an improved stiffness with time due to the increase of effective stress, the short-term condition governs the design. The short-tem is the condition of just completed construction and it is represented by the test results. However, if the site would have been subject to a lowering of the effective stresses (due to, say, an excavation and/or rise of the groundwater table), the long-term curve would plot below the test curve. Then, that latter curve, determined similarly, is the one to consider in the assessment of the piled foundation.

For the subject case, one can apply one or more of the many different methods for determining a "capacity" to the pile-head load-movement curve and, then, reduce the value by a locally prescribed factor to see if the so-determined allowable (or factored) load would be larger or smaller than the desired total load, the 800-kN load assigned to the pile. Some methods will show that 800-kN is safe and others that it is not. That is, the methods employed in practice vary considerably (Fellenius 2017) and, therefore, also the answer to a question about whether or not the assigned load is 'safe' is totally "in the eyes of the beholder". It is beyond the purpose of this report

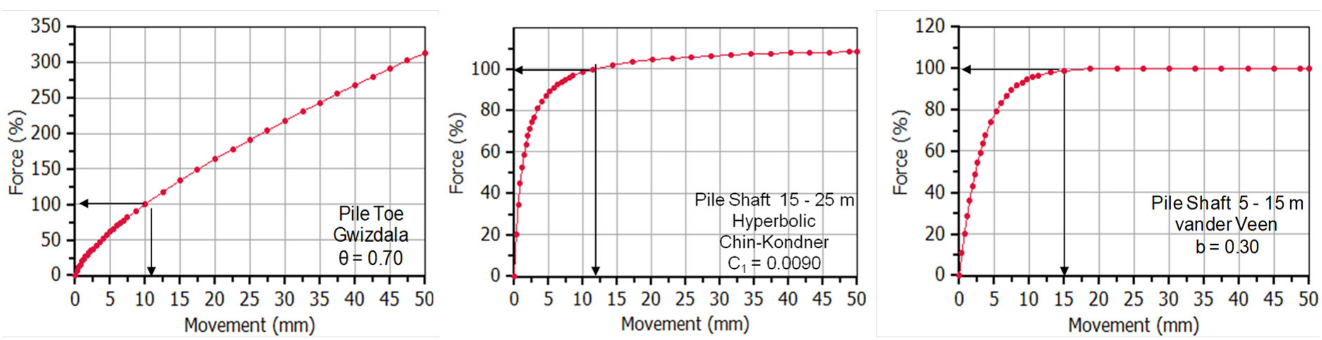

Figure 7. Force movement functions for the pile elements (t-z) and the pile toe (q-z)

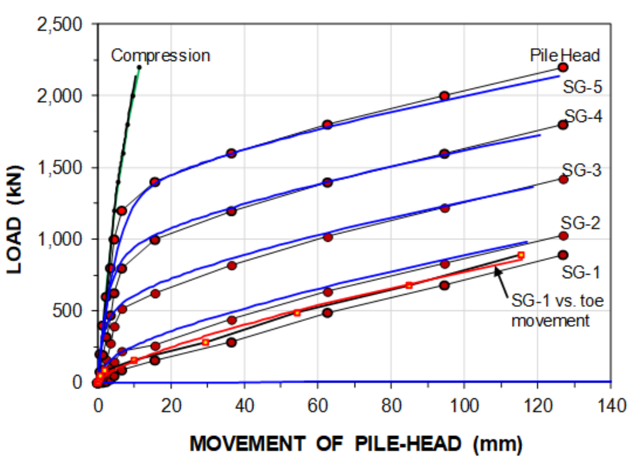

Figure 8. Applied load versus movement

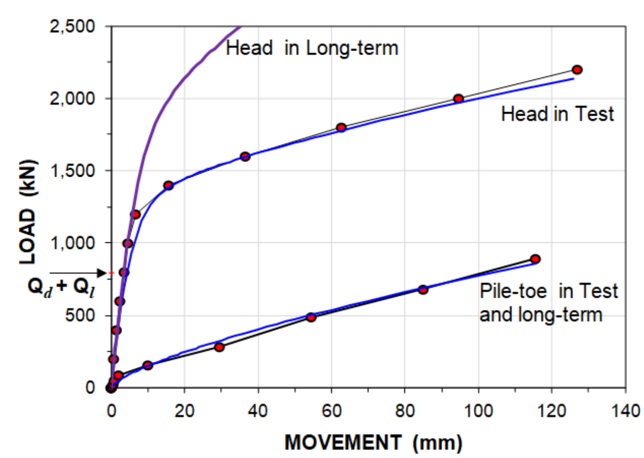

Figure 9. Static loading test curves for the test and for long-term conditions

to select one or other of the methods or to express an opinion on the potential relevance and usefulness of one or the other.

The assessment of the settlement for a single pile (or a narrow pile group, or perimeter piles of a wide group) requires having determined the settlement of the soil (Figure 2) and the long-term response of the pile in terms of load distribution, particularly of the pile-toe response. This is addressed by applying the Unified Design Method of analysis (Fellenius 1988, 2021). The foregoing analysis of the test records has established all the necessary information.

The Unified Method of Analysis

The Unified Design Method (Fellenius 1984; 1988; 2021) involves determining two graphs, as illustrated in Figure 10. The left graph shows the long-term load distribution for a pile toe force and the right shows the distribution of soil settlement and pile settlement. The force and settlement equilibriums (neutral plane, N.P.) are indicated as determined by the toe force development, matching the toe force in the load distribution graph and the toe movement in the settlement graph. The long-term distribution needs the analysis to consider the increase of shaft resistance due to the consolidation settlement and pile toe movement. Retaining the back-calculated beta-coefficients, the increase will be proportional to the increase of effective stress. While simulating the load distributions of the test pile can equally well be carried out using the stress-independent approach (α-method) as using the effective stress analysis (ß-method), determining a similar increase by applying improved values of shear resistance due to consolidation and, at depth, reduced preconsolidation margin, is considerably more complex and fraught with uncertainty. It is more convenient to retain the back-calculated ß-coefficients and apply the increase of effective stress due to the fill and lowered groundwater table.

The graph to the right shows the calculated distribution of soil settlement together with a settlement equilibrium correlated to a pile toe movement that corresponds to the pile-toe forces applied according to the q-z function for the pile toe (left graph). Of the many equilibrium depths possible to develop in each of the graphs, only the two shown in the figure occur at the same depth in both graphs. For details of the procedure, see Fellenius (2021). The figure indicates that the long-term settlement of the single pile will be 60 mm, which is the end result of the analysis.

Analyzing the settlement of a pile group is slightly more complex. In addition to the load-transfer movement, a pile

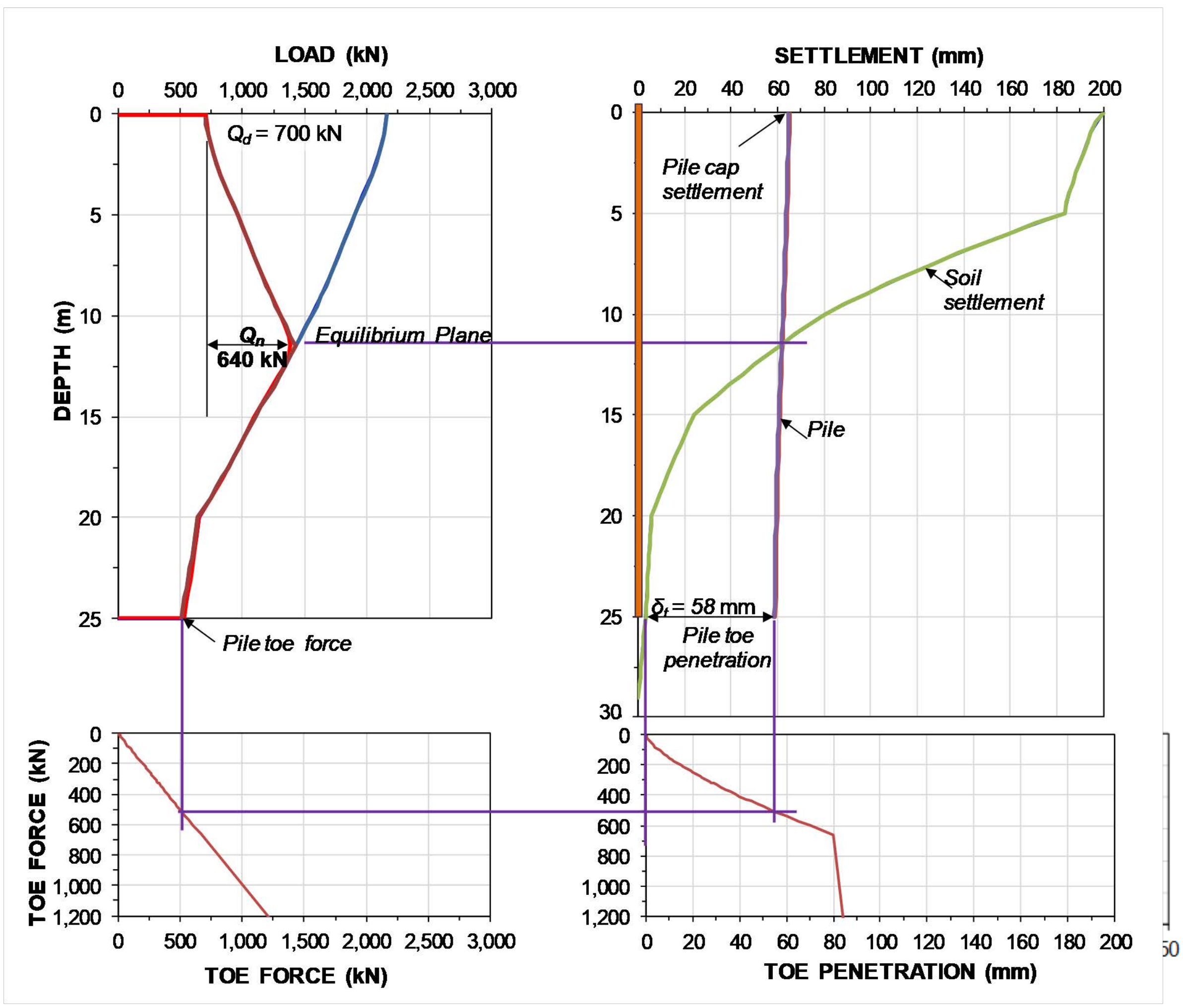

Figure 10. The unified method for determining the settlement of a single pile

group is also affected by the compression (settlement) of the soil below the pile toe level. The process can be modeled as an equivalent raft placed at the pile toe level and loaded with a uniformly distributed stress equal to the sustained load applied to the pile cap. The settlement and its distribution over the pile group area is then calculated conventionally and added to the load-transfer movement and the compression of the piles due to the distribution of axial force.

The method of equivalent raft analysis is different for a wide pile group to that of a narrow group. A wide pile group is a group with four or more piles along its breadth. The response of the perimeter piles in a wide group is similar to that of a single pile. However, the interior piles will transfer load by shaft resistance along the perimeter piles and from the pile toe level upward. Settlement of the interior piles will not be affected by downdrag (Fellenius 2019, 2021). As the survey project does not include any wide pile groups, this account is limited to the narrow group analysis.

The settlement of small (narrow) pile groups is affected by the transfer of the applied load to the soil, which starts at the N.P. A simple approach is to consider the load on the pile cap as placed on an equivalent raft at the N.P and then, calculating the settlement for a so-loaded raft with due reference to the fact that the pile axial stiffness has greatly reduced the compressibility of the zone between the N.P. and the pile toe. However, between the N.P. and the pile toe level, the axial force in the piles will reduce due to shaft resistance and the transfer will widen the zone affected by the applied load at and below the pile toe level. The rest of the load goes unencumbered to the pile toe. This makes for an awkward and uncertain modeling and analysis.

A simple model of the condition is assuming that the effect of the applied load reaches an equivalent raft placed at the pile toe with the dimension of the pile cap plus a widening calculated as the distance between the N.P. and the pile divided by 5, thus distributing the load at (1H)/(5V). (A wider spread would result in a too large area affected at the pile toe level. See Clause 7.17.2 in Fellenius 2021). The subject 4- and 6-pile foundations would thus have equivalent raft areas of 6.0 × 6.2 m and 6.0 × 7.2 m and uniform stress of 80 kPa and 100 kPa, respectively. A calculation of the effect of the so-loaded piled foundations does not change the calculated depth to the N.P., but it will indicate that the 4- and 6-pile foundations will settle about 10 mm more than the single-pile foundation.

Comments

In areas that are under the jurisdiction of obsolete codes, the realization that the pile will be subjected to a 900-kN drag force will likely make many a designer lengthen the project piles to add "capacity". However, this would increase the drag force—to reduce the drag force, one would either have to shorten the pile or increase the assigned dead load, a paradox that those looking at a drag force as something akin to the bogeyman-under-the-bed prefer to disregard. The pile will have no difficulty in resisting the combined 1,600-kN axial force. Indeed, the drag force is inconsequential for the design. Actually, the larger the drag force, the stiffer and better the pile. It is the settlement due to downdrag that is of concern for a piled foundation.

Before deciding on the design, however, it is necessary to sit back and evaluate the analysis. First of all, an experienced engineer would be alerted to the unrealistic load distribution graph (c.f., Figure 5). The steepening slope of each of the load distribution curves in the lowest length of the pile as opposed to the slope above indicates a unit shaft resistance (r_s) that reduces with depth, as is confirmed by the right-side graph in Figure 5, showing the distribution of the stress-independent unit shaft resistance and the effective stress beta-coefficient. The distributions can well be true for some conditions. However, for the subject case, the soil profile description and the CPTU diagram indicate that the soil below the upper 5 m is uniform. In a uniform soil, such as at this site, the slope of the load distribution should flatten with depth, that is, the ß-coefficient should be more or less constant with depth and the unit shaft resistance be increasing with depth. The appearance of a shaft resistance reducing with depth is, therefore, false. It is due to axial force in the pile not being zero at the start of the test (as so often is assumed), which force is termed "residual force".

The presence of residual force is a quite common situation. Unfortunately, it is often disregarded in test analyses, as exemplified in Figures 11 and 12, showing published cases demonstrating partial to full loss of shaft resistance in the last about one third to one quarter of the pile embedment. Judging from the CPT-diagram in Figure 12, the true shaft resistance would have been expected to instead increase with depth. Many more examples exist, a few of which are discussed in Fellenius (2002).

Figure 13 shows distributions presented in a classic paper (Gregersen *et al*., 1973), where the axial force present in a driven pile was measured before the start of the test. The graph shows the total load ("true distribution") as well as just the load imposed by the test ("false distribution"). The similarity to the "false distribution" curve to the load distribution curves of the subject test (Figure 5) is very convincing.

It is of course desirable to measure the distribution of axial force present at the start of a test. Unfortunately, this is rarely possible for many reasons, mainly due to lack of un-

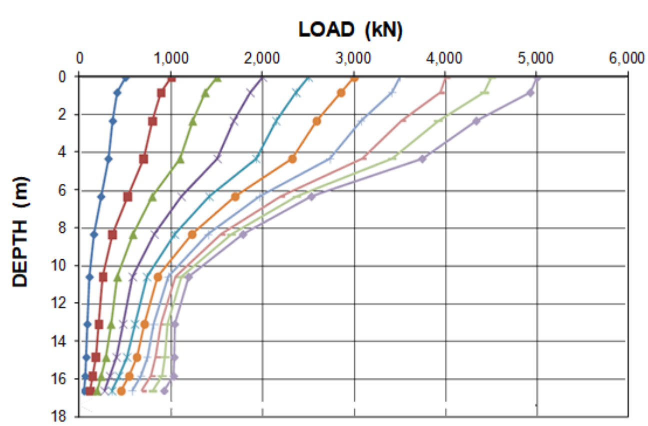

Figure 11. Load distribution for a 620 mm diameter, 16.6 m long screw pile (Burlon, 2016)

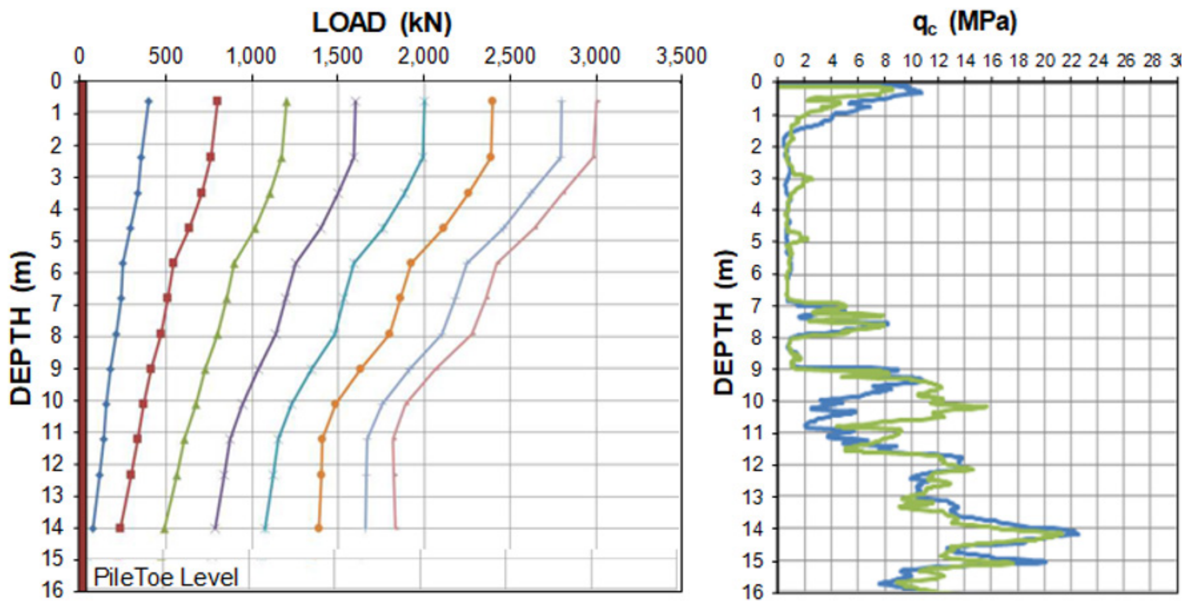

Figure 12. Load distribution for a 510 mm diameter, 16 m driven cast-in-place pile with toe enlarged to 600 mm width (Verstraelen *et al.*, 2016)

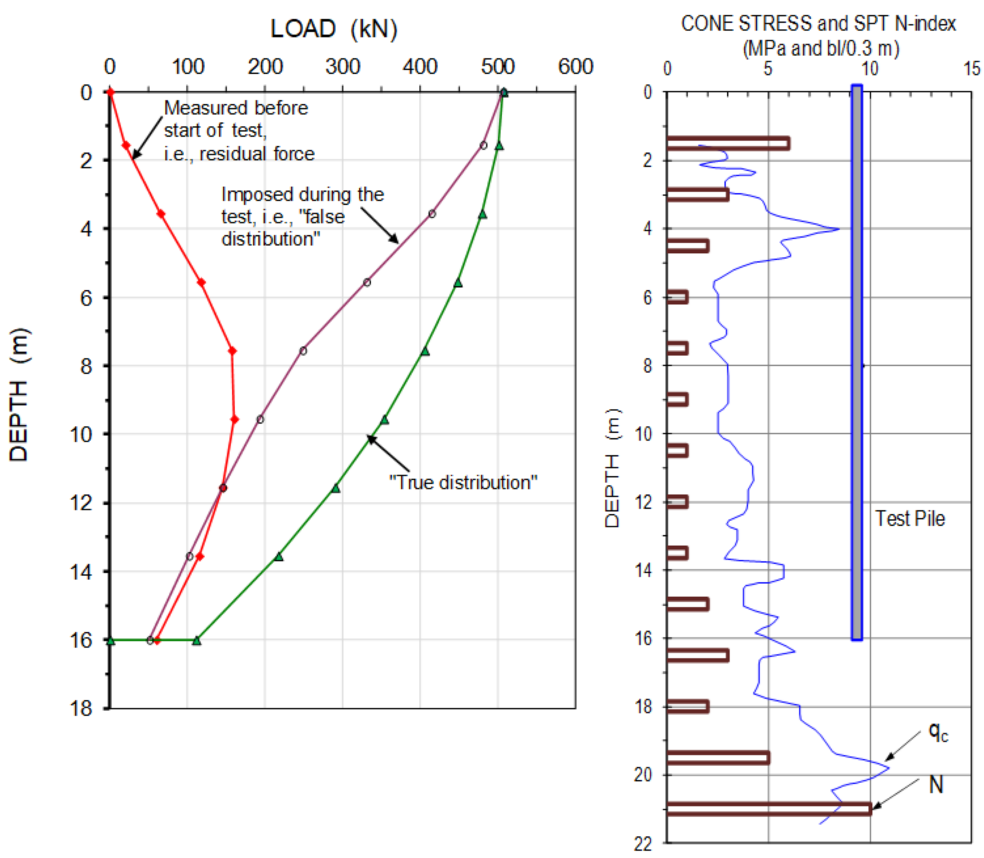

Figure 13. Load distribution for a precast concrete pile driven into sand (Gregersen *et al.*, 1973)

derstanding of the necessity. However, “the true distribution” can be estimated from the measured “false distribution”. The next section will present the principles of development and consequence of residual force along with comments on how to go from the “false” to the “true”.

Residual Force

Principles of Residual Force

Residual force develops in principle by the same mechanism as that for a drag force. The term “residual force” is used when the force has developed before the static loading test. The term "drag force" is used when the force develops after construction of the structure supported by piled foundation.

Figure 14 shows the development of residual force in a test pile due to subsiding soil above the N.P., determined in the analysis of hypothetical test records. The dashed curve shows a true virgin force movement condition for a pile element in the form of a t-z function starting at Point *O* going to Point *A*. In a following static loading test, when residual force has developed along Path *OB*, the shear along a pile element will follow Path *BB'A*. An interpreter of the test records, not realizing the presence of the residual force, will consider the path to be Path *OA'* and arrive at values of shaft resistance much larger than the true value, possibly even twice as large.

Below the N.P., as shown in Figure 15, the residual force has developed along Path *OB*. The loading test introduces a continued loading of the pile element along Path *BA* and the strain-gage record will indicate increasing strain—and load—in the pile However, an innocent interpreter would believe the path is along *OA'*, greatly underestimating the resistance—along the shaft and at the toe.

Figure 16 shows the principles for residual force remaining in a pile upon unloading of a driving force that mobilized a toe resistance. The impact force causes the pile to move via Path *OC* and, as the pile toe springs back along Path *CB*, it leaves a residual force at Point B. During the subsequent static loading test, the pile toe is engaged along Path *BC*. The interpreter will take the response along Path *BCA* and believe that the measured toe response is according to Path *OA'* and, maybe, remark that the toe force is rather small. The residual toe force is countered by a negative direction shear force along the pile elements above the pile toe and the effect on the static loading test records is then similar to that shown in Figure 14.

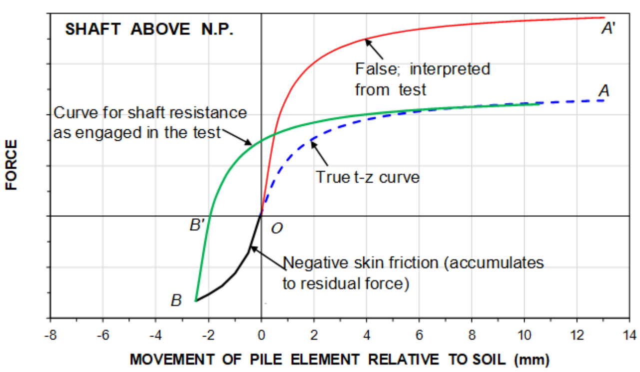

Figure 14. The development of "false" resistance from downdrag (negative movement is upward)

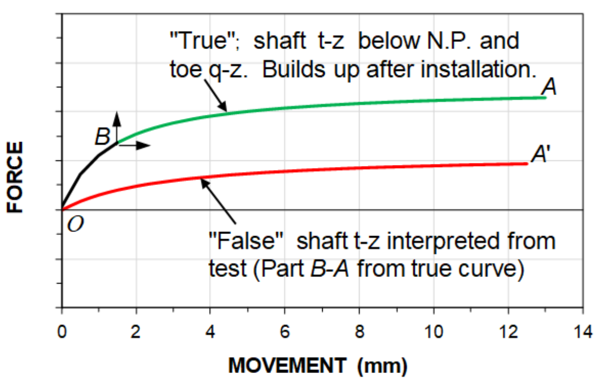

Figure 15. Pre-loading by residual force along the lower length of the pile and pre-loading of the pile toe

"False" and "True" Load Distributions and Determining the Distribution of Residual Force

The "true" distribution is difficult to find and it cannot be determined exactly from the test records, but if the "false" distribution is well ascertained, the back-and-forth trial-and error approach illustrated in Figure 17 will produce a fair

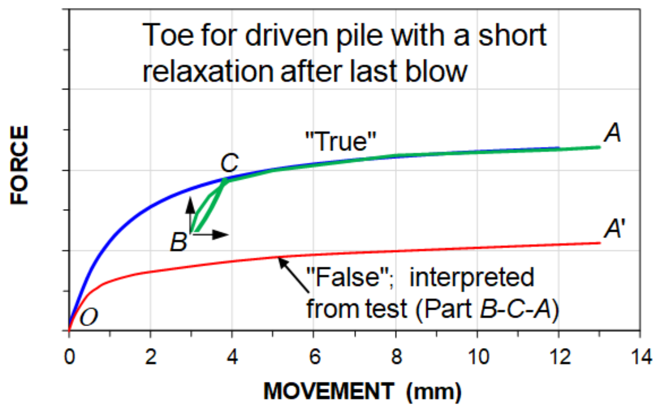

Figure 16. Residual force at the pile toe in case of a driven pile

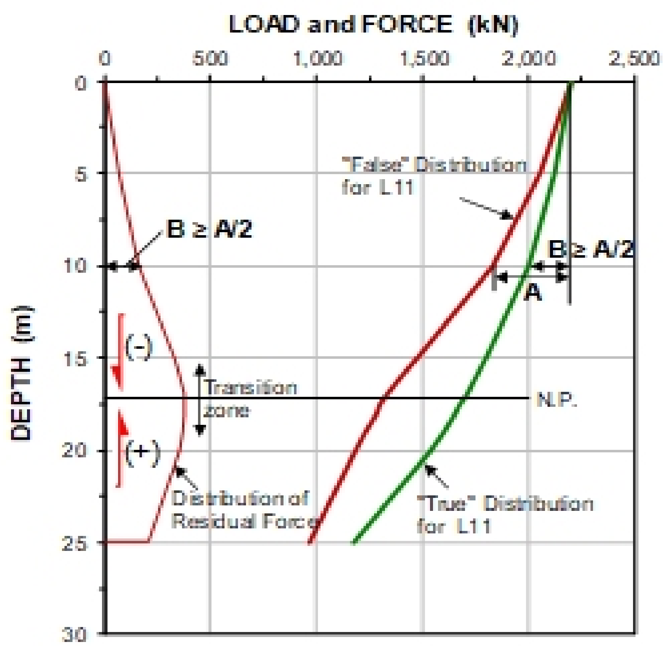

Figure 17. Procedure for estimating distribution of residual force and "true" resistance. For fully mobilized Residual Force B = A/2.

representation of the true distribution. For starts, the assumption is made that the shaft resistance along the upper length of the pile is "false" shaft resistance (A) and that the "true" resistance (B) along this length is equal to the negative skin friction accumulating to residual force. That is, the "false" distribution incorporates a negative direction shaft shear that is exactly twice that of the positive "true" shaft resistance. For what length of pile along this might be correct, if at all, is not known, of course. However if the so-determined "true" distribution is extended down the soil, it will be soon be obvious that the negative skin friction that accumulates to the residual force must start to diminish and, at some depth, change to positive shaft resistance. This requires the "true" distribution curve to be adjusted accordingly, while recognizing that the interacting curves must not show kinks or sudden or reversed changes. Near the pile toe, the slope of the residual force distribution cannot be less steep (be flatter) than the "true" distribution. The two slopes are equal if the residual force in this zone is due to fully mobilized shaft resistance. In the latter case, the "false" distribution curve becomes vertical (c.f., Figures 11 and 12). For more details on the procedure see Fellenius (2021).

Figure 18 is Figure 5 with the distribution added for the 1,400-kN Target Load corrected for residual force according to the mentioned procedure and the fully compensated residual force. The array of curves to the left of the residual force curve indicates the amount of residual force that is compensated for Load Increments L1 - L6. The load-strain curves in the right graph are interactive with the left graph. I assumed that the true beta-coefficient in the upper 5 m was half that of the back-calculated test, i.e., 0.40 as opposed to 0.80. In the clay below, from 5 m through 25 m, I applied a constant value of ß = 0.30, which determined the toe resistance for the Target Load to 450 kN. I could have continued by fine-tuning the beta-coefficients, setting ß at 5 m depth to 0.28 and letting it increase to about 0.32 at 25 m depth. This would have given a 480-kN toe resistance; much the same result. It is not likely that the increase of the beta-coefficient with depth could have been larger than about 0.04 because this would have made the lower portion of the "true" resistance less steep and, also, unrealistically reduced the calculated toe resistance.

The gradual compensation of the residual force during the first load increments, was estimated by trial-and-error calculations aiming to both obtain smooth load distribution curves and consistent shape of the load-strain curves.

Figures 19A and 19B compare the load distributions before and after adjustment for residual force. Although the t-z functions for the shaft resistance pile element and the q-z function for the pile-toe element do not deviate much from those shown in Figure 8, overall, the response of a pile subjected to residual force will appear stiffer than a pile without presence of residual force.

To simulate a load-movement response that includes effect of the presence of residual force and the adjusted *ß*-coefficients, new t-z and q-z functions are now input. The response is plotted in Figure 20 showing the simulated curves ("Head, Adj.") of a static loading test unaffected by residual force as well as the load-movement curve of the actual test

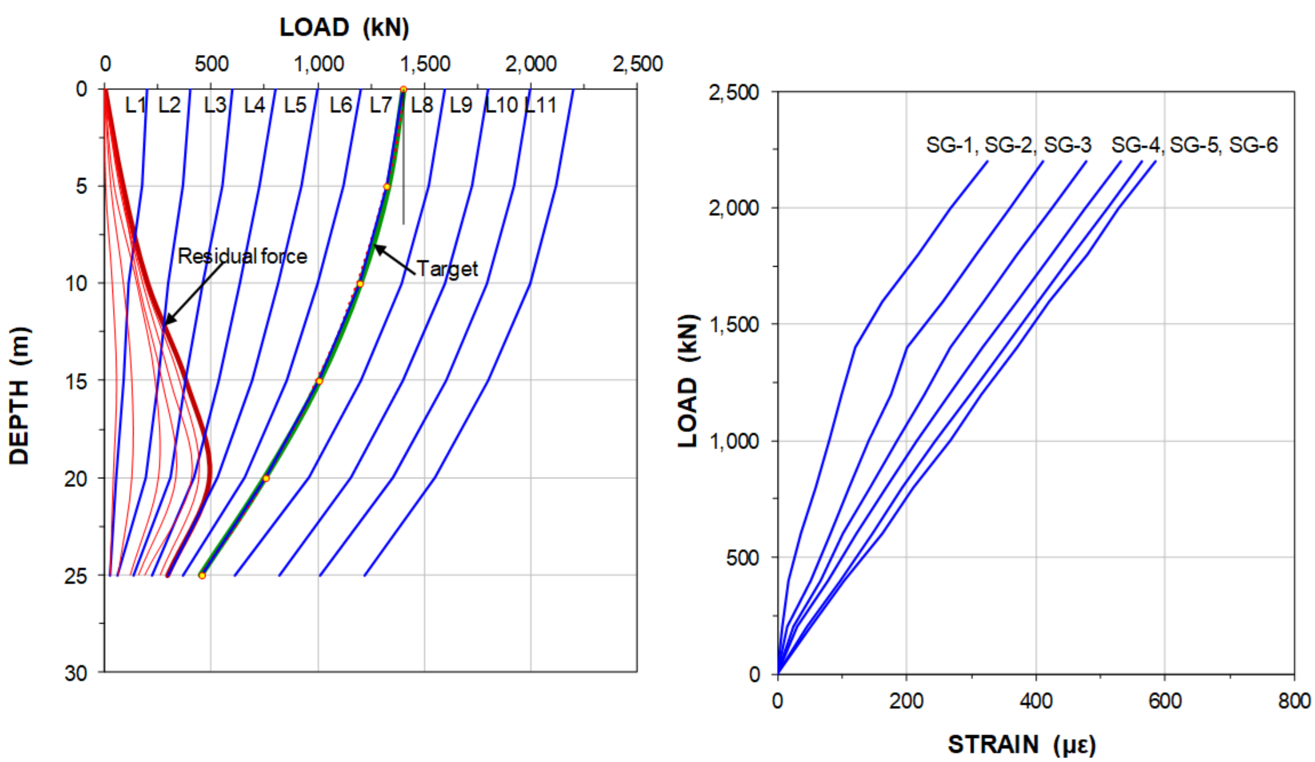

Figure 18. Result of applying the procedure for eliminating the residual force from the test pile records

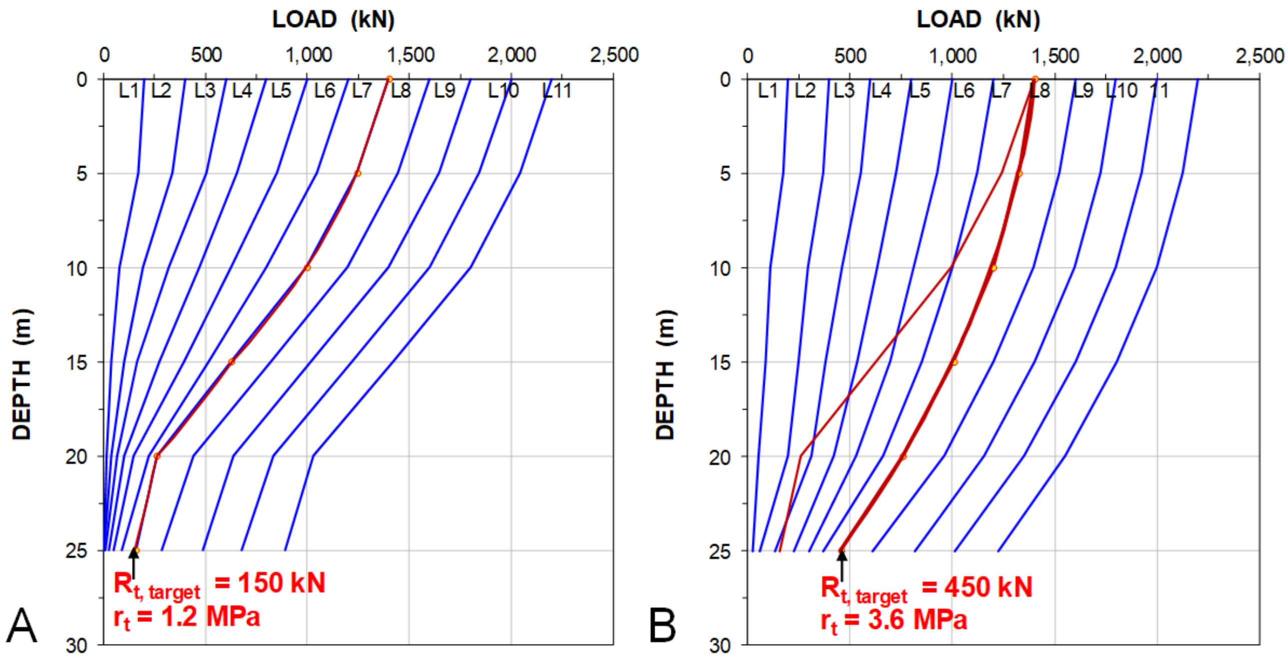

Figure 19. The load distributions with the residual force removed

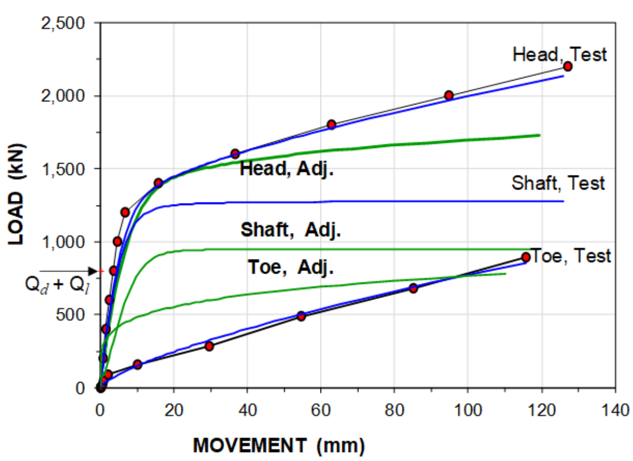

Figure 20. Load-movement curves: in test and after adjustment for residual force

(test data and simulation, c.f., Figure 9). The adjusted pile response is less stiff, the shaft resistance is smaller, and, while the toe resistance is stiffer at small movements, the resistance tends to be smaller at large movements. It is conceivable that an ultimate resistance interpreted from a test on an unaffected pile would be smaller than that interpreted from the test on the actual pile. Therefore, a design based on the shape of the pile-head load-movement curve needs to consider whether or not the pile is affected by presence of residual force and, if so, whether or not the residual force is due to general subsidence and, therefore, reliable, or due to reloading and unloading of the test pile before or during the actual test, in which case the interpretation of the curve shape may be unreliable.

Bidirectional Test and Residual Force

Determining the true force distribution by analysis is not as reliable as measuring the distribution of residual force in the pile before the start of a static loading test. However, such measurements come with many questions about changes of measured strain due to effects other than buildup of residual force, which influence the measured strain without resulting in any shear forces along the pile. Such factors are the temperature change in the pile from that above grade to that in the ground, the temperature changes during hydration of concrete in a bored pile, and the effect of swelling of a precast pile when the concrete absorbs water from the soil. Strain-hardening and strain-softening are additional factors of uncertainty affecting interpretation of strain measurements.

A bidirectional test (BD) supplies measurement of axial force in the pile that is unaffected by residual force. In the bidirectional test, the load at the cell (BD load) is the true load at that location. This coupled with the fact that the load at the pile head is known—note, a load of 0 kN is still a known value—enables the interpreter to determine the distribution of the true axial force between the pile head and the cell quite accurately by analytical methods even without records of strain. Similarly, the distribution between the bidirectional cell level and the pile toe can be determined by analytical methods at a far greater reliability than working from knowing only the pile head load.

Figure 21A shows the results of a hypothetical bidirectional test on the test pile applying the same soil parameters as used for the adjusted head-down test. Figure 21B shows the distribution for the Target BD load applying the same 10-mm Target shaft movement and the same q-z response as used for the simulation of the head-down test.

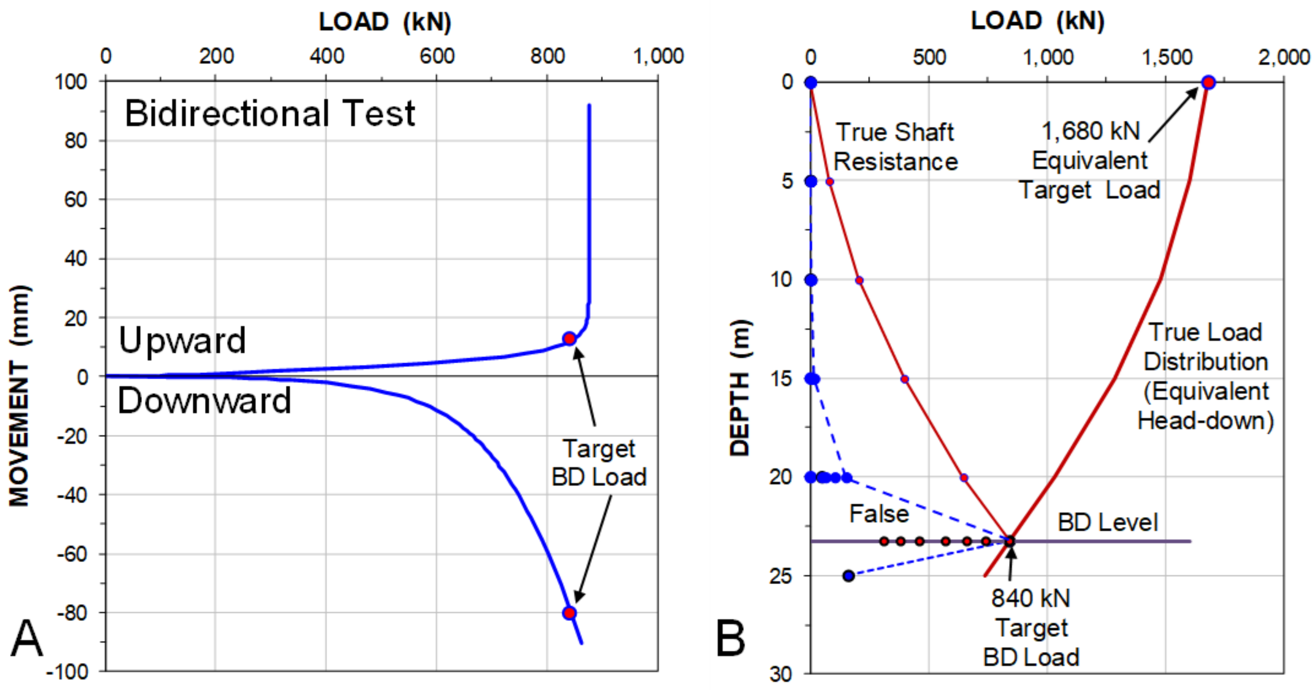

Figure 21. (A) Results of bidirectional test, (B) Equivalent load distribution and True and False shaft resistance

Piled Foundation Settlement for the Test Results Adjusted for Residual Force

The important, perhaps the decisive, difference between analysis disregarding or including presence of residual force is in regard to the long-term settlement. Figure 22 is similar to Figure 10 but incorporates adjustment for residual force. The adjustment has removed the overly large shaft resistance along the upper length of the pile and restored the hidden resistance along the lower length plus returned a stiffer long-term pile-toe response. The adjustment shows a calculated long-term settlement of the single-pile foundation will be 25 mm, much smaller than resulting from the first calculation that disregarded the presence of residual force.

Again, the settlement calculated for the 4- and 6-pile groups will have to consider the transfer of load below the N.P. Because the length of pile below the N.P. is now smaller, the areas of the equivalent rafts will be 5.2 × 5.2 m and 5.2 × 6.4 m and stressed to about 100 and 120 kPa, respectively. The calculated settlement for the 4- and 6-pile foundations is now about 15 mm larger than for the single-pile foundation.

Compilation of Survey Results

The main objective of the survey was, first, to see if the participants would recognize that the test pile was affected by the presence of residual force and, second, how they would approach determining the settlement of the single pile at a subsiding site.

The survey received a total of 52 submissions from a total of 66 participants (a few submitted joint replies) from 20 different countries, as listed in Appendix A. All 52 submissions provided the requested load distribution graph. Two remarked that precise load distributions could not be determined because the information did not include the pile modulus and proceeded by applying a presumed modulus. A few others had difficulty in back figuring the pile stiffness, EA, from the strain records. However, most applied the tangent stiffness method and a few also the secant method as applied to the uppermost gage level, SG-6, and had no difficulty in determining EA = 3.8 GN.

Several seemed to be unsure on what was meant by "load distribution" and instead provided the distribution of shaft resistance. A couple of the latter calculated the shaft resistance as the difference of force between two gage levels. This is theoretically correct, of course, but it is a differentiation method that very much magnifies the imprecision in the measurements.

The majority of the submissions reported a "capacity", determined by a variety of methods, referencing this "capacity' in considering whether or not the intended working load would be acceptable. One submission stated that the drag force was too large for the pile to be used in the foundations. One other submission also expressed concern over the presence of drag force and suggested that in order to reduce the drag force, the piles should be bitumen-coated in the clay (after increasing the penetration into the sand) without realizing that this would reduce the shaft resistance in the clay in equal measure. Surprisingly, two indicated that because the pile length to diameter ratio was too large (25/0.45 = 55), the pile could not be used due to risk for buckling.

Residual Force

Although I expected that the participants would not have any problem in converting the measured strains to axial force (that is, determining the pile axial stiffness, EA, and applying this to the records), that effort was a part of the survey.

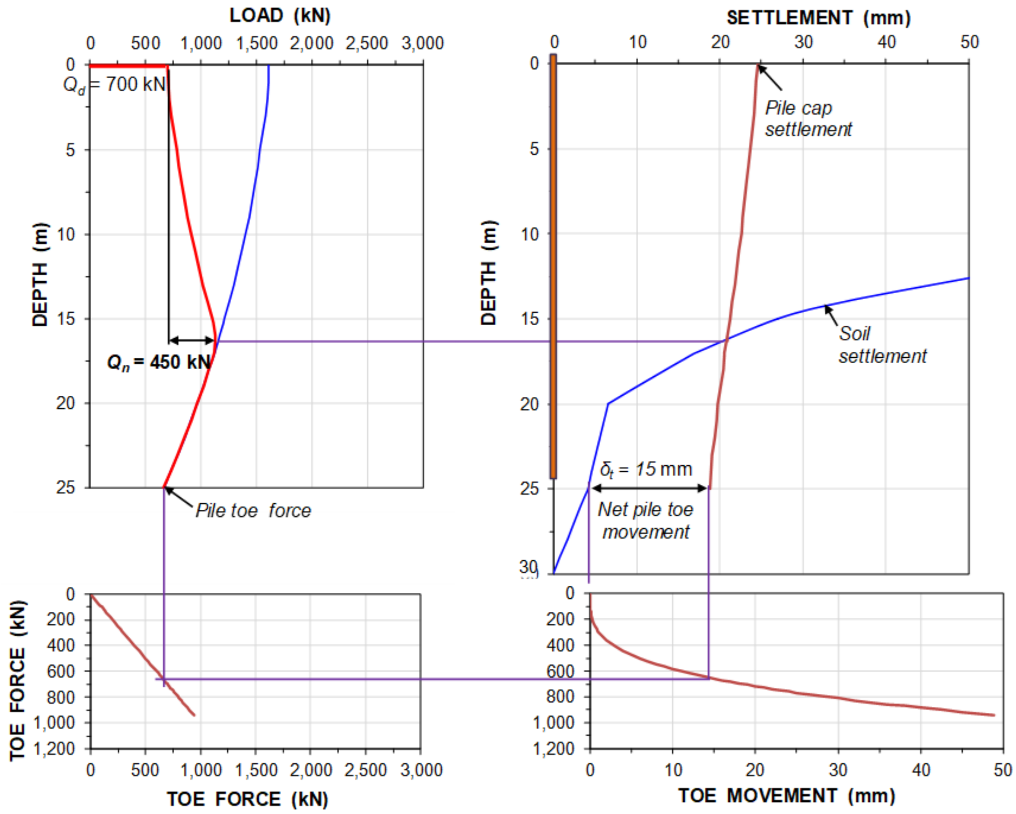

Figure 22. The unified method for determining the pile settlement after adjustment for residual force

Psychologically, while we might question supplied data, we tend to accept data from our own calculation. Had a table of axial force been provided instead of strain, a few more submis-sions would probably have questioned the records and then, recognized the presence of residual force.

Indeed, of the 52 total submissions, only 12 recognized that the test pile was affected by residual force and calculated the distribution. Two other remarked that the records indicated presence of residual force but did not estimate its magnitude and distribution. Of the 12, one stated that "there is no way to assess and incorporate residual forces to determine a true internal force profile" but submitted an 'estimated' curve.

The 12 submitted distributions are compiled in Figure 23. The red curve, the 13th, shows the distribution presented above, c.f., Figure 18. The submission separated out as a dashed curve is incorrect in the 20 to 25 m layer. It would suggest a minimal "true" shaft resistance in this layer. Apart from this comment, no one distribution curve is less believable than the other. This said, a 500-kN residual toe-force would seem a bit unrealistic, when added to the strain-record determined ("false") 150-kN toe resistance for the 10-mm toe movement developed at the 1,400 Target Load.

Settlement of Ground and Piled Foundation

A total of 45 submissions included values of long-term settlement of ground surface and of a foundation on a sin-

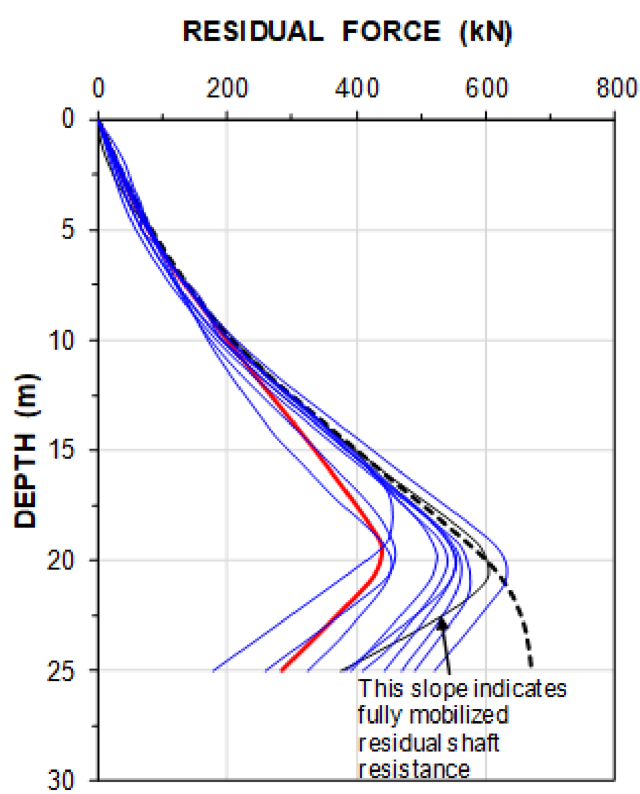

Figure 23. Distributions of residual force

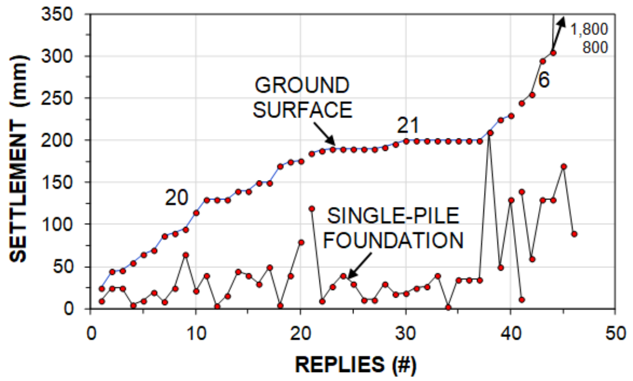

Figure 24. Distributions of calculated settlement of ground surface and foundation on single piles

gle pile, as compiled in Figure 24. Determining the settlement distribution for three soil layers with thickness and compressibility defined and loaded by uniform stress and uniform change of pore pressure is a very routine task of calculation. Even when accepting that a "back-of-the-envelope" calculation could show an about ±10 % deviation from the calculated 200-mm value, only a "middle crowd" of 21 submitted a correct value. One of the other 25 submissions assumed that the lowering of the groundwater table due to water mining only affected the upper silt/sand layer. Another did not understand, it seems, what compressibility is when expressed in terms of compression ratio, CR, or modulus number, m, and used the average densities to determine void ratio, e_0, and applied this together with estimated values of compression index, C_C. The 800- and 1,800-mm settlement values are obvious miscalculations.

Regarding determining the settlement of the piled foundation, a few submissions confused "pile settlement" and "pile movement" and mistakenly reported the pile load-transfer movement instead of the pile long-term settlement; a static loading test does not measure "settlement", but "movement" or "displacement".

Of the 21 "middle crowd" submissions, 15 submitted values of piled foundation settlement ranging from 10 through 50 mm. Most of these applied the unified method and the deviations from the about 25 mm value shown in Figure 22 are due to not including or not applying a fully realistic assumption about the long-term pile toe response. Most of the 12 who recognized the presence of residual force are included in "middle crowd". Two of the 26 submissions from outside the "middle crowd" indicated a settlement for the single pile that was calculated by the Terzaghi-Peck method (an equivalent raft at the lower third depth) but did not include the size of the assumed equivalent raft. Only 5 submissions included settlement values for the narrow pile groups and the values ranged widely: from 10 through 400 mm.

Most of those submitting ground settlement values below and above those of the "middle crowd" also applied the unified method; however, frequently with unrealistic assumptions in regard to the pile-toe long-term load-movement.

True Response to the Applied Loads in the Static Loading Test

My purpose of the fourth point in the instructions was to give an opinion on whether or not the static loading test records of strain and movement represent the true response to the ap-plied loads, indeed, to entice the participants to take a second look at the test records and, perhaps, then realize the presence of residual force. Understandably, many found the point a bit wooly. One replied that because the test records did not in-clude residual force, the records are not true and one that the measured strains were "too exact"—yes, as mentioned in the survey invitation, a couple of numbers had been "polished". One submission criticized the test because the 15-minute load increment duration was too short, as it would result in significant loss of data on 'creep', stating that incorporating unloading-reloading cycles with each peak load held for six hours would have provided a better base for estimating long-term performance (respectfully, I disagree, here). It was also claimed that it is not possible to back-analyze loading test records unless more soil exploration data would be available, e.g., laboratory undrained shear strength values, CPT pro-files, and SPT N-indices (there is likely a confusion between back-analysis and prediction here).

"Would a Foundation Pile Equal to the Test Pile be Accepted as a Foundation Pile"

The fifth point was whether a pile foundation supported on a single pile would be acceptable if the conditions would be identical to those for the test pile. The subject test records were intentionally picked because the analysis results do not equivocally make clear that the desired load can be accepted from either "capacity" or settlement aspects. Of the 52 submissions, 23 answered "yes" to the 5th point, 20 answered "no", and 9 gave no reply. Of the "no" answers, 11 stated that it was due to settlement being larger than the permissible 30-mm value. One "no-answer" questioned the 30-mm settlement limit and indicated that accepting or not accepting settlement must be based on a criterion for differential settlement; indeed, the only of the submissions who reacted to this incomplete information.

The second part of the fifth question was on the use of the pile for the intended foundations. Of the 52 submissions, 29 submitted comments, while 23 did not. Almost everyone of the 29 suggested that the piles should be installed into the lower sand. The recommended increase of the pile penetration ranged from "a few feet" through 10 m. A few suggested that the load per pile should be reduced and the number of piles be increased; not a realistic suggestion for single-pile foundation.

A couple of the submission suggested changing to larger diameter piles and or bored piles. One suggested to add means to expand the pile toe so as to achieve a larger toe resistance (by means of the "simple and economical Expander-Base solutions now available"). One suggested to change to a spun pile, as such pile are "stiffer" and would, therefore, show smaller pile compression. (Yes, because they are often installed by jacking which builds in significant axial residual force in the pile—preloading it, in effect, which results in a 'stiffer' response).

Discussion and Conclusions

Whether or not the desired unfactored 800-kN working load is acceptable—as based on the evaluated pile-head load-movement curves—depends on the interpreter's preferred definition of "capacity" and applicable codes or standards. However, basing a design on some prescribed resistance factor applied to a somehow perceived "capacity" is, in my opinion, not a satisfactory approach to foundation design. As to the measured pile-head movement, unfortunately, the test did not include monitoring of the reference beam. Even if the supports of the reference beam were placed at the usually prescribed 2-m distance from the test pile, when the kentledge load is transferred to the pile, the supports can heave considerably. The heave is then interpreted as downward movement of the pile head unless the beam movement is monitored, and the values used to correct the records.

The survey invitation quoted the original project statement that permissible long-term settlement of the piled foundation is 30 mm. The value probably originated in the one-inch value that is frequently included in geotechnical reports without much thought of the response of the structure to settlement, notably, not indicating whether it refers to total or to differential settlement, which omission I kept with the survey questionnaire. The 30-mm "permissible settlement" is an unintended marker indicating a lax attitude toward the design. It is also overly strict for most structures. With respect to differential settlement, a value is meaningless unless coupled with the distance over which the settlement will occur and coupled with information in regard to whether the differential settlement is by hogging or sagging. In most cases, the boiler-plate "one-inch" value is on the 'safe' side, which is probably why the reference still prevails.

The geotechnical and structural engineer need to compare notes. The structural engineer must establish the extent of permissible total and differential settlement of the structure and the geotechnical engineer must see if these limits can be accommodated by the proposed foundation and, if not, discuss what changes to the foundation and/or the structure could be implemented to ensure the proper long-term response and best possible economy of the finished project. For the subject case, complete foundation recommendations cannot be provided. For example, it is not clear to what extent, if any, the proximity of the pile to the lower sand layer has affected the toe response. The toe resistance of the test pile is small. Whether or not a 25 m long pile placed where the lower sand is well below the pile toe level would have shown a similar response to that of the test pile, is not known. Therefore, it was a mistake to place the test pile with the pile toe at the layer boundary. Because a design based on a pile entirely within the clay layer, and no longer than about 25 m, would have had significant effect on the project costs, performing two tests, one with the pile toe terminating in the clay just above the sand and one with the pile toe at least 2 m into the sand, would certainly have been worthwhile. However, if the presence of residual force would not be recognized, the shaft and toe resistances mobilized in the two tests would be different. This would certainly have caused some consternation for the test interpreter.

Depending on details regarding acceptable differential settlement, it is likely that the 25 m pile would be acceptable for the design of the project piled foundations, if based on an analysis recognizing the presence of the residual force. In contrast, , is quite possible that a final recommendation not based on assessing the presence of residual force might have been to require the piles to be longer. Moreover, because pile groups would experience settlement in the clay below the pile toe level, it is probable that piled foundations supported on pile groups would have to be on piles driven into the sand layer. (The actual depths to the sand layer across the site would need to be established and the pile lengths would have to be adjusted to the actual soil profile geometry).

Can the piles, if needed, be extended beyond 25 m? What is the cost effect for extending the piles a metre or two? Due to restrictions at the concrete plant and regarding transportation to the site, 26 m length is the limit for the particular precast pile option. However, prestressed precast piles can be equipped with mechanical splices that make it possible to drive the pile deeper. Indeed, using two 15 m long segments spliced in the field and driving to almost 30-m pile embedment would also probably have saved costs and time. But the engineers for this project were reluctant to the idea as they did not have experience with spliced precast piles.

Moreover, it would have been interesting and enlightening to see results of dynamic tests carried out at end of driving and at restrike after full set-up. A few of the suggested solutions to the marginally acceptable piled foundation may appear less thoughtful and realistic. However, it is necessary to understand that the submissions cannot be considered fully representative for what the participants would suggest in a real case with ample time for review of all conditions for a project. For any test, the details of the analysis of test results is a matter of personal experience, preference, and practice. Indeed, it is very sobering that the survey results show that two engineers, working from the same test data, will not necessarily come up with the same conclusions for the design.

References

Burlon, S. (2016). *Design methods based on static pile load tests*. ISSMGE - ETC 3 International Symposium on Design of Piles in Europe, General Report, Vol. 1, Brussels, April 28-29, pp. 87-96.

Fellenius, B.H., 1988. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75-82.

Fellenius, B.H., 1984. Negative skin friction and settlement of piles. Proceedings of the Second International Seminar on Piled Foundations, Nanyang Technological Institute, Singapore, November 28 - 30, 12 p.

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. (2002). Determining the true distribution of load in piles. International Deep Foundation Congress, An International Perspective on Theory, Design, Construction, and Performance, ASCE, GSP116, Edited by M.W. O'Neill, and F.C. Townsend, Orlando, Florida, February 14-16, 2002, Vol. 2, pp. 1455 – 1470.

Fellenius, B.H. (2017). Report on the B.E.S.T. prediction survey of the 3rd CBFP event. *Proceedings of the 3rd Bolivian International Conference on Deep Foundations*, Santa Cruz de la Sierra, Bolivia, April 27-29, Vol. 3, pp. 7-25.

Fellenius, B.H. (2019). Observations and analysis of wide piled foundations. *Canadian Geotechnical Journal*, 56(3), 378-397. doi.org/10.1139/cgj-2018-0031.

Fellenius, B.H. (2021). *Basics of foundation design—a textbook*. Electronic Edition, www.Fellenius.net, 534 p.

Goudreault, P.A. & Fellenius, B.H. (2014). UniPile Version 5, User and Examples Manual. UniSoft Geotechnical Solutions Ltd. [www.UniSoftGS.com], 120 p.

Gregersen, O.S., Aas, G., & DiBiagio, E. (1973). Load tests on friction piles in loose sand. *Proc. of 8th ICSMFE*, Moscow, August 12-19, Vol. 2.1, Paper 3/17, pp. 109–117.

Verstraelen, J., Maekelberg, W., & Medaets, M. (2016). Recent experiences with static pile load testing on real job sites. Symposium on Design of Piles in Europe, Vol. 1, Brussels, April 28- 29, pp. 63-85.

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Shukla, Jaykumar	India
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Sulaiman, Ahmad	Indonesia
Suri Surendra	USA
Tara, David	Canada
Terceros, Mario H.	Bolivia
Terceros, Mario, A.	Bolivia
Togliano, Gianni	Switzerland

CPTU pushed next to Test Pile, (a = 0.80. (copy it and paste it into Excel for numerical access)

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)	Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)	Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
0.050	1.661	9.8	58.4	3.050	6.750	43.5	129.8	6.050	0.618	8.6	240.4
0.100	1.884	16.9	88.4	3.100	6.013	44.2	128.1	6.100	0.632	8.0	234.0
0.150	1.684	31.6	61.8	3.150	5.133	42.5	118.5	6.150	0.587	7.1	249.4
0.200	1.617	18.9	34.7	3.200	4.900	35.2	123.0	6.200	0.591	8.2	247.1
0.250	2.694	20.7	19.2	3.250	5.088	32.2	125.3	6.250	0.627	5.6	234.1
0.300	5.531	70.3	78.1	3.300	5.295	31.6	132.2	6.300	0.588	10.9	242.5
0.350	5.900	68.2	87.1	3.350	5.588	34.4	132.0	6.350	0.580	10.6	260.1
0.400	6.818	61.8	83.9	3.400	6.298	36.9	136.8	6.400	0.614	8.5	223.7
0.450	6.494	54.9	81.8	3.450	7.172	38.3	136.6	6.450	0.595	7.5	223.7
0.500	6.704	53.6	87.8	3.500	7.565	42.5	134.5	6.500	0.577	7.1	240.0
0.550	6.741	46.8	90.3	3.550	7.186	44.7	132.0	6.550	0.623	7.0	227.8
0.600	6.595	50.7	93.3	3.600	6.659	44.7	130.1	6.600	0.591	7.1	234.6
0.650	6.522	48.5	92.0	3.650	5.990	39.4	124.5	6.650	0.615	7.1	244.0
0.700	6.848	47.5	95.7	3.700	5.202	35.5	123.6	6.700	0.622	8.9	256.6
0.750	6.871	48.7	98.4	3.750	4.915	33.4	127.6	6.750	0.599	9.2	233.6
0.800	6.411	51.4	86.4	3.800	4.750	27.3	128.5	6.800	0.628	10.6	257.1
0.850	5.945	51.9	88.7	3.850	4.559	25.8	134.9	6.850	0.643	8.5	222.4
0.900	5.437	54.7	93.4	3.900	4.696	26.4	138.9	6.900	0.584	8.0	246.7
0.950	5.066	53.8	95.2	3.950	4.916	27.5	139.2	6.950	0.588	6.1	257.9
1.000	4.618	54.6	95.6	4.000	5.177	30.1	140.8	7.000	0.586	5.8	246.7
1.050	4.398	54.6	95.6	4.050	5.483	33.0	141.2	7.050	0.589	6.5	251.6
1.100	4.202	54.0	95.6	4.100	5.973	35.2	141.4	7.100	0.570	6.6	260.7
1.150	3.904	54.2	93.6	4.150	6.981	40.3	143.9	7.150	0.627	7.4	268.4
1.200	3.670	55.1	89.1	4.200	8.062	47.7	147.9	7.200	0.632	8.0	230.9
1.250	4.343	30.5	69.8	4.250	5.088	32.2	125.3	7.250	0.604	9.1	262.6
1.300	4.429	40.5	61.4	4.300	5.295	31.6	132.2	7.300	0.592	12.0	261.1
1.350	5.787	45.0	90.4	4.350	5.588	34.4	132.0	7.350	0.986	14.1	237.7
1.400	6.980	29.4	36.8	4.400	6.298	36.9	136.8	7.400	0.703	18.3	218.7
1.450	7.352	35.2	5.6	4.450	7.172	38.3	136.6	7.450	0.707	16.6	225.3
1.500	6.578	41.8	3.4	4.500	7.565	42.5	134.5	7.500	0.808	18.2	234.5
1.550	6.328	41.6	14.9	4.550	7.186	44.7	132.0	7.550	0.828	16.0	202.2
1.600	6.202	39.7	35.5	4.600	6.659	44.7	130.1	7.600	0.684	14.4	211.2
1.650	5.655	45.5	56.4	4.650	5.990	39.4	124.5	7.650	0.658	11.7	244.0
1.700	5.423	42.2	74.5	4.700	5.202	35.5	123.6	7.700	0.650	10.0	257.3
1.750	5.495	41.8	92.7	4.750	4.915	33.4	127.6	7.750	0.620	11.4	252.9
1.800	5.638	42.9	90.3	4.800	4.750	27.3	128.5	7.800	0.871	16.6	253.5
1.850	5.511	44.7	80.2	4.850	4.559	25.8	134.9	7.850	0.720	19.1	176.8
1.900	5.576	41.0	87.9	4.900	4.696	26.4	138.9	7.900	0.701	20.0	226.5
1.950	5.631	39.9	89.3	4.950	4.674	32.9	228.8	7.950	0.631	11.5	245.6
2.000	5.746	36.1	90.0	5.000	1.222	22.6	215.1	8.000	0.705	14.0	262.0
2.050	5.572	40.8	88.0	5.050	0.641	12.8	195.6	8.050	0.913	13.8	206.6
2.100	5.059	34.2	86.9	5.100	0.579	8.4	236.8	8.100	1.036	18.1	189.5
2.150	3.589	43.0	87.0	5.150	0.584	6.7	244.4	8.150	0.844	22.2	171.9
2.200	3.464	31.3	73.5	5.200	0.597	7.2	242.7	8.200	0.755	24.2	192.6
2.250	5.422	30.2	82.0	5.250	0.612	8.2	225.5	8.250	0.643	20.5	228.7
2.300	6.304	35.8	65.2	5.300	0.684	10.2	230.4	8.300	0.606	10.2	264.6
2.350	6.190	42.2	70.4	5.350	0.831	15.5	200.3	8.350	0.642	8.9	250.3
2.400	5.546	43.4	86.4	5.400	0.594	16.4	229.5	8.400	0.828	15.0	261.2
2.450	5.571	39.3	102.4	5.450	0.566	7.9	234.5	8.450	1.036	23.9	155.7
2.500	5.930	39.6	115.7	5.500	0.534	6.1	245.7	8.500	0.939	24.8	152.8
2.550	6.233	40.7	122.5	5.550	0.560	5.8	249.0	8.550	0.916	21.9	214.0
2.600	5.944	42.2	123.0	5.600	0.612	8.6	229.1	8.600	1.039	23.5	173.1
2.650	5.426	41.4	117.7	5.650	0.632	6.7	215.1	8.650	0.794	19.9	197.1
2.700	5.051	37.1	118.8	5.700	0.586	6.1	229.8	8.700	0.657	14.4	233.5
2.750	5.083	38.0	121.0	5.750	0.562	6.3	251.2	8.750	0.634	9.4	263.3
2.800	5.349	27.7	125.3	5.800	0.568	5.7	249.4	8.800	0.759	9.4	272.8
2.850	5.693	30.7	125.5	5.850	0.563	5.5	246.6	8.850	1.100	10.6	189.9
2.900	5.720	35.7	131.2	5.900	0.558	6.4	249.4	8.900	0.976	18.2	169.7
2.950	5.894	37.6	127.6	5.950	0.586	7.2	244.1	8.950	0.763	16.0	199.6
3.000	6.444	39.5	131.5	6.000	0.646	6.8	244.9	9.000	0.647	10.0	237.6

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
9.050	0.623	6.8	259.1
9.100	0.625	7.1	269.3
9.150	0.639	9.2	272.0
9.200	0.715	8.4	261.9
9.250	0.741	18.4	238.6
9.300	0.640	21.6	217.9
9.350	0.758	18.1	247.3
9.400	0.648	13.7	211.9
9.450	0.642	12.3	232.1
9.500	0.623	11.1	254.3
9.550	0.680	9.2	246.2
9.600	0.685	9.8	248.5
9.650	0.764	9.7	230.5
9.700	0.659	8.3	232.4
9.750	0.621	7.7	261.5
9.800	0.621	6.0	267.9
9.850	0.612	6.1	279.3
9.900	0.607	6.2	285.7
9.950	0.611	6.5	283.2
10.000	0.687	11.6	281.4
10.050	0.719	10.5	244.4
10.100	0.704	11.5	273.2
10.150	0.686	8.3	253.5
10.200	0.651	8.3	279.6
10.250	0.658	12.0	292.7
10.300	0.787	16.0	265.5
10.350	0.813	18.4	271.0
10.400	0.701	15.7	226.4
10.450	0.682	12.4	256.1
10.500	0.675	11.9	280.5
10.550	0.659	10.5	285.8
10.600	0.691	8.3	282.1
10.650	0.708	8.1	273.9
10.700	0.657	8.8	279.6
10.750	0.659	9.8	289.6
10.800	0.746	8.9	269.2
10.850	0.718	8.1	256.9
10.900	0.661	8.8	272.4
10.950	0.655	7.1	289.3
11.000	0.660	8.3	293.6
11.050	0.744	9.4	286.0
11.100	0.723	10.6	276.4
11.150	0.697	10.4	284.2
11.200	0.687	9.3	291.5
11.250	0.689	14.9	291.0
11.300	0.746	15.6	260.6
11.350	0.798	14.3	276.8
11.400	0.686	10.6	231.6
11.450	0.684	7.8	271.4
11.500	0.663	7.6	288.5
11.550	0.667	7.7	297.2
11.600	0.669	7.4	303.9
11.650	0.668	7.4	310.6
11.700	0.680	7.8	317.9
11.750	0.696	7.7	323.3
11.800	0.788	9.0	337.7
11.850	0.817	10.2	334.6
11.900	0.835	11.2	346.3
11.950	0.847	10.9	345.8
12.000	0.844	10.7	353.0

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
12.050	0.847	11.1	358.0
12.100	0.837	10.9	356.3
12.150	0.835	10.8	354.8
12.200	0.829	10.8	347.2
12.250	0.812	10.7	341.7
12.300	0.797	10.9	335.4
12.350	0.792	10.6	328.3
12.400	0.794	11.2	330.5
12.450	0.801	11.4	329.2
12.500	0.818	13.4	333.2
12.550	0.809	13.7	328.0
12.600	0.825	14.3	345.8
12.650	0.829	14.0	344.0
12.700	0.825	14.1	361.6
12.750	0.851	15.4	350.0
12.800	0.840	14.4	341.5
12.850	0.863	15.6	357.2
12.900	0.889	16.1	343.1
12.950	0.883	16.1	335.0
13.000	0.865	16.4	343.2
13.050	0.893	17.4	344.6
13.100	0.884	15.6	350.3
13.150	0.877	16.2	345.5
13.200	0.890	14.1	341.7
13.250	0.880	16.7	354.2
13.300	0.858	17.0	345.4
13.350	0.894	16.9	340.0
13.400	0.872	16.8	346.3
13.450	0.834	15.3	349.4
13.500	0.831	14.0	364.3
13.550	0.836	13.1	370.1
13.600	0.853	12.6	372.9
13.650	0.873	12.9	374.9
13.700	0.888	13.4	366.6
13.750	0.881	13.6	360.9
13.800	0.880	14.0	362.9
13.850	0.890	14.1	348.8
13.900	0.894	15.0	357.7
13.950	0.884	15.0	356.9
14.000	0.886	15.3	356.3
14.050	0.873	14.1	358.5
14.100	0.819	13.8	352.9
14.150	0.836	15.4	349.4
14.200	0.852	13.0	350.5
14.250	0.845	15.6	358.4
14.300	0.810	15.1	329.6
14.350	0.857	15.5	349.9
14.400	0.864	14.7	353.7
14.450	0.872	14.7	351.6
14.500	0.862	14.9	353.5
14.550	0.852	14.7	354.7
14.600	0.854	14.1	351.8
14.650	0.875	14.5	350.3
14.700	0.885	15.7	351.2
14.750	0.893	16.1	355.5
14.800	0.887	15.3	343.8
14.850	0.909	14.5	350.8
14.900	0.867	14.2	349.9
14.950	0.874	14.9	359.3
15.000	0.869	14.3	357.8

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
15.050	0.894	14.4	355.7
15.100	0.819	15.2	351.5
15.150	0.842	15.1	358.0
15.200	0.891	13.6	367.1
15.250	0.866	14.4	361.6
15.300	0.861	15.3	359.8
15.350	0.893	15.8	358.5
15.400	0.863	15.5	356.4
15.450	0.845	15.2	363.8
15.500	0.844	14.1	368.8
15.550	0.881	14.3	382.3
15.600	0.924	15.2	376.9
15.650	0.925	14.6	374.8
15.700	0.929	15.9	378.2
15.750	0.925	15.8	371.5
15.800	0.900	15.3	379.6
15.850	0.896	14.4	381.4
15.900	0.895	14.7	375.1
15.950	0.879	14.7	367.1
16.000	0.899	15.3	376.5
16.050	0.902	14.7	367.1
16.100	0.915	14.7	380.0
16.150	0.921	16.0	371.8
16.200	0.909	15.1	372.9
16.250	0.879	18.2	366.6
16.300	0.931	17.2	378.6
16.350	0.962	18.0	378.0
16.400	0.929	17.1	370.7
16.450	0.916	16.4	373.8
16.500	0.961	17.2	376.0
16.550	0.961	17.2	366.2
16.600	0.948	17.6	368.4
16.650	0.935	16.9	373.6
16.700	0.926	16.6	376.4
16.750	0.936	16.4	385.5
16.800	0.932	16.4	384.6
16.850	0.934	15.7	384.5
16.900	0.932	16.3	386.4
16.950	0.944	15.9	385.9
17.000	0.934	15.5	391.1
17.050	0.934	15.3	385.3
17.100	0.925	16.6	381.4
17.150	0.928	14.8	376.9
17.200	0.897	14.9	377.8
17.250	0.933	17.4	371.8
17.300	0.955	17.6	373.8
17.350	0.959	18.1	374.7
17.400	0.937	17.1	365.6
17.450	0.933	17.2	374.2
17.500	0.932	16.2	374.7
17.550	0.940	16.6	376.9
17.600	0.924	15.8	368.4
17.650	0.927	16.2	373.3
17.700	0.936	16.3	377.1
17.750	0.922	15.8	374.0
17.800	0.916	15.7	365.2
17.850	0.922	15.8	368.7
17.900	0.922	15.5	373.0
17.950	0.926	16.0	376.5
18.000	0.939	15.8	380.3

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)	Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)	Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
18.050	0.925	16.4	373.1	21.050	0.948	14.8	387.3	24.050	0.945	11.2	428.1
18.100	0.927	16.7	366.1	21.100	0.929	13.1	387.7	24.100	0.932	10.9	427.8
18.150	0.899	15.6	362.2	21.150	0.900	13.6	376.0	24.150	0.951	14.9	423.8
18.200	0.902	13.1	367.1	21.200	0.875	13.3	366.6	24.200	0.966	16.9	412.1
18.250	0.932	16.8	360.8	21.250	0.884	15.1	381.0	24.250	1.106	16.7	329.0
18.300	0.899	17.7	341.7	21.300	0.899	15.0	382.3	24.300	1.021	17.8	364.3
18.350	0.915	18.0	353.0	21.350	0.931	15.8	382.0	24.350	0.986	14.7	388.6
18.400	0.894	17.0	365.7	21.400	0.910	15.6	383.4	24.400	0.980	11.9	398.6
18.450	0.879	14.7	368.1	21.450	0.918	14.8	381.8	24.450	0.965	10.6	399.4
18.500	0.877	14.7	361.0	21.500	0.901	15.0	378.7	24.500	0.940	9.6	409.3
18.550	0.895	15.3	367.4	21.550	0.920	13.3	390.4	24.550	0.976	9.6	422.2
18.600	0.874	14.5	362.9	21.600	0.908	12.6	390.1	24.600	0.954	10.2	434.1
18.650	0.861	13.0	372.8	21.650	0.961	13.7	398.6	24.650	0.943	10.6	432.3
18.700	0.900	13.9	382.8	21.700	0.959	13.3	398.0	24.700	0.949	10.2	431.0
18.750	0.924	14.0	365.2	21.750	0.983	13.0	401.8	24.750	0.970	9.9	431.4
18.800	0.900	16.1	375.6	21.800	0.989	13.5	388.9	24.800	0.973	11.7	432.3
18.850	0.932	13.8	387.3	21.850	0.984	14.7	374.7	24.850	0.971	12.3	431.5
18.900	0.901	15.4	385.9	21.900	0.962	14.0	386.4	24.900	3.374	22.5	234.0
18.950	0.938	14.5	378.1	21.950	0.938	13.1	387.3	24.950	3.502	23.5	195.1
19.000	0.932	15.1	377.3	22.000	0.947	15.0	380.4	25.000	3.531	23.0	159.8
19.050	0.960	16.2	370.6	22.050	0.939	13.4	390.0	25.050	8.659	52.6	208.8
19.100	0.929	15.7	361.6	22.100	0.963	13.6	391.0	25.100	8.439	49.7	208.7
19.150	0.914	15.6	358.8	22.150	0.992	15.1	373.5	25.150	8.604	50.8	213.4
19.200	0.930	15.9	366.6	22.200	0.980	14.8	384.1	25.200	9.302	53.8	222.5
19.250	0.898	16.1	364.7	22.250	0.966	17.7	396.2	25.250	9.813	55.5	213.8
19.300	0.941	17.0	390.7	22.300	0.977	17.0	394.5	25.300	10.619	58.7	217.7
19.350	0.977	16.8	377.4	22.350	0.952	15.6	369.7	25.350	11.210	63.7	217.0
19.400	0.952	17.2	361.4	22.400	0.931	15.0	390.7	25.400	11.291	77.0	211.7
19.450	0.938	16.3	359.8	22.450	0.915	18.7	389.1	25.450	11.423	54.6	201.8
19.500	0.923	15.3	353.5	22.500	0.904	24.3	391.5	25.500	11.333	55.3	213.4
19.550	0.931	16.4	362.1	22.550	1.244	27.9	307.5	25.550	11.718	61.5	220.5
19.600	0.929	16.0	357.0	22.600	1.715	26.9	387.3	25.600	11.516	62.5	211.1
19.650	0.920	15.3	361.3	22.650	1.785	29.2	388.4	25.650	11.199	64.9	206.5
19.700	0.929	15.2	372.0	22.700	1.240	28.2	304.2	25.700	10.334	60.2	201.7
19.750	0.924	14.8	359.4	22.750	1.709	24.3	311.0	25.750	8.538	55.3	190.7
19.800	0.924	16.1	364.7	22.800	1.498	20.7	328.9	25.800	7.490	47.7	190.5
19.850	0.927	15.4	363.0	22.850	1.172	24.9	336.1	25.850	7.391	41.9	206.4
19.900	0.910	15.4	375.6	22.900	0.996	19.5	345.0	25.900	7.424	42.5	213.4
19.950	0.914	14.9	375.3	22.950	0.979	14.6	321.5	25.950	7.478	44.3	215.7
20.000	0.923	14.6	380.3	23.000	0.989	11.6	339.0	26.000	7.616	41.0	220.0
20.050	0.929	15.1	373.2	23.050	0.984	10.2	356.6	26.050	8.738	44.2	224.8
20.100	0.940	16.2	375.1	23.100	0.972	9.9	361.7	26.100	8.830	49.9	223.4
20.150	0.922	16.7	376.1	23.150	0.991	11.8	372.4	26.150	8.591	54.7	218.0
20.200	0.929	16.1	380.5	23.200	0.966	12.1	378.3	26.200	9.197	59.5	229.8
20.250	0.931	18.9	381.0	23.250	0.950	10.5	381.1	26.250	9.773	55.2	227.0
20.300	0.968	18.4	371.2	23.300	1.023	11.9	373.8	26.300	11.161	63.5	192.8
20.350	0.961	18.0	376.2	23.350	1.076	12.1	324.1	26.350	10.223	45.7	240.8
20.400	0.985	17.3	384.7	23.400	1.019	11.6	346.9	26.400	10.308	46.6	222.5
20.450	0.964	16.1	390.9	23.450	1.045	9.3	343.1	26.450	10.780	49.0	224.8
20.500	0.927	15.6	382.5	23.500	0.982	9.2	364.2	26.500	10.890	53.9	224.8
20.550	0.925	15.0	389.4	23.550	0.966	10.3	380.0	26.550	11.156	49.0	230.4
20.600	0.938	15.1	394.1	23.600	0.966	10.2	398.1	26.600	10.804	50.2	231.7
20.650	0.948	14.8	389.1	23.650	0.936	10.0	408.1	26.650	10.067	53.1	231.7
20.700	0.960	14.9	386.8	23.700	0.936	9.4	417.5	26.700	9.371	54.6	232.6
20.750	0.936	15.2	376.9	23.750	0.914	9.8	423.2	26.750	9.156	49.6	231.7
20.800	0.936	15.0	378.7	23.800	0.920	10.9	417.3	26.800	9.317	46.8	241.0
20.850	0.945	14.7	381.4	23.850	0.933	10.7	420.6	26.850	10.046	49.4	255.6
20.900	0.964	15.1	390.4	23.900	0.940	11.2	416.1	26.900	11.297	50.0	256.9
20.950	0.957	14.9	368.4	23.950	0.972	11.4	422.0	26.950	12.024	56.9	259.3
21.000	0.952	15.5	384.2	24.000	0.950	11.3	425.1	27.000	12.326	60.4	247.9

Depth (m)	qc (MPa)	fs (kPa)	U2 (kPa)
27.050	12.156	61.3	246.1
27.100	11.844	58.9	242.7
27.150	11.276	59.9	237.1
27.200	10.333	53.2	238.1
27.250	9.802	52.0	237.7
27.300	9.835	49.1	247.7
27.350	9.982	49.9	250.0
27.400	10.476	49.7	250.0
27.450	10.522	49.5	249.6
27.500	10.628	49.3	250.1
27.550	10.595	48.1	250.0
27.600	10.614	48.0	251.9
27.650	10.706	51.0	254.7
27.700	10.733	65.6	252.3
27.750	10.783	47.1	251.4
27.800	10.217	44.9	260.5
27.850	10.995	44.7	261.5
27.900	11.370	47.9	261.5
27.950	11.374	50.9	255.4
28.000	11.654	52.8	258.9
28.050	11.731	54.1	256.7
28.100	11.222	55.5	252.3
28.150	10.196	52.7	240.8
28.200	9.299	46.4	236.6
28.250	8.869	41.9	247.7
28.300	8.494	44.9	250.6
28.350	8.330	40.7	254.6
28.400	7.835	48.8	245.4
28.450	7.733	47.5	276.3
28.500	8.224	38.8	249.7
.550	8.381	31.4	256.7
28.600	8.614	32.7	254.6
28.650	8.865	36.3	254.6
28.700	8.793	39.5	262.7
28.750	9.026	35.7	263.9
28.800	9.576	35.3	267.5
28.850	9.907	37.6	274.2
28.900	10.121	40.1	267.7
28.950	10.121	43.8	269.9
29.000	10.213	45.0	270.5
29.050	11.162	46.5	277.5
29.100	11.816	51.5	277.6
29.150	12.242	55.7	275.2
29.200	13.510	60.7	279.9
29.250	14.251	56.9	277.5
29.300	14.813	64.5	270.8
29.350	14.352	66.7	277.5
29.400	12.591	64.6	249.7
29.450	11.950	56.9	248.4
29.500	12.217	51.5	261.3
29.550	12.474	57.7	266.0
29.600	12.923	82.0	272.5
29.650	10.282	61.6	275.2
29.700	10.206	46.8	252.3
29.750	9.764	39.1	263.7
29.800	9.755	39.2	266.5
29.850	9.916	39.6	270.6
29.900	9.857	43.8	277.5
29.950	10.127	38.8	279.8
30.000	10.374	38.9	282.0