Determining the True Distributions of Load in Instrumented Piles

Bengt H. Fellenius

1905 Alexander Street SE
Calgary, Alberta
T2G 4J3

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Determining the True Distributions of Load in Instrumented Piles

Bengt H. Fellenius, Dr.Tech., P.Eng.

Abstract

Tests on piles—driven or bored—sometimes include instrumentation for determining the load distribution. The instrumentation can range from a telltale or two through a sophisticated array of strain gages, and from tests with the limited purpose of separating shaft and toe resistances through tests for the purpose of detailing the shaft resistance distribution along the pile. Most of the time, the measurements are analyzed from the assumption that the "zero readings", which are the readings taken at "zero" time, i.e. at the outset of the test, also have registered "zero" load. This assumption is more than a little off. It neglects the existence of locked-in loads—residual load—in the pile and is one of the sources of the myth of the so-called "critical depth". Neglect of the residual load distribution is also the main reason for conclusions of instrumented tests that suggest shaft resistance to be smaller when the pile is loaded in tension as opposed to when it is in loaded in compression. Neglecting residual load negates the primary objective of instrumenting a test pile. This paper presents examples of measured distributions of residual load and true resistance, and indicates how to determine the distribution of residual load from measurements when the residual load distribution is not measured directly.

Introduction

More often than not, basing a pile foundation design on only the ultimate resistance of the pile is not enough, the designer must also know the distribution of ultimate resistance along the pile, so that the long-term load distribution can be considered in the design. Of course, with knowledge of the soil profile and having determined appropriate soil parameters in the laboratory, a theoretical calculation can be made to establish the distribution within upper and lower boundaries. However, such boundaries are often impractically wide requiring the resistance distribution to be confirmed in full-scale static loading tests on instrumented piles. In analyzing data from such tests, it is common to assume that each test starts at zero conditions, that is, no prior history exists of load, movement, strain, and stress in the pile. However, the installation locks in stress in the pile and additional locked-in stress in the pile is caused by the subsequent reconsolidation and other time-dependent phenomena. Such locked-in stress and strain, also called "residual loads", must be considered in the analysis of the test data, or erroneous conclusions will be drawn from the test results.

Residual load will always develop in a pile, be it a driven or a bored pile and this understanding is not new. Early discussions of consequences of the residual load were presented by Nordlund (1963), Hunter and Davisson (1969), Hanna and Tan (1973), Holloway et al., (1978), Briaud (1984), and others. The following case histories show examples of measured distributions of load and resistance and how to analyze test data to determine residual loads as to magnitude and distribution.

Direct Measurements of Residual Load for Piles Driven in Sand

Gregersen et al. (1973) performed static loading tests in Drammen, Norway, on four instrumented, 8 m and 16 m long, 280 mm diameter precast concrete piles driven into a very loose sand. The case history of Gregersen et al, (1973) is remarkable as it both includes measurements of residual load before the start of the static loading test and of the true load distribution in the piles at the ultimate load.

Fig. 1 shows the soil profile at the site in the form of CPT cone stress values and SPT N-indices. Total density is indicated as 2,000 kg/m^3, natural water content is 18 % to 20 %, and the void ratio is 0.5. The groundwater table lies at a depth of 3.2 m.

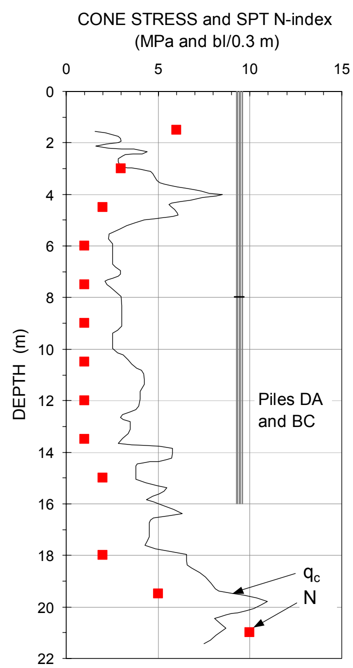


Fig. 1 CPT and SPT profiles at the Drammen site (Data from Gregersen et al., 1972)

The piles denoted Piles A, B, DA, and BC, were made up of four 8 m long segments denoted A, B, C, and D. Piles DA and BC were made up of Segments D and A, and B and C, respectively, where the lower segments consisted of Segments A and B (i.e., Piles A and B), respectively. Segment B was tapered to a 200 mm pile toe diameter, resulting in a taper of 8 mm/m. (Tapered piles usually have a 2.5 times larger taper, about 200 mm/m—0.25 inch/foot). The piles were instrumented with several levels of vibrating-wire strain-gages cast in the piles. A load cell was placed at the pile toe. The gages were calibrated together with the pile prior to the pile driving. On completion of the testing, the test piles were extracted and the pile and gages were recalibrated.

Of interest here are the results from Piles DA and BC. The distributions measured in Piles DA and BC immediately before the static tests are shown in Fig. 2A. Both distributions are characterized by that the load in the pile increases below the pile head due to progressively increasing negative skin friction. At a depth of about 6 m or slightly below, a gradual reduction of negative skin friction and transition to positive shaft resistance begins. An equilibrium (neutral plane) between downward and upward acting forces exists at a depth of about 10 m below which the transition continues with increasing positive shaft resistance.

The values of locked-in toe resistance—residual toe load—is the same for both piles, 50 KN. The unit negative skin friction along the upper about 6 m length of the piles corresponds to a beta-coefficient of 0.35 in an effective stress analysis.

The static loading tests on Piles DA and BC reached ultimate resistances for loads applied to the pile head of 510 KN and 450 KN, respectively. The toe resistance of Pile DA, starting from the locked-in value of about 50 KN, increased linearly to 110 KN. The toe resistance of Pile BC increased to only 70 KN. The values of total shaft resistance for Piles DA and BC were 400 KN and 380 KN, essentially equal in magnitude.

A 400-KN ultimate shaft resistance value corresponds to a beta-coefficient of 0.20 determined in an effective stress analysis. Obviously, the loading of the piles, which included several unloading and reloading cycles, caused a degradation of the shaft resistance. The reduction of the shear stress was about 40 %. The unloading and reloading cycles also resulted in a scatter of the strain-gage determined loads in the piles, making it difficult to give full weight to each individual gage value. However, the trend of values agrees quite well with the results of the effective stress analysis of the load distribution.

The results at the ultimate resistance of Pile DA are presented in Fig. 2B, showing the <u>measured</u> resistance distribution labeled "True" and a curve fitted to the data by an effective stress calculation, as well as the <u>measured</u> distribution of residual load.

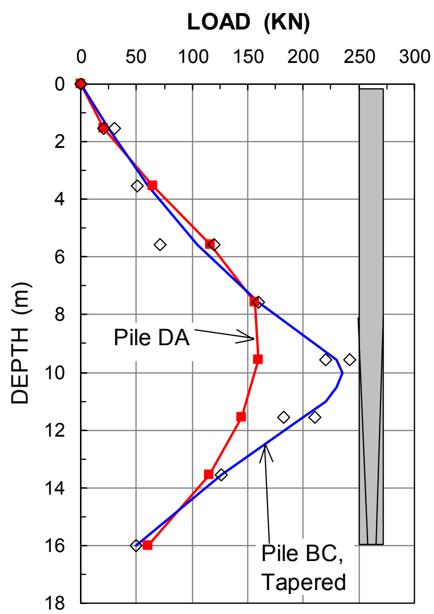


Fig. 2A Distribution of residual load in Piles BC and DA before start of the loading test (Data from Gregersen et al., 1972)

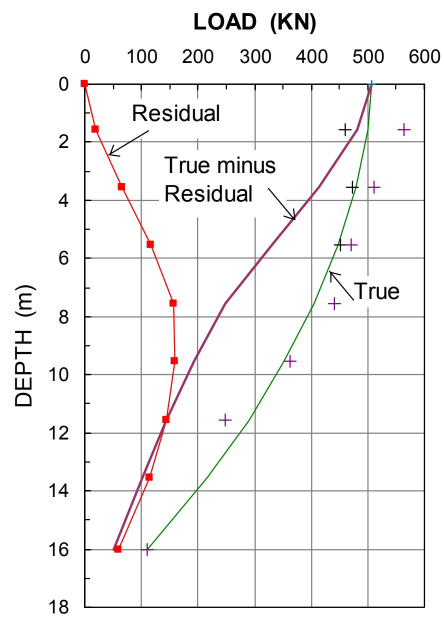


Fig. 2B Load and resistance distributions in Pile DA

Subtracting the residual load from the true load distribution results in the distribution labeled, "True minus Residual". The latter is the distribution that would have been found if the gages had been zeroed before the start of the test. The "squeezed-S" shape of this curve is typical of test evaluations that disregard the existence of residual load. Note also that with a bit of good will a "critical depth" is discernable at about 7.5 m depth. Indeed, as discussed by Fellenius and Altaee (1994), one of the reasons for the critical depth fallacy is the neglect of residual load in the interpretation of test data.

O-Cell Test on Piles Driven in Sand

Determining the residual load in a pile, driven or bored, is a routine part of the Osterberg sacrificial jack method of testing (the O-Cell method). The O-Cell test is a static loading test performed by applying load with a sacrificial jack placed at the toe of the pile while measuring the upward movement of the pile shaft and the downward movement of the pile toe (Osterberg 1998). Usually, the pile is instrumented with vibrating-wire strain-gages.

To illustrate how an O-Cell test measures both residual load and true resistance distribution, results of a test reported by McVay et al. (1999) are used. The case involves an O-Cell test on a 457-mm, 11.9 m long, prestressed, square precast concrete pile driven with a Delmag D46-23 hammer in Vilano Beach, St. Augustine, Florida. The soil profile consists of silty sand to a depth of about 7 m followed by sand. The SPT N-indices indicate that the sand is loose to compact in upper about 3.6 m and compact between 3.6 m and 10.5 m. Fig. 3 presents the SPT N-indices at the site together with the cone stress, q_c, determined in a cone penetration test, CPT. The groundwater table is located at a depth of 1.8 m.

The load-movement curves for the shaft and the toe of the pile are shown in Fig. 4. The buoyant weight of the pile, 44 KN, has been subtracted from the shaft load values.

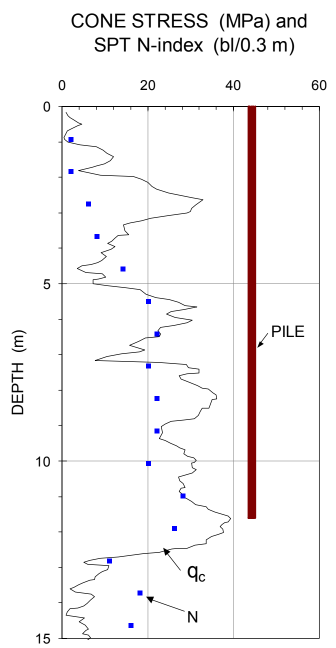


Fig. 3 CPT q_c and SPT N-index profiles at the Vilano Beach site

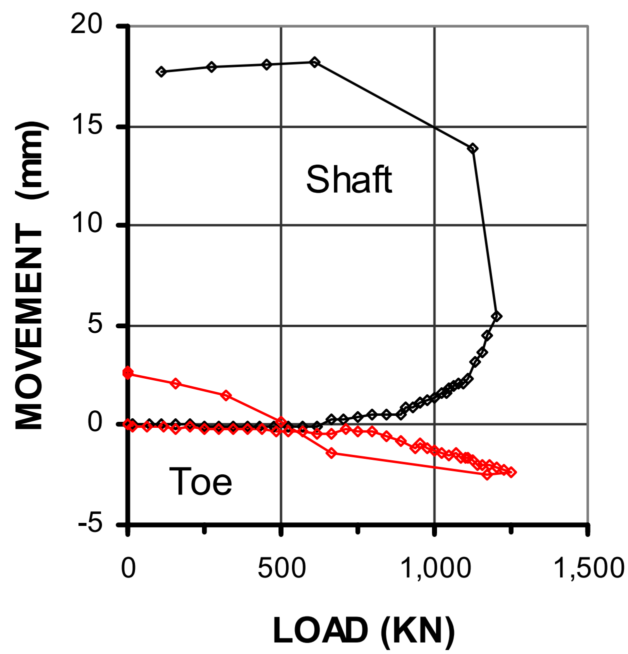


Fig. 4 Load-movement curves for pile shaft and pile toe at the Vilano Beach site

The test was run until the ultimate shaft resistance was reached (at an O-Cell load of 1,230 KN). As Fig. 4 shows, the shaft response is that of strain softening, i.e., the pile shaft resistance reduced after its peak value. The release of the pressure in the O-Cell unloaded the pile toe to zero load. The subsequent pile-toe elastic rebound curve was about parallel to the latter part of the loading-up curve, and, at the same time, the rebound length was larger than the toe movement from start to end of the test. This is a clear indication of the existence of a pile toe load at the start of the test, i.e., residual toe load. The value of this residual load is approximately where the backward extrapolation of the unloading toe load-movement curve intercepts the load axis, that is, about 700+ KN. A more precise value can be obtained from the measured separation of the upper and lower steel plates in the O-Cell presented in Fig. 5. The separation ("extension") occurred at an O-Cell load of 754 KN, which value includes the buoyant weight of the pile, 44 KN.

The pile was instrumented with several levels of vibrating wire strain gages. One of these was placed immediately (0.45 m) above the O-Cell. The O-Cell load values can therefore be used to calibrate the Young's modulus of the pile in a tangent-modulus approach (as described by Fellenius 1989; 2001). As shown in Fig. 6, the calibration analysis indicates that the modulus is constant and equal to 31 GPa. Because of the small load range, about 900 KN to 1,200 KN, no stress or strain dependency can be discerned. The consistency of the tangent modulus plot (Fig. 6) and the agreement between load increments in the O-Cell and load increments calculated from the gage immediately above the O-Cell indicate good quality data.

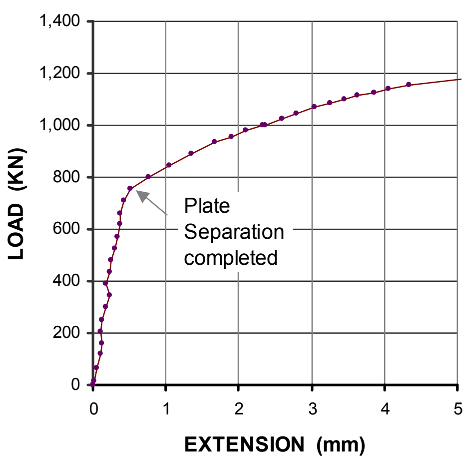


Fig. 5 O-Cell Load-Extension

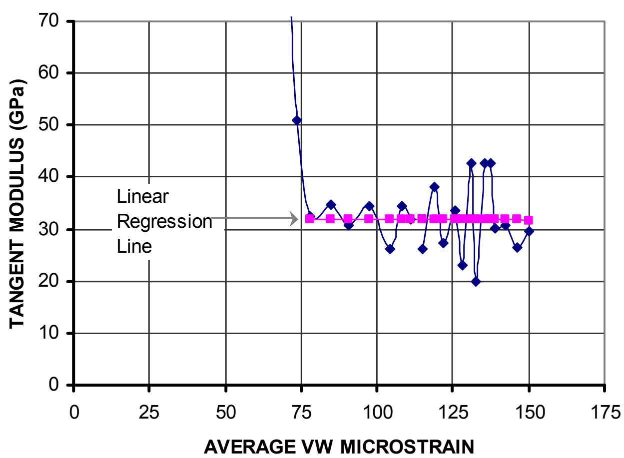


Fig. 6 Tangent modulus versus average strain

The total loads calculated from the measured strains and the O-Cell loads are presented in the load distribution diagram shown in Fig. 7. Above the Elev. –4 m level, the load distributions appear to indicate that almost no shaft resistance developed during the test. It would be easy to conclude that there was no soil resistance along this length of the pile. However, this would be false. The gage levels at Elev. –4 m and above are located where the pile is subjected to more or less fully developed negative direction forces. In this zone, therefore, the upward force of the O-Cell does not add any appreciable new load to the pile and, therefore, no appreciable change of strain can develop.

The fact that the sand is uniform can be used to determine the true load distribution in an effective stress analysis. However, the test was carried out already eight hours after the end of driving. It is therefore likely that pore pressures remain in the silty sand and as these are unknown, the results of an effective stress analysis would not be correct. In the lower sand, under the assumption of hydrostatic pore pressure distribution, the beta-coefficient is about 1.3, which is a high value.

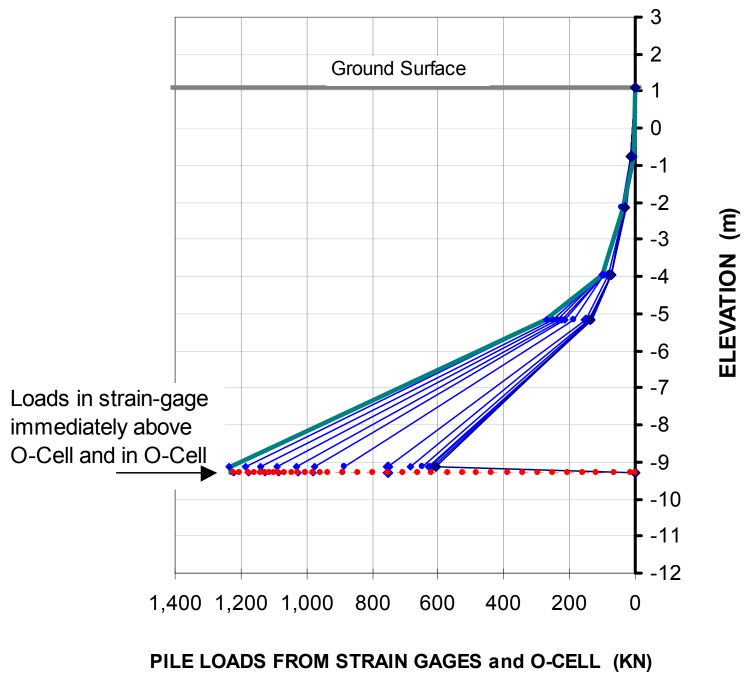


Fig. 7 Measured load distribution

Fig. 8 presents the measurements in four curves:

1 the true shaft resistance distribution (to the left of the vertical zero line)

2 the resistance distribution plotted from the total ultimate resistance, i.e., twice the maximum O-Cell load, like in a conventional "head-down" test

3 the change of load measured during the test

4 the residual load measured in the test.

As seen, very little change developed at the uppermost three gage levels. As mentioned, this is because the load in the pile did not change due to that negative direction shear forces were here already fully mobilized by the residual load. A slight change was measured for the fourth gage level, indicating that additional shaft resistance developed here. Most of the resistance developed between the O-Cell and the Elev.-4 m gage level.

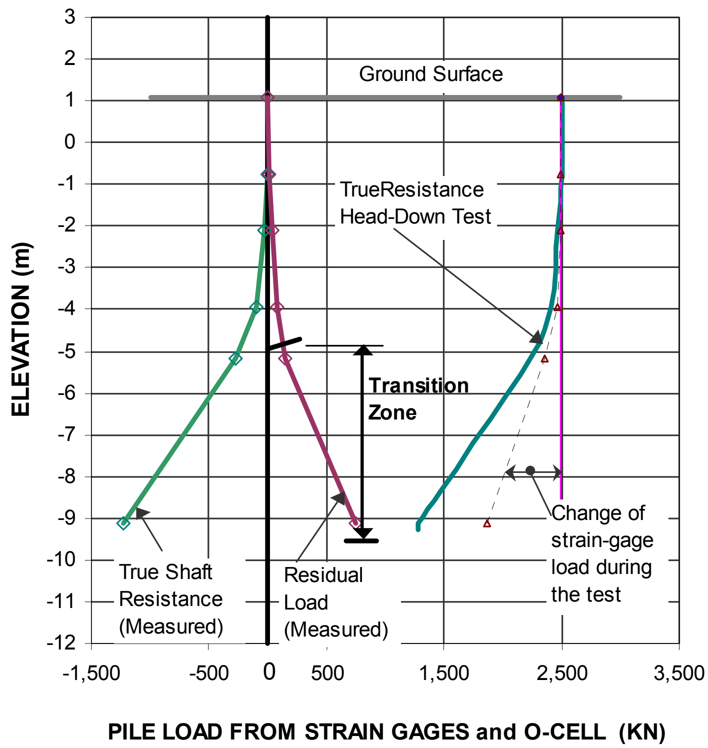


Fig. 8 Load and resistance Distributions

The measured distribution of residual load indicates that the pile is subjected to negative skin friction along its entire length (in contrast to the results presented for Piles DA and BC in Fig. 2). As concluded from the curve "change of load", the negative skin friction is fully developed down to about Elev. -4 m. Hereunder and to the pile toe, the "transition zone", the negative skin friction is only partially mobilized.

Dynamic and Static Tests on Piles Driven in Sand

The two cases presented in the foregoing involved measurements of the residual load and the true shaft resistance distribution. For tests where the measurement gages have been zeroed immediately before the test, only the change of strain is available for evaluation. If residual load is present in the pile, and it is in most cases, a test evaluation not including the influence of the residual load will be in error**:** In a head-down test, the shaft resistance will be overestimated where negative skin friction occurs and underestimated where positive shaft resistance occurs. The pile toe resistance will be underestimated. Where the negative skin friction is fully mobilized, i.e., above the transition zone, the shaft resistance can be mistaken as double the true value. Where the positive shaft resistance is fully mobilized, i.e., below the transition zone, the measurements will appear to indicate that no resistance exists. As indicated in the example case, in an O-Cell test, such a shaft resistance evaluation could lead to similar mistakes, although the direct measurement of the toe resistance will normally put the evaluation back on track.

What then to do if zero drift in the gages or other matters have made it impossible to obtain reliable measurements of the conditions before the start of the loading test? A typical such test is the dynamic test. The resistance distribution for the tested pile determined in a CAPWAP analysis does not include the residual load (notwithstanding that the analysis has an option for considering the load locked-in from the preceding blow). An example of such a case is presented in the following.

Axelsson (1998) performed dynamic and static testing on three 235 mm square precast concrete piles driven to a depth of 19 m into a uniform fine to medium sand deposit at Fittja, Sweden. Fig. 9 shows the results of a CPT test performed at the site.

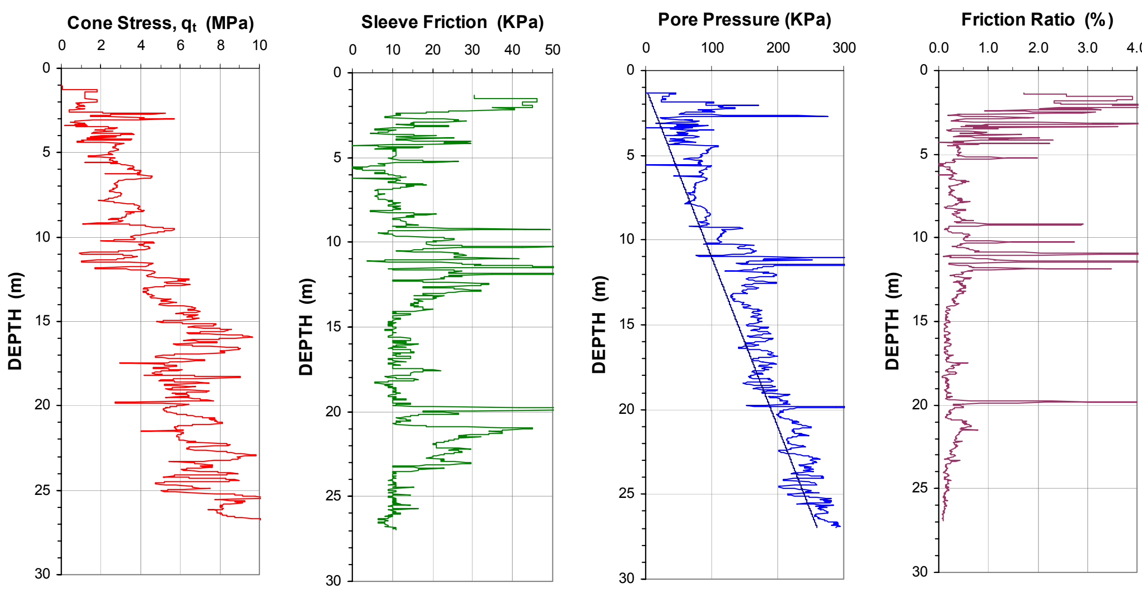


Fig. 9 CPT q_t-Profile at the Fittja Site

The piles were driven with a 4,000-kg drop hammer with a height-of-fall of 200 mm and restruck repeatedly after completed installation. The pile driving was monitored by means of the Pile Driving Analyzer (PDA) and the pile capacities were determined by a CAPWAP analysis performed on the last blow of the End-of-Initial-Driving (EOID) and on the first blow from four restrike events (Only one or two blows were given at each restrike event. For one pile, the analysis was made on the second blow) at 1 hour, 1 day, 6 days, 37 days, and 143 days after EOID. The penetration for each restrike blow was about 5 mm.

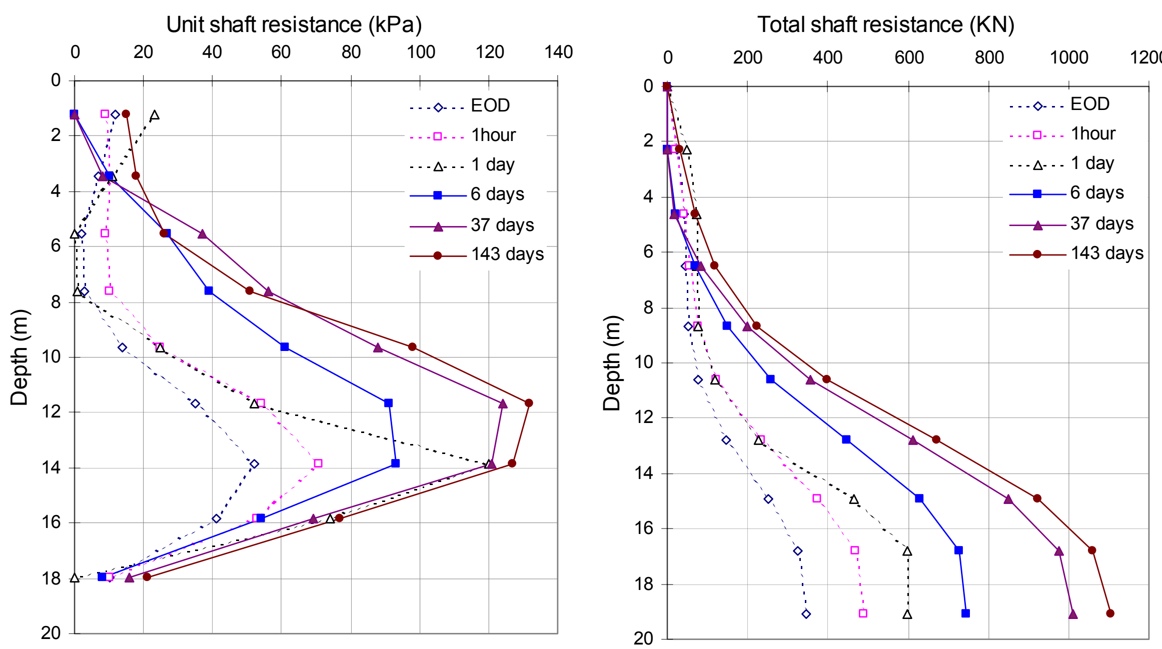
The CAPWAP determined capacities were:

Event	EOD	1 hour	1 day	6 days	37 days	143 days
Capacity	560 KN	703 KN	890 KN	1,247 KN	1,354 KN	1,441 KN
Shaft Res.	344 KN	492 KN	595 KN	746 KN	1,016 KN	1,097 KN
Toe Res.	216 KN	211 KN	295 KN	501 KN	338 KN	344 KN

The results of the CAPWAP analyses showed an increase of capacity for each restrike event. As the excess pore pressures due to the pile driving would have dissipated within the first minutes or hours of EOID, it would appear that the increase of capacity was due to time-dependent gain.

Axelsson (1998) presented the distribution of unit shaft resistance per pile element used in the CAPWAP analysis in a diagram similar to the one shown Fig. 10A. The corresponding distribution of total shaft resistance is shown in Fig. 10B.

The Fig. 10 curves appear to support a conclusion of time-dependent increase of shaft resistance. In both diagrams, however, the three curves showing the results for the analyses from the 6th day after EOD are essentially parallel, except for the portions the between depths of about 12 m and 16 m, i.e., 15 m of the 19 m total length. Where such curves are parallel, the unit shaft resistances are equal. The explanation to the conundrum becomes obvious if the data plotted in the form of a resistance distribution curve as shown in Fig. 11.



A Unit shaft resistance determined by CAPWAP analysis. Values are plotted at midheight of each element

B Shaft resistance distribution Values are plotted at the lower boundary of each element

Fig. 10 Results of CAPWAP analysis of 19 m long pile at the Fittja site (data from Axelsson, 1998)

The curves shown in Fig. 11 are practically parallel, except for the portion between the depths of about 12 m and 16 m. Moreover, the curves show a "squeezed-S" shape similar to those of Fig. 2B. In fact, the measured distributions are typical of piles subjected to residual load.

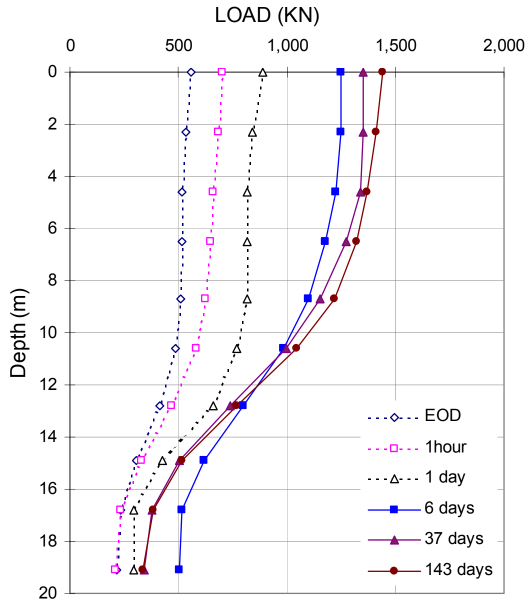


Fig. 11 CAPWAP-determined resistance distributions for the 19 m long pile

The test data are of exceptionally good quality and allow a detailed analysis of the magnitude and distribution of the residual load in the pile, as follows. The small downward soil movement always occurring after the construction of a pile develops negative skin friction along the upper length of the pile. The corresponding dragload results in a small downward movement of the pile that is resisted by positive soil resistance in the lower portion of the pile. Because shaft resistance is more or less fully developed at very small movement, about a millimetre, the negative-direction shear forces along the upper portion and, usually, also the positive-direction shear forces along the lower portion of the pile can be considered fully mobilized. Similar to that shown in Figs. 2 and 8, a transition zone of some length exists between negative and positive direction forces. In contrast to driven piles, bored piles frequently exhibit only a small residual toe load. For driven piles, each blow forces the pile toe against the soil and, in unloading, the toe load is not fully released, while for a bored pile the load locked-in by the construction is usually limited to the weight of the fluid concrete. Therefore, in case of a driven pile, the small consolidation movements along the shaft start to increase the load at the pile toe immediately. In contrast, the bored piles require a larger movement before a residual toe load can develop. Moreover, a bored pile can have a more or less distinct layer of softened soil immediately below the pile toe that the concrete was not able to displace. This layer has to be compressed before a toe resistance, residual and otherwise, can be developed.

When, as in the example case, one has reason to believe that the measurements are influenced by residual load, one can be reasonably sure that the residual load includes fully mobilized negative skin friction along the upper portion of the pile. Along this length, therefore, the measured distribution shows a shaft resistance that is twice the true resistance; distribution of the true resistance is simply a curve twice as steep as the measured. (For a discussion on why the magnitude of shaft resistance is independent of direction of movement, see Fellenius, 2001). In and below the transition zone, no similar method of evaluation exists. However, if the length above the transition zone is reasonably long, the there-determined true curve can be determined in an effective stress back-calculation of the beta coefficients (with input of the basic soil parameters and pore pressure distribution). Provided that the soil is homogeneous, the assumption is then made that the effective stress strength parameters are also valid in and below the transition zone for an extrapolation of the distribution of the "true" resistance. The difference between the so-determined true resistance and the measured curve is the distribution of the residual load. The three load distribution curves, i.e., the measured loads, the residual loads, and the true loads, are interdependent and the analysis is based on establishing agreement between the calculated and measured loads.

The data for the 143-day distribution are analyzed using the mentioned approach. Fig. 12 shows the results: the measured (143 days) and the true resistance distributions at the ultimate load (i.e., the CAPWAP determined capacity), as well as the difference between the two, the residual-load distribution. The true distribution curve matched to an effective stress calculation of shaft resistance results in a beta-coefficient, β, of 0.3 to a depth of 11 m and β = 0.6 below this depth. (The ratio between the two coefficients is approximately the same as the ratio between the cone values for these depths, see Fig. 9). Based on the assumption that the soil parameters determined for the 11 m through 13 m depths also apply to the continued length of the pile, the true resistance distribution is extrapolated to the pile toe. The residual load distribution is the true minus the residual.

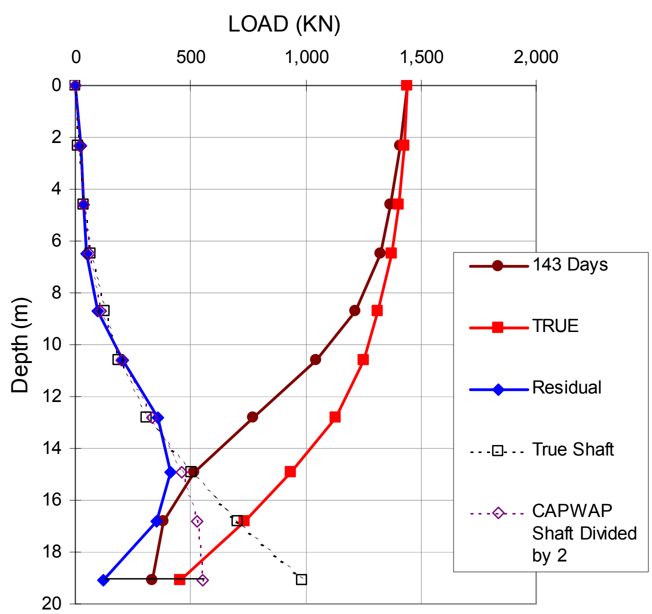


Fig. 12 Measured 143-day and true load distributions with distributions of residual load and shaft resistance

The same analysis made on all measurement events, but for the end of initial driving event, results in true distributions very similar to that shown in Fig. 12. The distributions of residual load will be slightly different, though. Mostly, the early tests show a longer transition zone and a smaller residual load. The analyses indicate that the increase of total capacity is mainly due to an increase of toe resistance for each restrike event concealed by the increasing residual load.

Distribution and magnitude of the residual load along the pile depends on the magnitude of the residual load at the pile toe. For each restrike event, the pile mobilized a larger toe resistance and the locked-in load is therefore larger. This has the effect of moving the neutral plane (level of equilibrium between negative and positive direction shear forces) downward in each test, which causes the total shaft resistance to appear larger for each test, that is, were the CAPWAP value taken as the true value. While it is possible, indeed plausible, that the sand at the site could exhibit some time-dependent increase of capacity, the test data do not show this.

Axelsson (1998) also reports static loading tests performed at the site on a 12.8 m long pile having the same diameter as the 19 m piles and installed by the same hammer. The 12.8-m pile was subjected to static loading tests performed 1 day, 8 days, 4 months, and 22 months after completion of pile driving. (At the 4-month test, the pile was reloaded immediately on completion of the first loading; The 22-month test results are reported in Axelsson, 2000). The load movement curves of the 12.8-m pile are shown in Fig. 13. Fig. 13A plots the test data from a common origin. This manner of presenting the data appear to show an increase of capacity with time after driving. However, when plotting the results according to stress and movement history, as shown in Fig. 13B, a different explanation emerges for the increase of capacity**:** Simply, each static test only continues where the preceding test left off, much the same way as the dynamic tests did for the 19-m pile. Note, to the embedment depth of 12.8-m, the data for the 19 m pile showed no increase of resistance between the test events (Fig. 11).

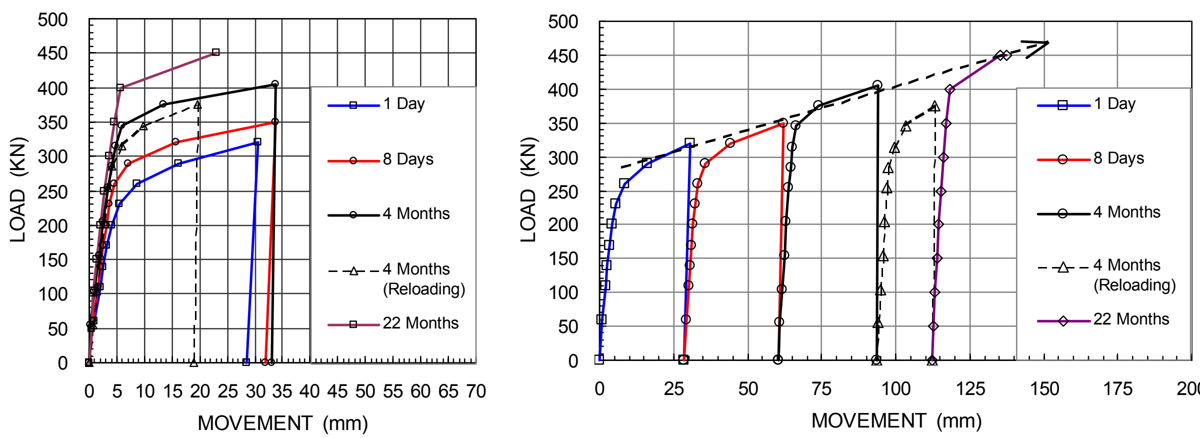


Fig. 13 Results from static loading test on a 12.8 m pile
(Data from Axelsson, 1998)

Test on Instrumented Bored Piles

To illustrate the application of the analysis procedure, data from static loading tests on two bored piles will be presented. Baker et al. (1990) performed a series of tests sponsored by the FHWA on 0.9 m diameter bored piles in clay and sand. (The tests are also reported by Briaud et al., 2000). Of interest for the subject topic are the static loading tests on two piles denoted Piles 4 and 7. Pile 4 was 10.4 m long and Pile 7 was 9.5 m long. Pile 4 was drilled "wet" with a bentonite slurry displaced by the concrete and Pile 7 was drilled "dry".

The soil at the site of Pile 4 consists to a depth of 8 m of loose to compact silty sand and of dense clean sand between 8 m and 12 m. The soil at the site of Pile 7 consists of stiff silty clay. The soil profiles at the two FHWA sites are illustrated in the CPT and SPT profiles shown in Fig. 14.

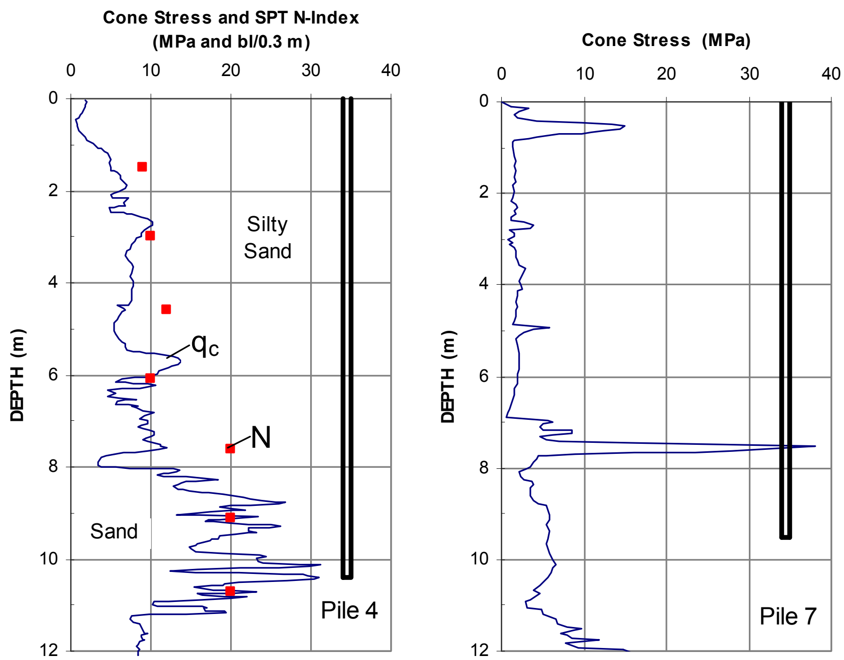


Fig. 14 CPT q_c and SPT profiles at the FHWA sites

The piles were instrumented with strain gages. Pile 4 had gages at three levels and Pile 7 had gages at two levels. The measured load distributions in Piles 4 and 7 were determined by connecting the values of load applied to the pile head and the measured loads as shown in Figs. 15A and 15B, respectively. At first glance, it would appear as if either or both of the two lowest level strain gages in Pile 7 are faulty, as they indicate that there is no shaft resistance in the lower two-thirds of the pile. Also the gages in Pile 4 seem to be suspect, as they indicate that the unit shaft resistance along the lower portion of the pile is smaller than along the upper portion. However, this is not so. The data are typical for piles subjected to residual load.

The method described in the foregoing was applied to the test data and gave the distribution of true resistance and residual load shown in Fig. 15A and 15B. Note the similarity between the measured load distribution in Fig. 15B and the "true minus residual" distribution in Fig. 2.

Fig. 15A shows the results from Pile 4. The analysis indicates a total shaft resistance of about 2,500 KN (of the 4,150-KN capacity) corresponding to unit shaft resistance values of about 60 KPa at the ground surface increasing approximately linearly to about 80 KPa near the pile toe. The toe resistance is about 1,650 KN and the residual toe load is 1,150 KN. The true resistance distribution for the pile corresponds to a total stress analysis applying a unit shaft resistance of 60 KPa at the ground surface and 90 KPa near the pile toe. Alternatively, the distribution can be fitted to a constant unit shaft resistance of 50 KPa coupled with a beta-coefficient of 0.25 per the classic Coulomb relation.

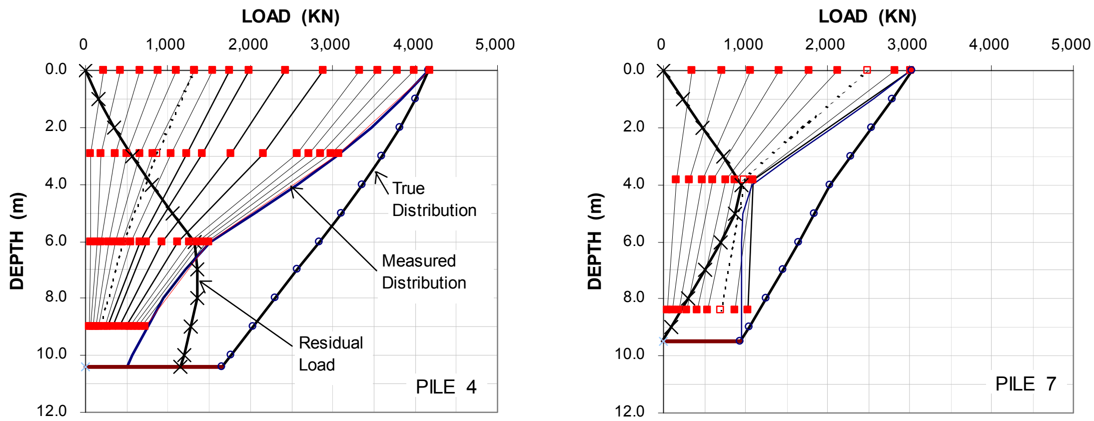


Fig. 15 Results of static loading tests on Piles 4 and 7 at the FHWA site: Distributions of measured load, residual load, and loads corrected for residual load (Data from Baker et al., 1990)

Fig. 15B shows the results from Pile 7. The analysis indicates a total shaft resistance of about 1,900 KN and the toe resistance is about 1,000 KN. The shaft resistance corresponds to values of about 60 KPa at the ground surface increasing linearly to about 80 KPa near the pile toe. These shaft resistance values are similar to those determined for Pile 4 despite the different construction procedures and soil conditions. No or only very little residual toe resistance is discernable by the analysis, However, a considerable residual load in the pile still develops along the pile shaft.

The case history is of particular interest because the analysis belies the common belief that bored piles cannot exhibit residual load.

Conclusions

The case histories presented deal with driven and bored piles, piles in sand and in clay, static tests and dynamic tests, and conventional head-down tests and O-Cell tests.

It is obvious from the shown examples that if no consideration is made with regard to the residual loads, correct conclusions cannot be drawn about the magnitude and distribution of shaft resistance measured in a pile test. Therefore, neglecting the residual load negates the primary objective of instrumenting a test pile.

Residual load is associated with negative direction shear forces along the upper length of the pile and positive direction shear forces in the lower length of the pile plus some toe resistance. A transition zone of some length exists between the two shear force directions.

For a pile instrumented to determine the change in load at certain levels along the pile tested in a “head-down” test, the true load distribution can be determined based on the assumption that the negative direction shear forces are fully mobilized from the pile head down to the beginning of the transition zone. Over this upper length, the true distribution has a slope exactly half of the measured slope, which establishes the basic soil parameters to use in a back-calculation of the distribution of shaft resistance. It also establishes the distribution of residual load over this length of pile, as the measured load distribution, the true resistance distribution and the distribution of residual load are interrelated.

If the soil profile is reasonably uniform, the soil parameters determined from the upper length of the pile can be assumed to apply also along the transition zone and all the way to the pile toe. Some judgment is of course necessary and simple rules exist to assist the delineation of the true resistance distribution. If the residual load is fully mobilized (determined as the difference between the calculated "true distribution" and the measured distribution) in the lower portion of the pile (positive direction forces), the true distribution and the residual distribution are parallel. If it is not fully mobilized, the slope of the true distribution can never be steeper than the slope of the distribution of residual load along this length of the pile (because the unit shaft resistance for the true distribution cannot be smaller than the unit shaft resistance for the residual load distribution). The latter condition governs a boundary for what soil strength value to use for the soil near the pile toe. Another boundary is determined by that the residual load cannot be negative.

The O-Cell measurements offer the advantage of determining both the magnitude of the total shaft resistance and the value of the residual load acting at the pile toe. In case of an instrumented pile tested in an O-Cell test (which engages the shaft resistance in the negative shear force direction), gages located where the residual shear forces act in the negative direction will show no or very small increase of load during the test. Larger changes will be measured in the transition zone and over the length where the shear direction is positive.

An O-Cell test on an instrumented pile should preferably be followed by a head-down test with the O-Cell open so that no toe resistance is mobilized. If so, analysis of the gage measurements will establish the true shaft resistance distribution along the pile length along with a confirmation on the total shaft resistance. Conclusions are of course subject to judgment of shaft resistance degradation etc.

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