Results from long-term measurement in piles of drag force and downdrag

The Bäckebol site in June 1968

and in late 1969

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Results from long-term measurement in piles of drag load and downdrag

Bengt H. Fellenius

Abstract: Several full-scale, long-term tests on instrumented piles performed since the 1960s and through the 1990s are presented. The results of the tests show that a large drag load will develop in piles installed in soft and loose soils. The test cases are from Norway, Sweden, Japan, Canada, Australia, United States, and Singapore and involve driven steel piles and precast concrete piles. The test results show that the transfer of load from the soil to the pile through negative skin friction, and from the pile back to the soil through positive shaft resistance, is governed by effective stress and that already a very small movement will result in mobilization of ultimate values of shaft shear. The pile toe resistance, on the other hand, is determined by downdrag of the pile and the resulting pile toe penetration. Reconsolidation after the pile installation with associated dissipation of pore pressures will result in significant drag load. An equilibrium of force in the pile will develop, where the sustained loads on the pile head and the drag load are equal to the positive shaft resistance plus the pile toe resistance. The location of the force equilibrium, the neutral plane, is also where the pile and the soil move equally. The drag load is of importance mostly for very long piles (longer than 100 pile diameters) for which the pile structural strength could be exceeded. Downdrag, i.e., settlement of the piled foundation, is a very important issue, however, particularly for low-capacity short piles. Soil settlement at the neutral plane will result in a downdrag of the pile. The magnitude of the downdrag will determine the magnitude of the pile toe penetration into the soil, which will determine the pile toe resistance and affect the location of the neutral plane. Nature's iteration of force and soil settlement will decide the final location of the neutral plane.

Key words: piles, negative skin friction, drag load, downdrag, neutral plane, pile settlement.

Résumé : On présente plusieurs essais à pleine échelle et à long-terme de pieux instrumentés réalisés depuis les année 1960 jusqu'aux années 1990. Les résultats des essais montrent qu'une forte charge d'entraînement va se développer dans des pieux installés dans des sols mous et meubles. Les études de cas proviennent de Norvège, Suède, Japon, Canada, Australie, USA, et Singapour, et comprennent des pieux foncés cylindriques en acier et des pieux en béton préfabriqués. Les résultats d'essai montrent que le transfert de charge du sol au pieu dû au frottement superficiel négatif, de même que du pieu au sol dû à la résistance positive du fût est régi par la contrainte effective, et qu'aussitôt qu'un très faible mouvement se produit, les valeurs ultimes de cisaillement le long du fût va se mobiliser. Par ailleurs, la résistance à la pointe du pieu est déterminée par l'entraînement vers le bas du pieu et par la pénétration résultante du la pointe du pieu. La reconsolidation après la mise en place du pieu associée à la dissipation des pressions interstitielles va produire une charge d'entraînement appréciable. Un équilibre des forces dans le pieu va se développer lorsque les charges portant sur la tête du pieu et la charge d'entraînement sont égales à la résistance positive du fût plus la résistance à la pointe du pieu. La localisation de l'équilibre des forces – le plan neutre – se situe aussi là où le pieu et le sol bougent ensemble. La charge d'entraînement est importante principalement pour les pieux très longs, plus longs que 100 diamètres du pieu, pour lesquels la résistance structurale du pieu pourrait être excédée. Cependant, l'entraînement, i.e., le tassement de la fondation sur pieu, est un problème très important et particulièrement pour les pieux courts à faible capacité. Le tassement du sol au plan neutre va produire l'entraînement du pieu. La grandeur de l'entraînement va déterminer l'importance de la pénétration de la pointe du pieu dans le sol, qui va déterminer la résistance à la pointe du pieu et affecter la localisation du plan neutre. L'itération des force en nature et le tassement du sol vont décider de la localisation finale du plan neutre.

Mots clés : pieux, frottement négatif superficiel, charge d'entraînement, charge d'entraînement vers le bas, plan neutre, tassement d'un pieu.

[Traduit par la Rédaction]

Introduction

The 1960s saw an upsurge of interest in the use of full-scale instrumented piles to study the magnitude and development of negative skin friction due to soil settling around piles. Tests were performed in Norway, Sweden, Japan, and Canada. The author has revisited the pioneering papers pub-

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B.H. Fellenius. Bengt Fellenius Consultants Inc., 1905 Alexander Street SE, Calgary, AB T2G 4J3, Canada (e-mail: bengt@fellenius.net).

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lishing the results of these tests and presents here a summary of a reanalysis of the records. One of these tests was started in June 1968 by the Swedish Geotechnical Institute and consisted of recording load distribution in two instrumented, 300 mm diameter, single, precast concrete piles driven through a 40 m thick clay deposit and 15 m into an underlying sand deposit. The results obtained during the first 160 days and the first 1300 days were published by Fellenius and Broms (1969) and Fellenius (1971, 1972), respectively. The study continued beyond 1972 with the placing in September 1973 of an earth fill over a 41 m diameter circular area around the piles followed by frequent monitoring of forces in the piles and settlement until the site was closed up in 1983. The measurements taken until mid-1975 were presented by the Swedish Geotechnical Institute (Bjerin 1977). This paper reports the results until the end of the field test in August 1983, 5500 days after the start of the test.

Revisiting published full-scale tests

Norwegian Geotechnical Institute Sörenga site

In April 1962, the Norwegian Geotechnical Institute started a full-scale test on a telltale instrumented steel pile at Sörenga near Oslo (Bjerrum and Johannessen 1965). The test consisted of measurements on a 55 m long, 470 mm diameter pipe pile driven in an area to be reclaimed from the sea by means of a 10 m thick fill. The natural soil at the site consisted of a 43 m thick deposit of marine clay on bedrock, a calcareous schist. The natural water content of the clay was about 40% at the seabed, reducing to about 30% at the bedrock. The liquid limit was about 10% higher than the water content. The undrained shear strength ranged from about 20 kPa at the seabed to about 70 kPa at the bedrock.

The placing of the fill initiated a consolidation process in the clay deposit. Figure 1 presents the measured distribution of settlement, and Fig. 2 the distribution of excess pore pressures measured in April 1964, 2 years after the pile driving and start of placing the fill. Figure 2 also shows the calculated effective stress after full dissipation of the excess pore pressures. The difference, as indicated, is the effective stress existing in April 1964. The settlement resulted in negative skin friction along the test pile, accumulating to drag load in the pile. The drag load caused the pile to shorten, and Fig. 3 shows the pile shortening measured 1 year (May 1963) and 2 years (April 1964) after the pile driving. Bjerrum and Johannessen (1965) used the pile shortening to calculate the distribution of stress in the pile. The stress data have been converted to load in the pile and are presented in Fig. 4 (the load values are plotted, not at mid-point of each measuring length, L, but at $0.58L$ down from the upper end of the measuring length, L, as recommended by Fellenius (2002*a*, 2002*b*) for plotting of loads evaluated from telltale measurements).

As indicated in Fig. 4, a calculated drag load distribution for a constant beta (β) coefficient of 0.25 matches reasonably the measured drag load distribution of April 1964. Also included is the drag load distribution after full dissipation of pore pressures calculated using the same beta coefficient. A calculation using the undrained shear strength resulted in a drag load distribution very similar to that calculated for a beta coefficient of 0.25. In contrast with the effective stress calculations, however, the calculations using the undrained shear strength cannot be used to reproduce the measured changes in drag load for the actual distributions of excess pore pressure.

Bjerrum et al. (1969) reported that in 1967, after an additional 3 years of measurements, the maximum load in the pile and the settlement of the clay had increased to 4000 kN

Fig. 1. Profile and measured distribution of settlement at the Sörenga site. Data from Bjerrum and Johannessen (1965).

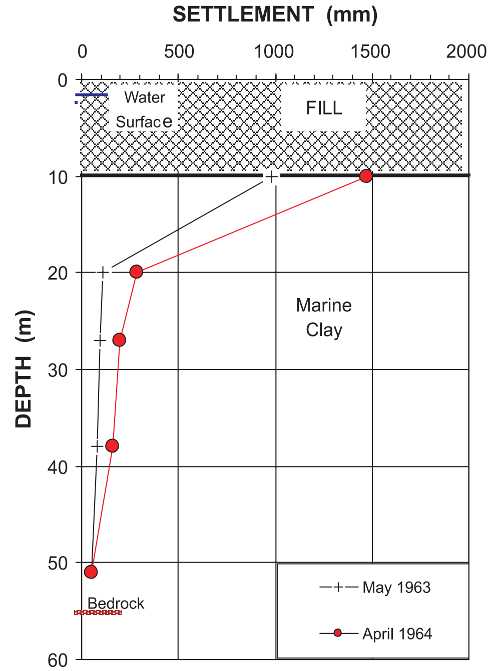


Fig. 2. Distribution of excess pore pressure (Δu) in April 1964 and calculated effective stress (σ_z') at the Sörenga site. Data from Bjerrum and Johannessen (1965).

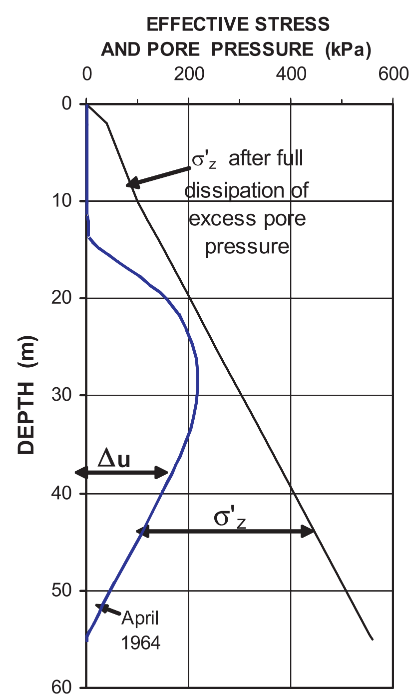


Fig. 3. Measured pile shortening at the Sörenga site. Data from Bjerrum and Johannessen (1965).

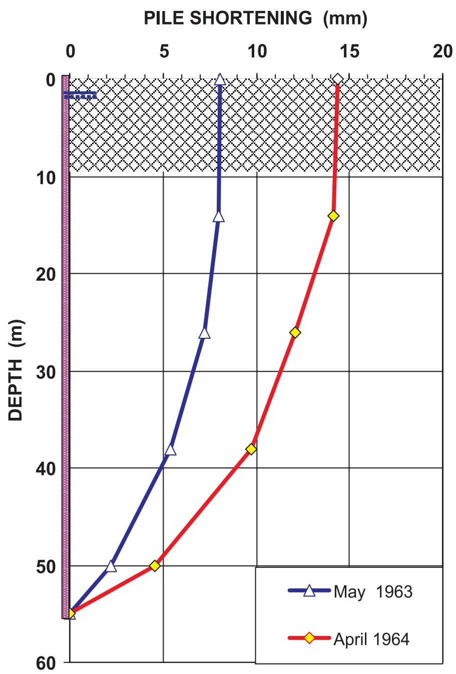
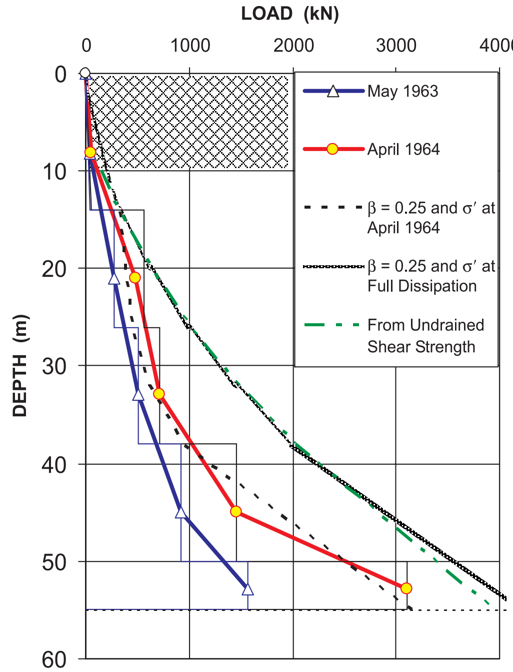


Fig. 4. Measured and calculated distribution of load at the Sörenga site. Data from Bjerrum and Johannessen (1965). β, beta coefficient; σ′, effective stress.



and 2000 mm, respectively, and that the movement of the pile toe into the calcareous schist, i.e., the downdrag, amounted to 100 mm. These observations mean that the neutral plane was located close to the pile toe, and the maximum load in the pile is also the toe resistance of the pile in the calcareous schist. As the maximum load of 4000 kN also corresponds to a stress of about 250 MPa, the structural strength of the steel, it is possible that instead of the pile toe being forced into the bedrock, the mentioned 100 mm toe movement is due to structural yielding of the steel pile at the pile toe.

Norwegian Geotechnical Institute Heröya site

In 1969, the Norwegian Geotechnical Institute published the results of additional full-scale investigations (Bjerrum et al. 1969) performed at three sites in 1962 and 1963. One of the additional test sites was located at Heröya in Oslo Harbor, where, to construct a dock, land was reclaimed from the sea by placing 6.5 m of fill over the seabed. The soil consists of an approximately 25 m thick layer of marine clay with a water content of about 30%, very close to the liquid limit, and is somewhat different from the clay at the Sörenga site. A gravel layer deposited on bedrock is below the clay at depths ranging from 30 to 35 m. Before the fill was placed, the undrained shear strength ranged from about 20 kPa at the top of the clay layer to about 40 kPa immediately above the gravel layer. After 2 years of consolidation, the undrained shear strength had increased to about 40 kPa at the seabed, but there was no change in undrained shear strength at the lower boundary of the clay layer.

In 1966, four 300 mm diameter, telltale-instrumented pipe piles were driven at the Heröya site. The piles were driven closed-toe with an enlarged size, 400 mm rock shoe, or "Oslo-point." The results from two of the test piles are presented in the following. The surface of one of the two test piles was treated with a 1 mm thick bitumen coating and the other was untreated. The settlement of the seabed amounted to 160 mm during the first year after the driving of the test piles. Similar to Fig. 2, Fig. 5 shows the measured excess pore pressures remaining in May 1967 and the calculated effective stress after full dissipation of the pore pressures. The difference is the effective stress existing in May 1967. Figure 6 shows the distribution of pile shortening measured in May 1967 for the bitumen-coated and uncoated piles. The bitumen is effective in reducing the shaft shear because of its viscous properties and inability to sustain shear forces and resist movement. A small strain shear rate will mobilize small shear resistance in the bitumen coat, sandwiched between the pile and the soil, which is less than the soil shear resistance (Fellenius 1975, 1979).

Figure 7 shows pile shortening measured during a 400 day period after the pile driving. The results demonstrate that the thin bitumen coat was highly effective in reducing, and almost eliminating, the drag load causing the shortening. In March 1967, about 300 days into the monitoring of the site, several widely spaced piles for the construction of the dock were driven within a distance of about 10 m from the test piles. The pile driving caused an increase of pore pressures in the soil and a corresponding decrease in effective stress. Coinciding with the decrease in effective stress, the piles lengthened, as indicated in Fig. 7. Settlement and heave were not measured near the test piles, and it can be argued that soil heave due to the pile driving would have contributed to the lengthening (unloading) of the piles. In the author's opinion, however, this is not likely. The reduced negative skin friction resulted in a reduction of the drag load and, consequently, a lengthening of the piles. As the pore pressures introduced by the pile driving dissipated, the effective stress returned to its previous level, as did the load in

Fig. 5. Distribution of excess pore pressure measured in May 1967 and calculated effective stress at the Heröya site. Data from Bjerrum et al. (1969).

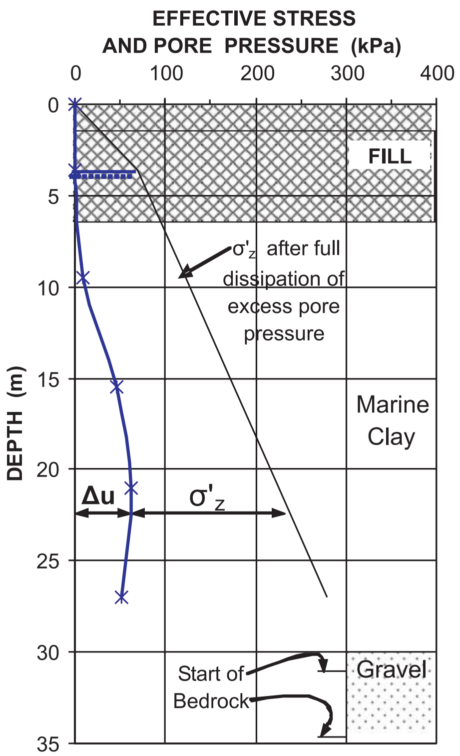


Fig. 6. Distribution of pile shortening in May 1967 at the Heröya site. Data from Bjerrum et al. (1969).

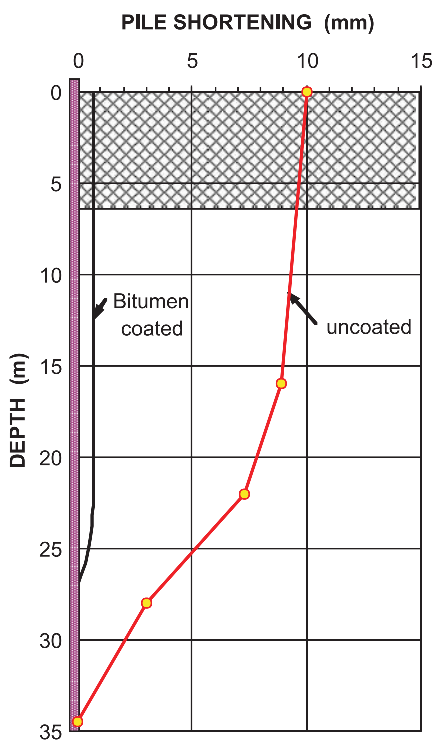
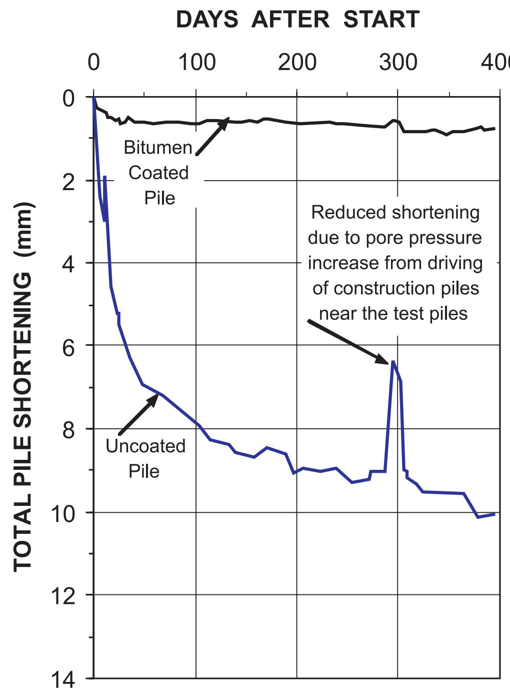


Fig. 7. Measured total pile shortening during May 1966 to June 1967 at the Heröya site. Data from Bjerrum et al. (1969).



the pile, and the piles shortened to the length they had before the construction piles were driven. This observation confirms the finding from the Sörenga test (Bjerrum and Johannessen 1965) that pile shaft shear in clay soil is a function of effective stress.

The pile shortenings measured in May 1967 were converted to load in the pile, and the load distributions for the two test piles are shown in Fig. 8 (plotted at the 0.58*L* point of each measuring length, *L*, indicated by the stepped curve). The load distribution of the uncoated pile shows how the negative skin friction accumulates to a maximum drag load value of about 1100 kN at about 25 m depth. Below this depth, the shaft shear turned to positive shaft resistance and the load in the pile decreased with depth to a mobilized toe resistance of about 550 kN. The negative and positive direction loads are in equilibrium. The location of the maximum value, the force equilibrium, is called the neutral plane. The paper by Bjerrum et al. (1969) is the first to show that the development of a force equilibrium is indeed the manifestation of load transfer for piles in a settling soil.

For comparison with the effective stress analysis, Fig. 8 also shows the distribution of drag load calculated for a beta coefficient of 0.30 applied to the effective stress of May 1967. The agreement is not as good as that shown for the Sörenga pile but is still remarkably good considering the crude telltale instrumentation system employed in the test.

Bjerrum et al. (1969) report that the ground surface settled 200 mm and the pile head settled 33 mm. Combined with about 4 mm of pile shortening at the neutral plane, this means that the settlement at the neutral plane, and therefore also the downdrag of the pile, was 29 mm, and the pile toe penetration into the gravel deposit above the bedrock was about 25 mm, or about 6% of the pile toe diameter (resulting in the 540 kN pile toe resistance).

The results of the other full-scale tests in the Norwegian clay also reported by Bjerrum et al. (1969) show similarly large drag loads, ranging from 1200 to 4000 kN, and that the

Fig. 8. Measured and calculated distributions of load at the Heröya site. Data from Bjerrum et al. (1969).

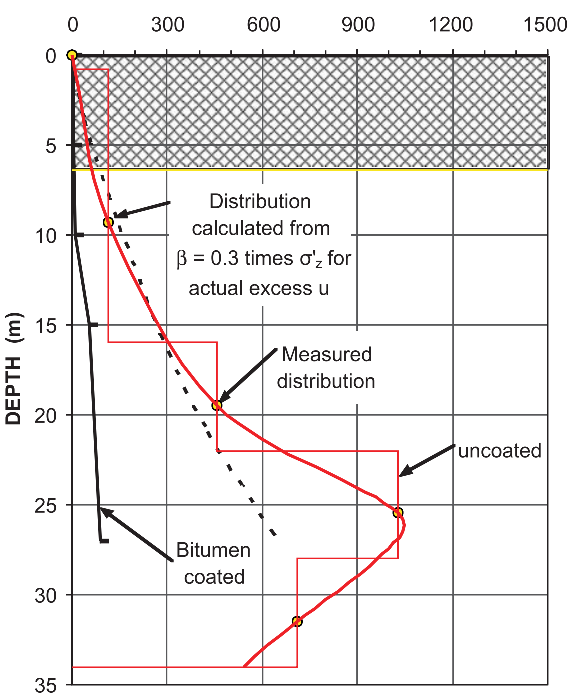
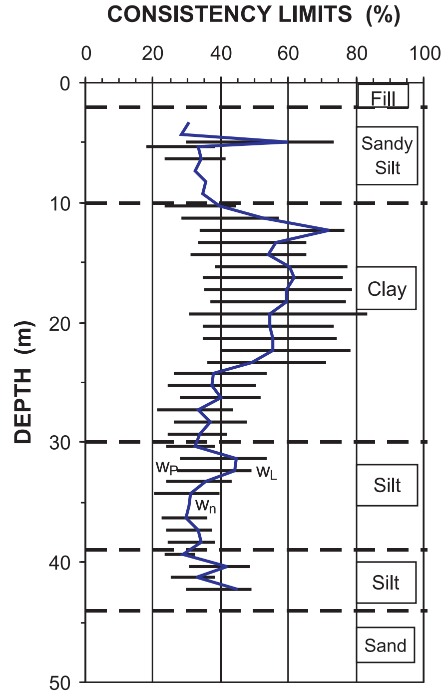


Fig. 9. Soil profile with water content (w_n), plastic limit (w_P), and liquid limit (w_L) at the Fukagawa site. Data from Endo et al. (1969).



negative skin friction correlates well with an effective stress analysis applying a beta coefficient of 0.20–0.30. For five of the test cases, the measured ground settlements ranged from 70 to 2000 mm. For the sixth case, where the test piles were driven at a site where no new fill was placed, only very small settlement occurred, which was caused by reconsolidation after the driving and ongoing secondary compression. Yet, the measured drag load of 3000 kN on the piles driven at this site was as large as that where large settlement was measured.

In summary, the tests in Norway showed that drag loads can be very large, negative skin friction is a function of effective stress, and a force equilibrium will develop somewhere down the pile. It is notable that, although the observations were included in the paper by Bjerrum et al. (1969), somehow the geotechnical practice missed drawing the conclusion that the relative movement between the pile and the soil needed to fully develop the shaft shear is insignificantly small. That is, ostensibly, settling soils are not a necessary requirement for large drag loads to develop on long piles.

Fukagawa, Tokyo, Japan

The proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering in Mexico City included a paper by Endo et al. (1969) reporting on a full-scale test on three strain-gage-instrumented, 43 m embedment, single, 609 mm (24 in.) diameter steel pipe piles with a wall thickness of 9.5 mm (0.375 in.). Below a 2 m thick layer of surface fill, the soil profile consisted of silty sand, 37 m of compressible sandy silt, clay, and silt, 4 m of silt, and loose sand to great depth. The groundwater table was located at a depth of 2 m. Ongoing pumping of water from the sand below 43 m depth had created a steep downward gradient at the site and consolidation of the compressible soil layers. The piles were instrumented with strain gages placed at every 6 m length of the piles, starting at a depth of 8 m, and seven settlement gages and seven piezometers were installed at 6 m intervals in the soil near the piles. Figure 9 shows the soil profile with basic soil parameters, and Fig. 10 shows the pore-pressure distribution at the start of the monitoring in June 1964 and about 2 years later in April 1966 (672 days). The pore pressures underwent only small changes during the monitoring period, reducing a further 10–20 kPa below 10 m depth.

A diesel hammer with a nominal energy of 115 kJ was used to drive the piles into the surface of the sand at 43 m depth, terminating the driving at a very light resistance of 25 mm/blow. One pile was driven open-toe, and two were driven closed-toe. One of the latter two piles was driven at an inclination of 8° to the vertical (1H:7V).

The consolidation settlement caused the soil to hang on the piles. For the vertical, closed-toe pile, Fig. 11 shows the load (converted from the strain-gage values) versus time over the 2 year long monitoring period. Figure 12 shows the load distributions in the same pile at five different times.

As shown in Fig. 13A, although the open-toe pile mobilized about half as much toe resistance as that of the other two piles, the development of drag load in the three piles was about the same. The closed-toe piles (one vertical and one inclined) show about the same load distribution. Near the ground surface and to a depth of about 25 m, the distribution of drag load in the open-toe pile was about equal to that of the closed-toe piles, but the drag loads differed below this depth. Over the about 10 m length above the pile toe, however, the slope of the distribution curve of the open-toe

Fig. 10. Measured distribution of pore pressure at the start of monitoring and 2 years later at the Fukagawa site. Data from Endo et al. (1969).

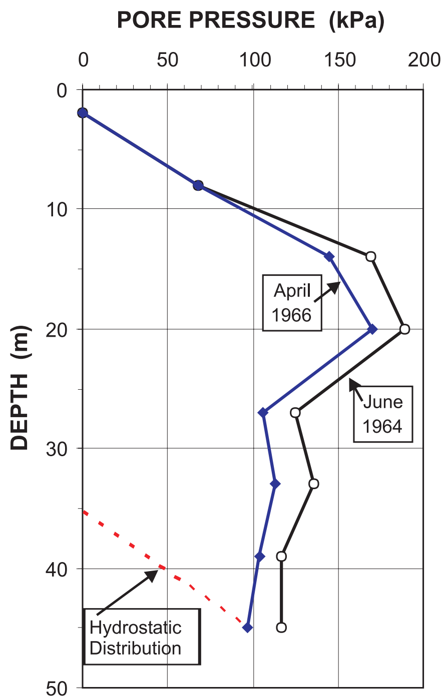
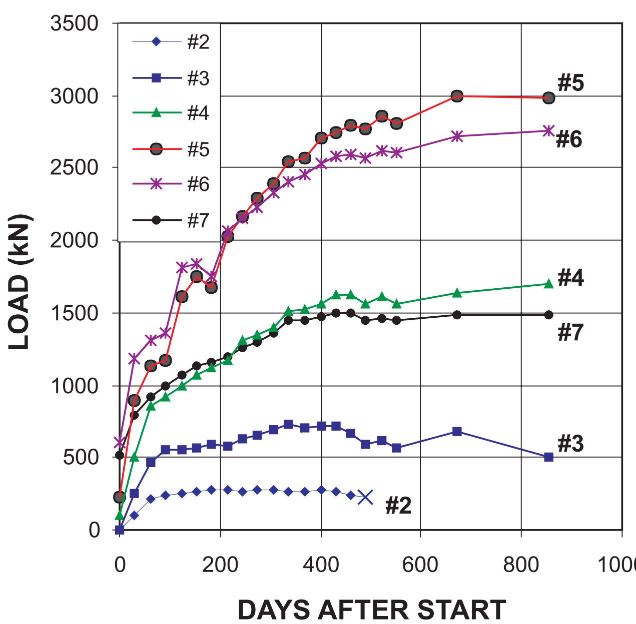
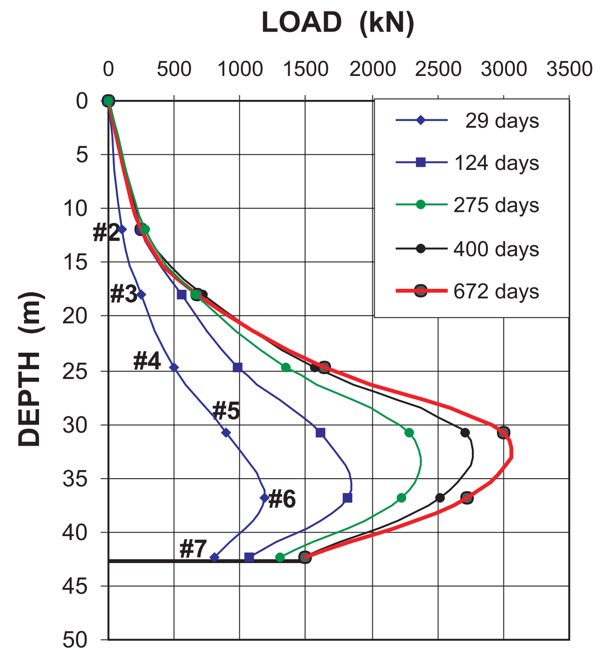


Fig. 11. Load measured at different times and telltale tip depths in the vertical closed-toe test pile at the Fukagawa site. Data from Endo et al. (1969).



pile is similar to that of the closed-toe piles, indicating that the unit shaft shear, here in the positive direction, is about the same for the three piles along this length. In contrast, in the in-between about 10 m long zone, from depths of 25–35 m, where the transition occurs from a negative-direction shear to a positive-direction shear, the open-toe pile exhibits a longer transition zone, starting slightly above the 25 m record, resulting in the appearance of a smaller magnitude shaft shear immediately below 25 m.

Fig. 12. Vertical distribution of load in the vertical closed-toe pile at the Fukagawa site. Data from Endo et al. (1969).



Neutral plane is the term for the location of the force equilibrium in the pile. It is also the term for the location where there is no relative movement between the pile and the soil, i.e., where the pile and the soil settle equally, as demonstrated in Fig. 13B. The paper by Endo et al. (1969) is the first to present observations revealing and confirming this fact. The understanding that the neutral plane is determined by both these aspects is important. Whereas the location of the force equilibrium can be determined by analysis of the resistance distribution or directly by testing an instrumented pile, the location of no relative movement between the soil and pile is much more prone to be in error when determined from settlement analysis. Determining the location of the neutral plane from the location of the force equilibrium is key to the settlement analysis of a pile group, as indicated by Fellenius (1984, 1989, 2004).

The location of the neutral plane is influenced by the pile toe resistance, which is determined by the magnitude of the toe penetration. Endo et al. (1969) provide data on movement of the pile toe and the mobilized pile toe resistance for the start of the monitoring and at three times during the monitoring. Figure 14 is prepared by combining these data and shows the mobilized toe resistance as a function of the net toe penetration. The four data points show a surprisingly linear trend. A more gradual shape similar to that of the broken line in Fig. 14 would perhaps have been expected. The 22 mm penetration after the start of the monitoring (the zero point is unknown) corresponds to only about 4% of the pile diameter, however, and it is not unrealistic for the toe resistance measurements to show a linear trend with increasing penetration at such small penetration.

The author has fitted an effective stress analysis to the load distribution data points of the 2 year measurements of

Fig. 13. Distribution of (A) load in full-length piles and (B) soil and pile settlement 672 days after the start of monitoring at the Fukagawa site. Data from Endo et al. (1969).

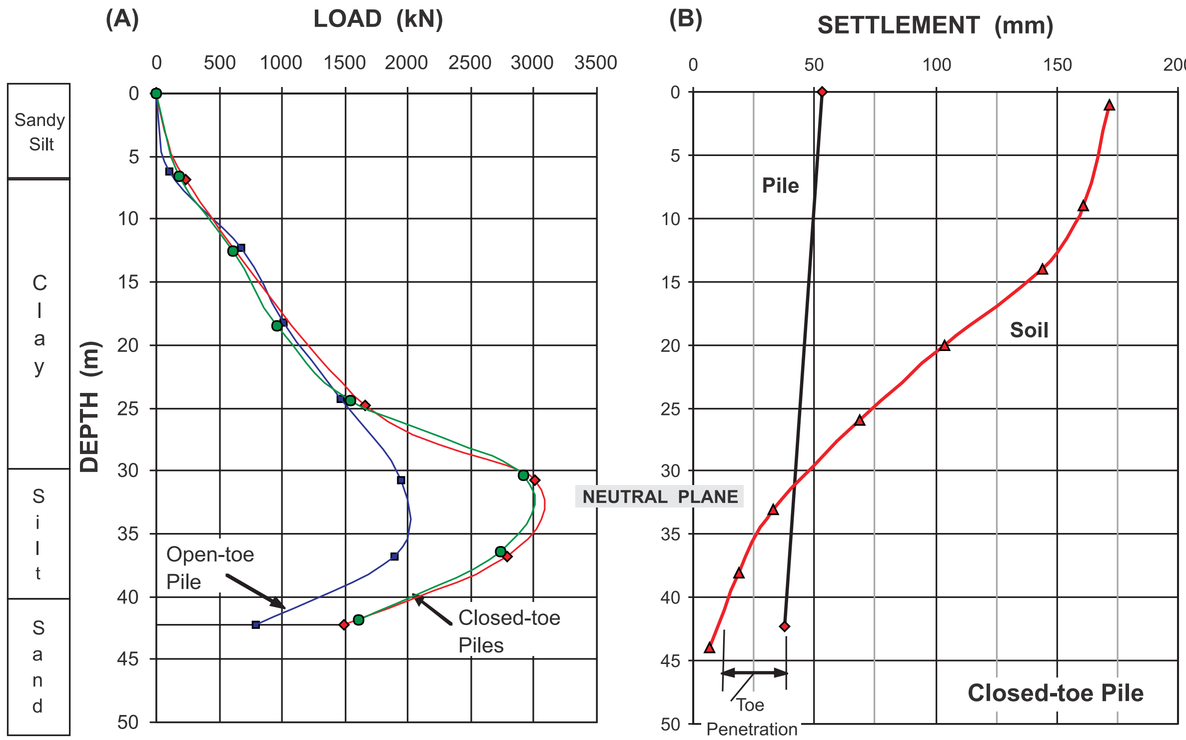
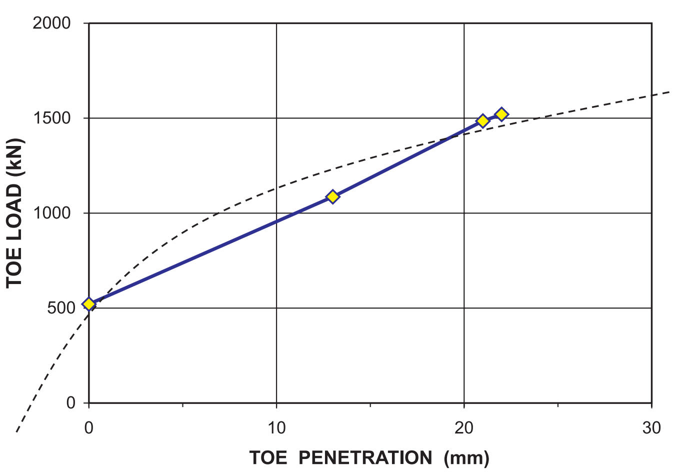


Fig. 14. Fukagawa site. Pile toe penetration of the vertical closed-toe pile. Broken line indicates approximate trend. Data from Endo et al. (1969).



April 1966 (day 672) for the vertical closed-toe pile, applying the then measured pore pressure distribution. The data points, the calculated curve fitted to the data, and the so-determined beta coefficients are presented in Fig. 15. The beta coefficients are about 0.30, which is similar to the values back-calculated for the Sörenga and Heröya cases. The measured toe resistance corresponds to a toe bearing coefficient, N_t, equal to 8, which is a low value and signifies that the pile toe is in a very loose soil or the net toe penetration into the sand is too small to generate a significant toe resistance. The thin broken line in Fig. 15 shows the load distri-

Fig. 15. Load distribution measured and calculated for actual pore pressures and matched to measured loads for the beta coefficients shown and the pore pressure distribution of April 1966 at the Fukagawa site. The thin broken line shows the load distribution when assuming that all shear forces act in the positive direction.

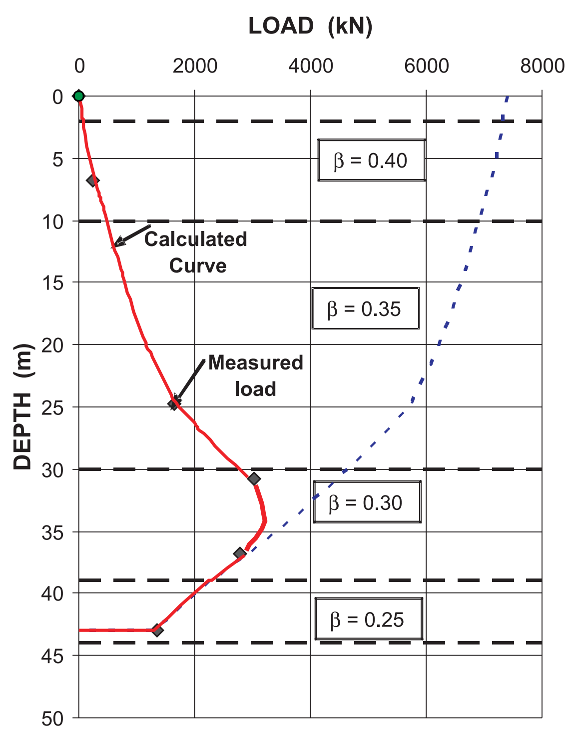
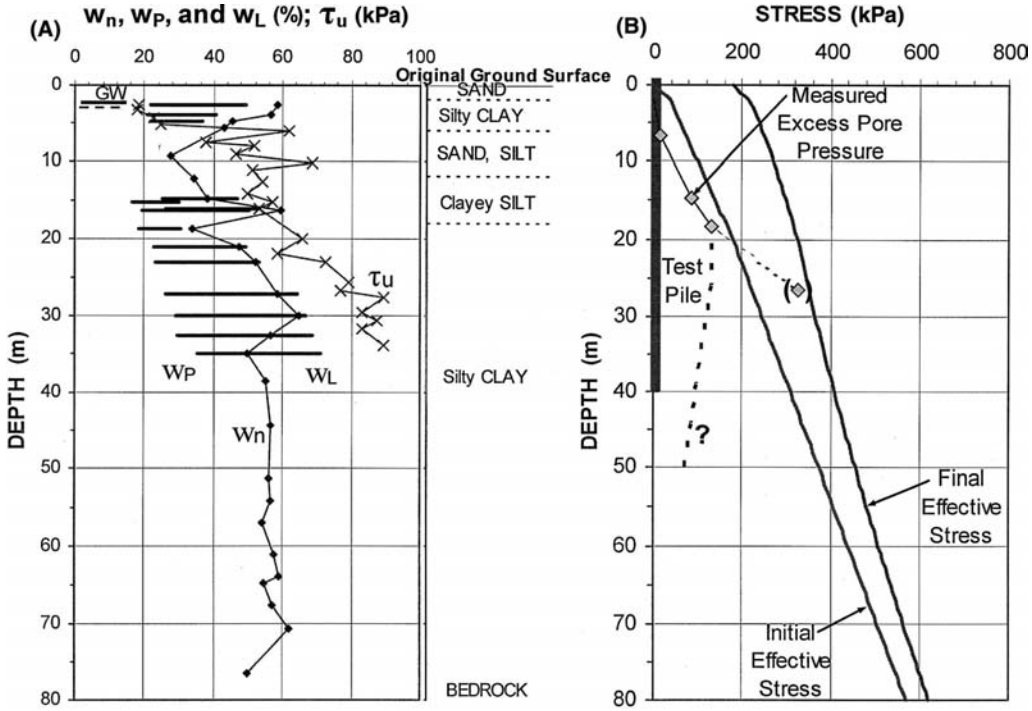


Fig. 16. Berthierville site. (A) Soil profile with water contents, plastic and liquid limits, and vane shear strengths (τ_u). Data from Bozozuk and Labrecque (1969). (B) Distribution of initial and final effective stress and measured excess pore pressures. The question mark indicates an envisaged pore pressure distribution. The measured pore pressure data are from Bozozuk (1972).



bution when assuming that all shear forces act in the positive direction, representing the distribution in a static loading test to the same mobilized toe resistance.

Berthierville, Quebec, Canada

Bozozuk and Labrecque (1969) and Bozozuk (1970, 1972, 1981) described a long-term measurement case history from Berthierville, Quebec, Canada. In October 1964, a highway embankment with a final 11 m height, a 300 m base, and 27 m road width was placed on an 80 m thick deposit of clay. In April 1966, a telltale-instrumented, 324 mm diameter, closed-toe pipe pile was driven through the embankment to a depth of 40 m below the original ground surface. Figure 16A shows the soil profile with basic soil parameters, indicating that the upper about 18 m consists of stratified fine soils deposited on a Champlain clay deposit. Figure 16B shows the calculated distributions of initial and final effective stresses (after full dissipation of excess pore pressures) and the distribution of *excess* pore pressures measured 5 years after the pile driving, i.e., April 1971. The excess pore pressure measured in the lowest piezometer, if believed, would indicate that the placing of the embankment induced an excess pore pressure that would be greater than the embankment stress and remain so even 5 years after the placing of the fill. The author believes that a more plausible 5 year pore pressure distribution would be more like the broken line shown continuing to 50 m depth, which is drawn assuming the excess pore pressure below 18 m depth would have the same relative portion of the imposed embankment stress as that at 18 m depth.

The embankment settled significantly. At the time of the driving of the test pile, 550 days after the start of the placing of the embankment, the ground surface had settled about 1.7 m. The vertical alignment of the road bed where it connected to the bridge was continually corrected by adding pavement asphalt. Figure 17 shows that during 5 years of observations after the driving of the test pile, the original ground surface next to the test pile settled an additional 540 mm. Settlement gages placed at depths of 6, 12, 30, and 44 m showed that most of the settlement occurred above 30 m depth. Figure 18A shows the vertical distribution of settlement and the downdrag (i.e., settlement) of the test pile. The pile head settled 470 mm, 70 mm less than the ground surface. The drag load distribution in the pile is shown in Fig. 18B. The two figures show the same location of the point of equal movement (Fig. 18A) and the point of

Fig. 17. Settlement measured at gages placed at the ground surface (S-5) and at depths of 6 m (S-4), 12 m (S-3), 30 m (S-1), and 44 m (S-10) during the first 5 years of monitoring after the driving (April 1966 to April 1971) at the Berthierville site. Data from Bozozuk (1972).

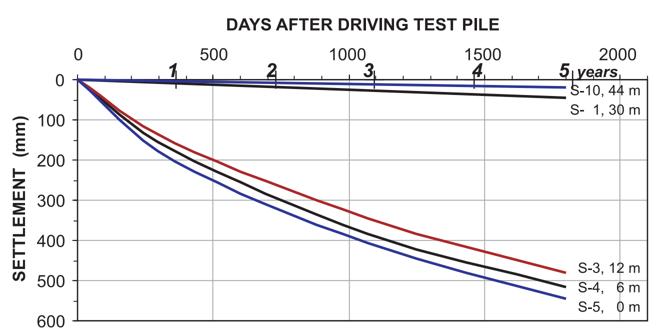
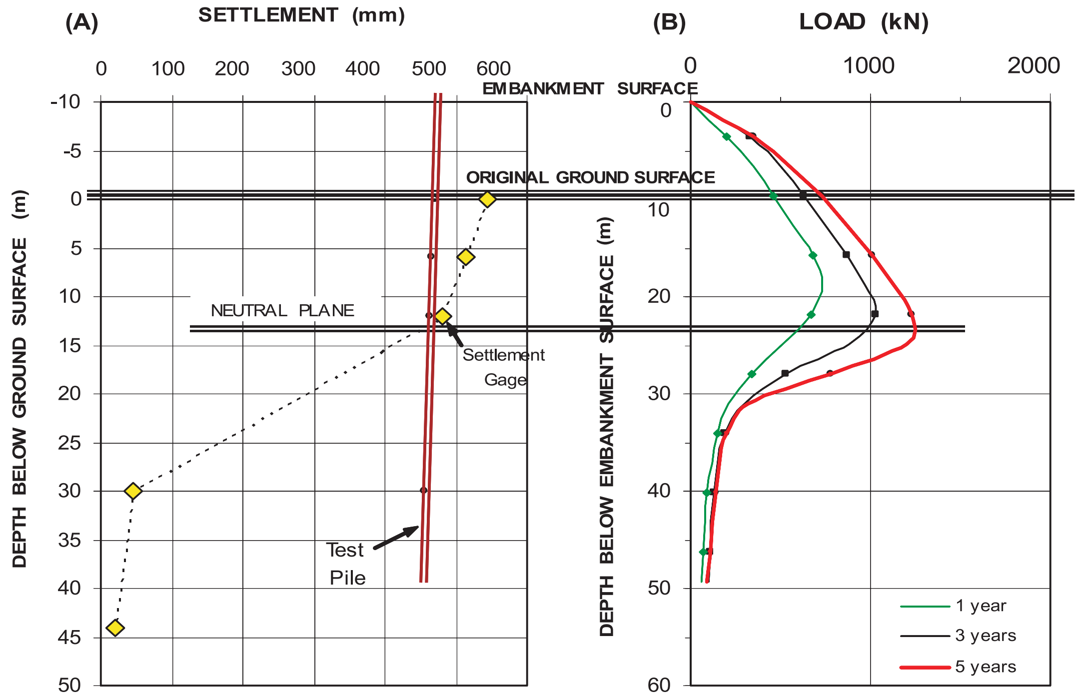


Fig. 18. Berthierville site: (A) distribution of settlement of the soil and test pile with depth measured over 5 years after the driving of the test pile; (B) distribution of load in the test pile at 1 year, 3 years, and 5 years after the driving of the test pile. Data from Bozozuk (1972).

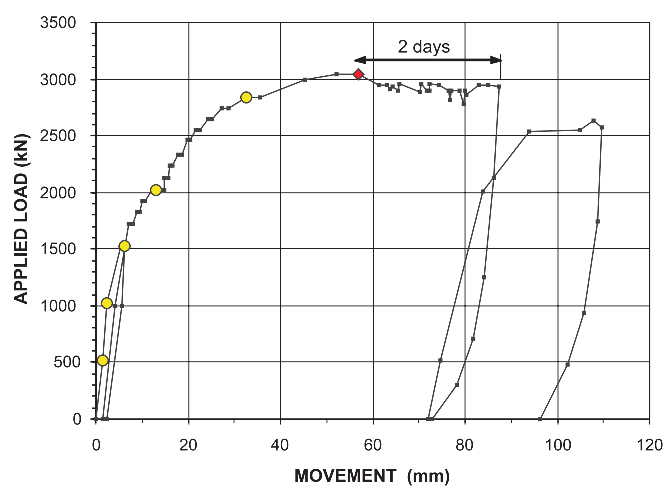


equal negative- and positive-direction forces (Fig. 18B), i.e., the neutral plane, at a depth of about 12 m to 14 m.

The distribution of load in the pile was calculated from pile shortenings measured by means of eight telltales. Figure 18B indicates that the slope of the load distribution curve below the neutral plane over a length of about 10 m is about half of that above the neutral plane and that, in contrast, the distribution curve is almost vertical over the following about 20 m length. The slopes are indicative of shaft resistance. If the two slopes are true, they indicate that the positive shaft resistance between the neutral plane and the next 10 m below would be more than about twice as large as the negative skin friction immediately above the neutral plane. Moreover, they indicate that a next to zero positive shaft resistance would have been mobilized along the lowest 20 m length of the pile. The soil above the neutral plane is not that different from the soil below, however, and the soil below the neutral plane is quite homogeneous throughout the length of the pile and beyond. Therefore, the positive mobilized shaft resistance cannot differ much along this length of the pile. Were the values true, the effective stress within the 10 m length below the neutral plane must be large, while at the same time being very small over the 20 m length farther below. The author believes the three lower telltales are affected by friction in their guide pipes, which has resulted in inaccurate load data below the neutral plane.

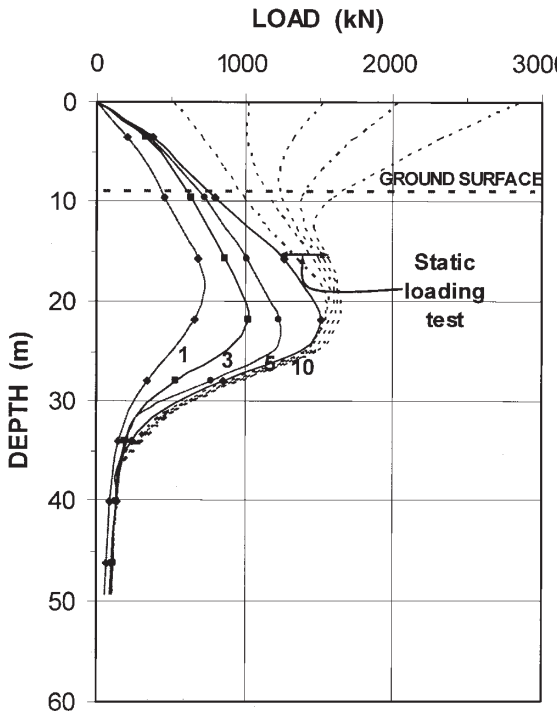
Bozozuk (1981) reported the results of a static loading test carried out 10 years after the pile driving. Figure 19 presents the measured load–movement curve. At the applied loads of 510, 1020, 1520, 2030, and 2820 kN, the load levels were maintained for 1 day before the test was continued. No significant movement developed during these load-holding occasions. At the applied load of 3060 kN, movements became progressively larger as the ultimate resistance of the pile was reached. During the next 2 days, the load that the soil could sustain gradually reduced. In the reloading cycle after unloading, the previous maximum load could not be reached again.

Fig. 19. Load–movement curves from static loading test 10 years after the pile driving at the Berthierville site. Data from Bozozuk (1981).



The vertical distribution of load in the pile evaluated at the five levels of sustained load is shown in Fig. 20, which also shows the 10 year distribution determined immediately before the static loading test and, for comparison, the distributions at 1 year, 3 years, and 5 years. The latter curves indicate how the increasing values of applied load progressively made the shaft shear change from the negative to the

Fig. 20. Load distributions at passive monitoring of the test pile at 1 year, 3 years, 5 years, and 10 years after the pile driving and for five sustained loads from the static test at the Berthierville site. Data from Bozozuk (1972, 1981).

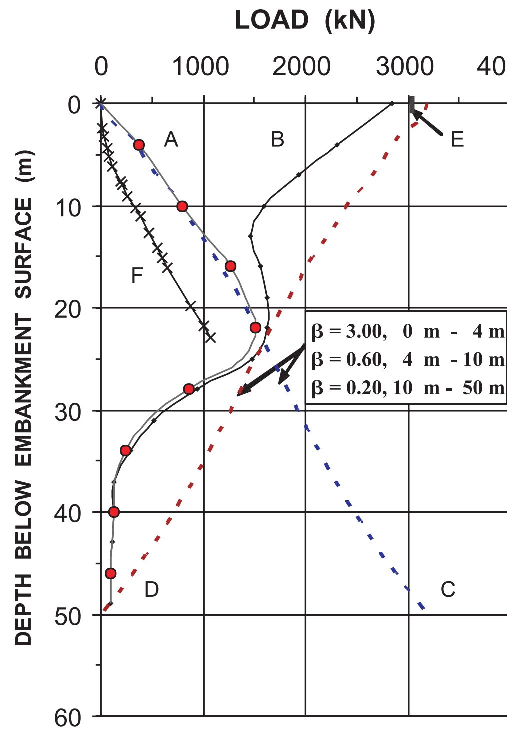


positive direction and that the applied load did not add to the existing maximum drag load in the neutral plane, demonstrating that live load does not add to the drag load in a pile but replaces it.

No measurements of load distribution could be obtained for the 3060 kN plunging load. The drag load of about 1500 kN was supported by an equal-magnitude shaft resistance present in the portion of the pile below the neutral plane, the point of force equilibrium. This means that half of the approximately 3000 kN applied load served to eliminate the drag load and the remaining half served to keep mobilizing the positive resistance below the neutral plane. In other words, the test results proved that the negative skin friction and positive shaft resistance are fully mobilized shear forces. Moreover, the results show that the forces slowly built up over a 10 year period and the forces mobilized in a relatively rapidly performed loading test were very similar in magnitude.

Figure 21 shows a compilation of the data and analysis results. Curves A and B show the load distributions at the 10 year measurements (before the static static loading test) and at the maximum sustained load in the static loading test (2820 kN), respectively. Curves C and D show load distributions calculated using effective stress in fitting the calculated distribution to the 10 year measured load curve. Curves C and D indicate the distribution of shaft shear acting in the negative and positive directions, respectively, along the full length of the pile. The intersection of curves C and D is the location of the neutral plane. Point E indicates the ultimate resistance (plunging load) in the static loading test. Lastly, curve F shows the load distribution calculated from the as-

Fig. 21. Berthierville site: (*i*) load distribution at the 10 year measurements (before the static test) (curve A), (*ii*) load distribution at the maximum sustained load in the test of 2820 kN (curve B), (*iii*) load distribution calculated using effective stress matched to the drag loads measured above the neutral plane (curve C), (*iv*) load distribution calculated using the same parameters as in those for curve C (curve D), (*v*) ultimate resistance in the static loading test (curve E), and (*vi*) load distribution calculated from the vane shear strength (curve F). Test data for curves A, B, and E from Bozozuk (1981).



sumption that the shaft shear was equal to the undrained shear strength determined in the vane shear test shown in Fig. 16A (after adjustment according to Bjerrum 1972). There is little or no relation between the evaluated shaft shear (negative skin friction) and the undrained in situ shear strength values. The vane tests were performed before the placing of the fill, however. New vane soundings after the more than 10 years of consolidation would have shown larger strength values. It is outside the scope of this text to discuss whether or not the new vane soundings would have produced a load distribution more in agreement with the measured load distribution, curve C.

The evaluated load distribution in the embankment fill corresponds to beta coefficients of 3.00 in the upper 4 m and 0.6 in the lower about 6 m. A beta coefficient of 3.00 is a very high value. As Bozozuk (1972) points out, the values reflect a combination of the large horizontal stress created by the compaction of the engineered fill and the effect of the embankment being settled more in the centre than toward the sides. The latter caused the embankment to bow, which increased the horizontal stress (the earth pressure) in the upper portion of the embankment. The beta coefficient of 0.20 in the clay is low but within the range of values found in normally consolidated marine clays. Because of approximations associated with the use of telltale instrumentation and necessary approximations in the author's evaluation, however, such as the assumption made with regard to the distribution of pore pressures over the lower half of the pile, the indicated beta coefficients are only approximate values. A repeat of the test using modern methods of instrumentation would probably have resulted in coefficients slightly different from those shown.

It should not be taken as rejection of the test results that the data below the neutral plane and the lowest piezometer value appear to be suspect. The case history confirms the results of the tests in Norway and Japan with regard to the following: that the load transfer is governed by effective stress; the shaft shear in the negative direction is equal to that in the positive direction; the shaft resistance acting over the long term is equal to that mobilized in a short-term static loading test; and the beta coefficient is a function not just of the overburden effective stress, but also of the horizontal stress in the soil. These are first-time findings, and the papers are enlightening major contributions to the advancement of the state-of-the-art.

Melbourne, Australia

Walker et al. (1973) installed a 760 mm diameter, closed-toe, strain-gage-instrumented, pipe pile to an embedment depth of 27 m by driving the piles through 6 m of silty sand deposited on 15.5 m of stiff silty clay, followed by 3 m of sandy silt on dense sand and gravel. The pile reached 2.5 m into the sand and gravel. An identical pile was coated with 1.5 mm of bitumen through its entire length and driven to an embedment depth of 24.5 m. The silty sand layer was compact, with SPT *N*-indices smaller than 20 blows per 0.3 m. The water content of the stiff silty clay layer ranged from 60% to 100%, and the liquid and plastic limits ranged from 90% to 120% and 30% to 50%, respectively. After completion of the pile driving and site instrumentation (piezometers and settlement gages), a 3.0 m thick layer of fill was placed over an area of 200 m × 100 m around the piles. The silty clay was overconsolidated and had a preconsolidation margin of about 80 kPa, much larger than the stress imposed by the 3 m of fill.

The surcharge resulted in an increase in pore pressures of about 20 kPa that did not change during the reported monitoring period. Settlement measured at the ground surface was small. Over a monitoring period of 238 days after the placing of the surcharge, only about 25 mm of settlement was recorded. Walker et al. (1973) indicated that the measured settlement was due to creep rather than consolidation. The strain gages in the test pile registered an increase in load. Figure 22 shows the loads evaluated from the strain gages at the 20 m embedment depth in the uncoated pile. The 238 day load distribution in the piles is shown in Fig. 23, and the bitumen all but eliminated the shear forces along the pile. Over the 238 days, the loads in the uncoated pile had built up to a maximum of about 1800 kN in the neutral plane (the 20 m gage). Because of the small relative movements between the pile and the soil, the transition zone is long and covers essentially the full thickness of silty clay layer. No pile toe load was mobilized, which means that the settlement at the neutral plane was so small that no toe penetration was created. The broken lines in Fig. 23 indicate the load distribution calculated in an effective stress analysis matched to the data. The beta coefficient fitted to the load from the fill layer is large, about 2.00. In the fine sand, the fitted beta coefficient decreases from 1.00 at the upper boundary to 0.80 at the lower boundary, still large for a silty sand. In the silty clay and sandy silt, the value of beta is 0.40 and 0.45, respectively. In the sand and gravel, calculations using a beta coefficient of 0.60 produce a load distribution that matches the measured load distribution.

Fig. 22. Settlement of ground surface and development in load at 20 m depth in the pile at the Melbourne site. Data from Walker et al. (1973).

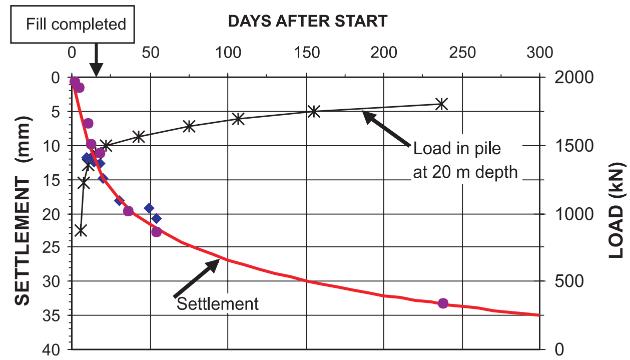
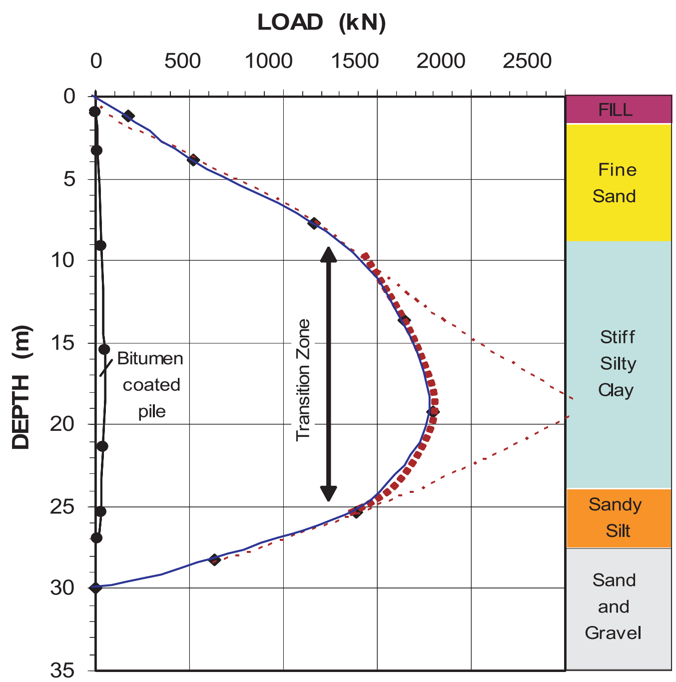


Fig. 23. Measured load distribution in uncoated and coated piles and calculated distribution at the Melbourne site. The broken line shows calculated distribution and the dashed line shows distribution extrapolated from the measured values. Data from Walker et al. (1973).



Keehi Interchange, Hawaii, United States

Clemente (1981) reports a full-scale test involving three instrumented, 419 mm diameter, octagonal, precast prestressed concrete piles on the island of Oahu, Hawaii. As in-

Fig. 24. Plan view and profile of fill, soil, and piles at the Keehi Interchange site. Data from Clemente (1981).

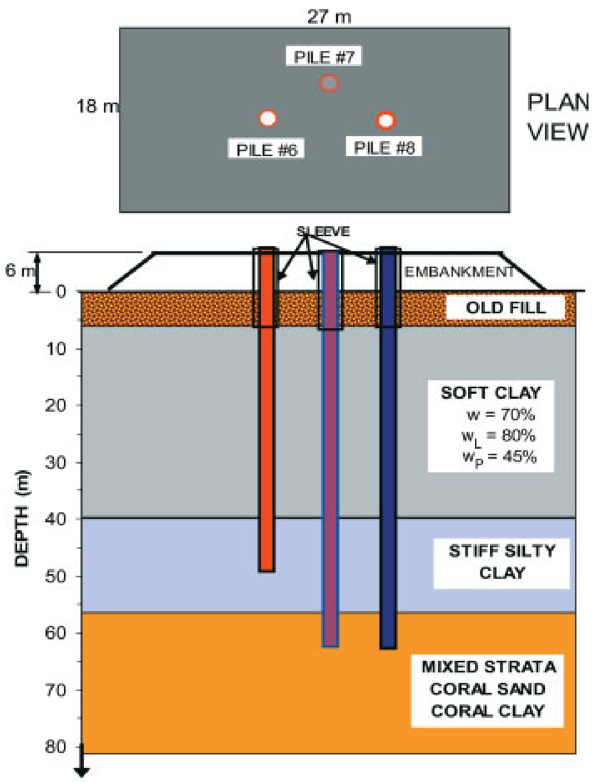
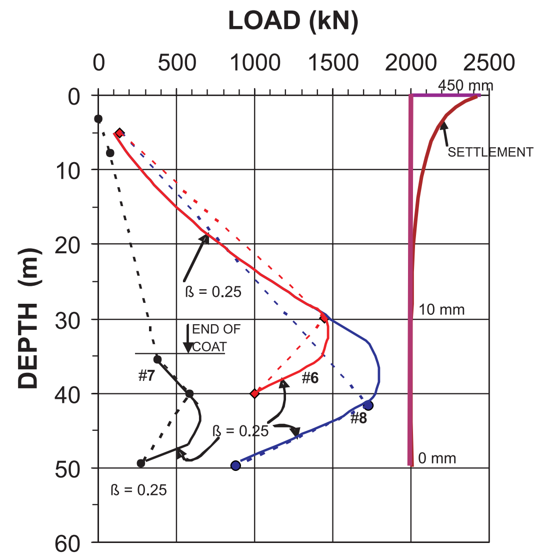


Fig. 25. Load and settlement distributions after 180 days at the Keehi Interchange site. Broken lines connect measured values. Data from Clemente (1981).



dicated in Fig. 24, the soils at the site consisted of a 4 m thick layer of sand fill placed a long time earlier over a 36 m thick deposit of soft clay followed by 16 m of stiff silty clay. At a depth of 56 m, the soil consisted of a mixture of medium dense to dense coral sand alternating with firm to stiff silty coral clay. The groundwater table was located at the interface between the fill and the soft clay. In July 1977, three test piles, numbered 6, 7, and 8, were driven and instrumented with multirod extensometers measuring pile shortening. Pile 6 was driven to 40.0 m depth, and piles 7 and 8 were driven to 49.7 m depth. Before the driving, pile 7 was painted with a thin asphalt primer and then coated with A85 bitumen over the upper 35 m length, leaving the rest of the pile (15 m) uncoated. Piles 6 and 8 were not coated. The coat was to be 1.5 mm thick, but much of the bitumen flowed off the upper surface and sides of the pile because of the warm temperatures. After the coating, the pile was driven to an embedment depth of 50 m. Piles 6 and 8 were driven to 40 and 50 m depths, respectively. In August 1977, a 6 m high embankment with a crest area of 18 m × 27 m was placed around the piles and soil settlements and pile shortenings were monitored for 180 days. The shortenings were converted to strains and loads in the piles.

Figure 25 shows the load distributions in the clay below the fill and the settlement distribution after 180 days. The symbols joined by the broken line show the evaluated loads, and the solid lines show the load distributions calculated by the effective stress method. The author derived a beta coefficient of 0.25 by matching the calculated loads to the evaluated loads for all three piles. The paper does not report any piezometer observations, and the author has assumed that no excess pore pressures remained in the clay at the end of the observation period. The beta value 0.25 is a lower bound value because any remaining pore pressures, if known and considered in the calculations, would result in higher coefficients. All three piles developed a force-equilibrium neutral plane. Because of the different conditions of lengths and coating, the neutral planes are at different depths, but the evaluated effective stress parameters are the same for all three piles.

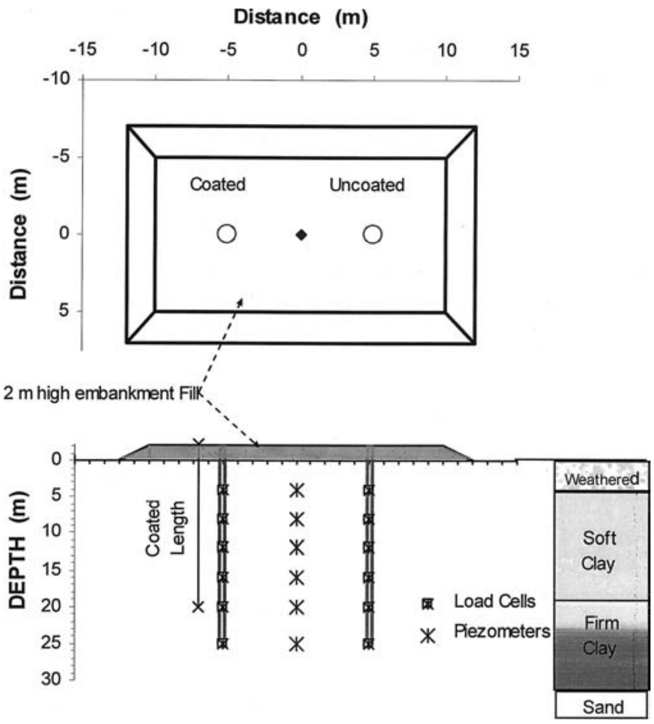
The embankment fill caused the ground to settle. At the ground surface, the settlement was 450 mm. Most of the settlement developed in the upper 10 m. The measurements show that farther down in the soil, the relative movement between the pile surface and the soil was no more than a few millimetres, and yet both negative skin friction and positive shaft resistance appear to have been fully mobilized. The drag load along the bitumen-coated length of pile 7 was 375 kN, whereas that along pile 8 to the same length was 1770 kN. This means that the thinner than about 1 mm thick coat of bitumen reduced the negative skin friction to 20% of that for the uncoated pile.

Bangkok, Thailand

Indraratna et al. (1992) presented a full-scale study of two 400 mm diameter, instrumented precast concrete piles driven into soft Bangkok clay. One of the two piles was coated with a layer of bitumen along its upper 20 m length. The application of bitumen was at 200 cm^3/m^2, theoretically producing a 0.2 mm thick coat. The soil profile consisted of an upper 2 m thick layer of weathered clay followed by 13 m of soft clay with a water content of about 80%, deposited on 14 m of firm clay with a water content of about 60% above sand at 29 m depth.

The piles were driven in five stages, with the driving interrupted at 8, 12, 16, and 20 m depths for a period of 9 days and an uplift, quick method, static loading test performed before the driving was resumed. The final pile depth was 25 m. After the driving, a 2 m high embankment was placed

Fig. 26. Plan view and section of the Bangkok site.



around the piles over an area of 14 m × 24 m, as indicated in Fig. 26. Pile loads and settlement were recorded over a period of 265 days after the placing of the embankment. Figure 27 shows load versus movement for the pull tests. The figure for the uncoated pile is supplemented with the plot of the distribution of the ultimate load for each pull test versus depth. Despite the quick static loading test and the very thin bitumen coat, the bitumen reduced the maximum shaft resistance to 50% of the shaft resistance of the uncoated pile.

Figure 28A shows the distribution of load measured 3, 25, 92, 156, and 265 days after the placing of the embankment and includes the ultimate resistances from the four uplift static loading tests. The drag load distribution measured at 265 days and the uplift resistance distribution are quite similar to one another. Figure 28A is supplemented with a curve showing the fitted load distribution for a beta coefficient of 0.25 above the neutral plane and 0.30 below. The measured toe resistance corresponds to a toe bearing coefficient of 3, which is small even in marine clay. The paper reports vane shear undrained shear strength values to a depth of 16 m. A distribution calculated using the vane shear strength values as unit shaft resistance results in a distribution of load that is close to that determined in the static uplifts tests and the drag-load distribution to 16 m depth (the distribution is not shown in Fig. 26A). Despite the thin bitumen coat, the maximum drag load for the coated pile was reduced to 40% of the maximum drag load in the uncoated pile.

Figure 28B shows the measured distribution of settlement and the settlement of the pile, with an enlargement superimposed of the settlement between 10 and 25 m depth. The settlement of the pile, the downdrag, is small because it is governed by the soil settlement at the location of the neutral plane, which is less than 10 mm. The relative movement between the pile and the soil that mobilized the positive shaft resistance below the neutral plane, the last approximately 7 m length of the pile, was about 2 mm and the toe penetration was about 2 mm, which is commensurate with the very small measured toe resistance.

Singapore Port

Leung et al. (1991) presented measurements in a strain-gage-instrumented, 280 mm square, precast concrete pile driven to 24 m embedment through 9.5 m of silt and clay fill, 8.5 m of marine clay, and 3 m of sandy clay on shale bedrock. The piles were installed to support a container deck, and the allowable load on the piles was 800 kN. Figure 29 shows the measured load distribution once the deck was built (57 days after the driving) and 2 years later (745 days). Leung et al. do not report the amount of soil settlement (due probably to the fill) or settlement of the piled foundation.

The container loads constituted a variable or live load on the piles, which amounted to only about 200 kN/pile during the 2 year measurement period. The container loads affected the load distribution in the upper 9 m of the pile. Below this depth, the load distribution was governed by the soil settlement, which was large enough to generate negative skin friction along the pile. A neutral plane developed at the lower boundary of the marine clay. The measured load distribution corresponds to a beta coefficient of 0.50 in the marine clay. Note the similarity between the distribution of load above the neutral plane and that shown in Fig. 20 for the second and third load levels of the Berthierville test.

Bäckebol, Göteborg, Sweden

Background information

In June 1968, a full-scale test was started in Bäckebol, Sweden, consisting of driving two single, 55 m embedment, 300 mm diameter, hexagonal cross section, load-cell-instrumented, spliced, precast concrete piles. The observations continued until August 1983, i.e., for 5500 days. The objective was to measure the forces developing in a pile due to reconsolidation of the soil after the pile driving, and as caused by soil settlement due to the placing of a fill around the piles.

The Bäckebol site is in an open area along the Göta River outside the city of Göteborg, southwestern Sweden. The area is virgin, untouched by construction since it rose from the sea after the end of the Ice Age. As shown in Fig. 30A, the main soil body consists of a 40 m thick layer of sensitive, marine postglacial clay, followed by silty sand to a great depth. To a depth of about 17 m, the natural water content is about 90% and is greater than the liquid limit. Below 17 m the water content decreases with depth but remains close to the liquid limit. The percentage of clay-size particles is about 80% to a depth of 20 m and about 55% from 20 m to 30 m. Details about the site are available in Fellenius and Broms (1969) and Fellenius (1972).

As indicated by the vane shear strength distribution shown in Fig. 30B, the clay is soft to a depth of about 10 m and firm below 10 m. Eleven consolidometer tests performed on samples from depths of 4.0–32.5 m show that the preconsolidation margin of the clay is about 20 kPa in the upper portion of the clay and increases to about 30 kPa in

Fig. 27. Pull test results at the Bangkok site for (A) the uncoated pile and (B) the bitumen-coated pile. Data from Indraratna et al. (1992). Inset graph shows load (kN) versus depth (m).

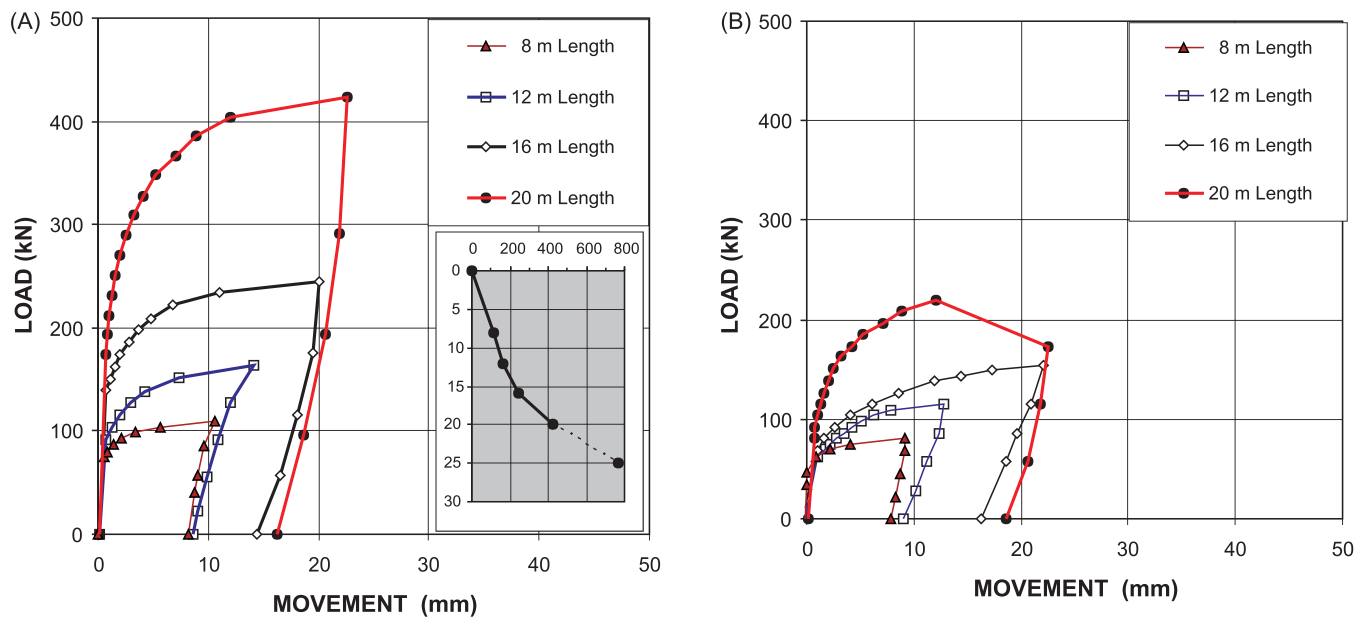
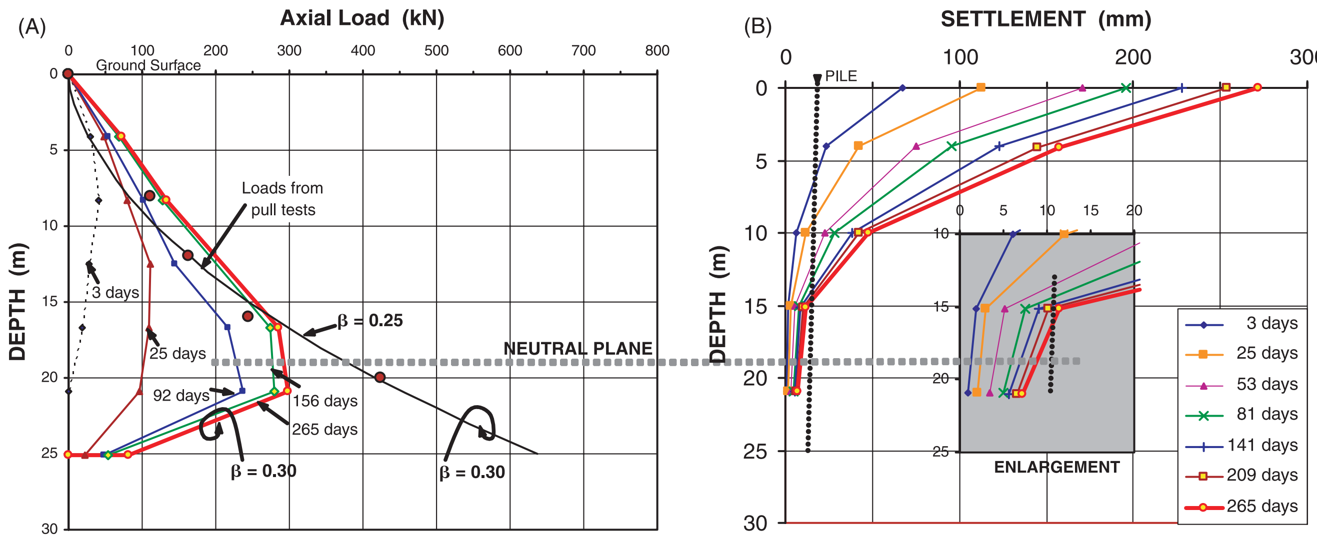


Fig. 28. Bangkok site: (A) load distributions, (B) settlement distribution. Data from Indraratna et al. (1992).

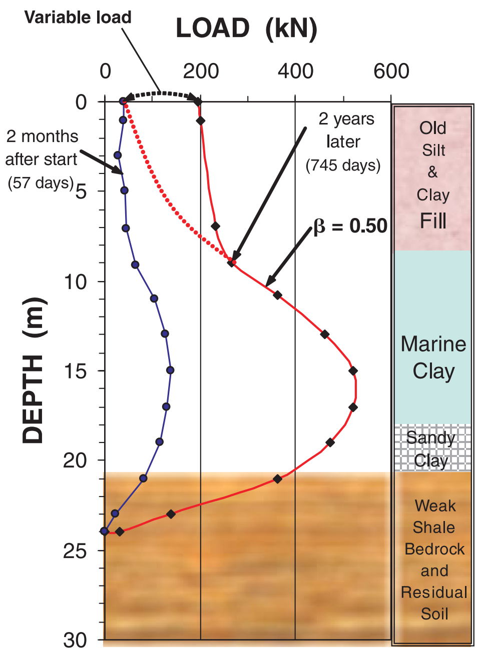


the lower portion. Figure 30C presents the compressibility of the clay expressed in Janbu modulus numbers (Canadian Geotechnical Society 1992; Janbu 1963, 1998) obtained from the tests. An example of the stress–strain curves representative of the consolidation tests is presented in Fig. 31. The virgin Janbu modulus number, m, ranged from about 4 to 7, with an average of about 5, which indicates a very compressible clay. The reloading modulus number, m_r, ranged from 60 to 80, with an average of 75. The void ratio, e_0, for the tested samples ranged from 1.3 to 2.4, the virgin consolidation coefficient, C_c, ranged from 0.06 to 0.16, and the reloading consolidation coefficient, C_r, ranged from 0.07 to 0.11. The depth to the groundwater table varies slightly seasonally from close to the ground surface to a depth of about 1.0 m. The pore-water pressure is hydrostatically distributed.

Before the pile driving, three pneumatic piezometers (Kallstenius and Wallgren 1956) were installed at depths of 9, 23, and 30 m at a distance of about 0.7 m outside the intended location of each pile. In addition, one axially flexible hose multipoint settlement gage was placed 0.1 m outside each pile location and one was placed 5 m away from the piles, enabling measurements of vertical movement at every 2 m to depths to about 33 m. Instrumentation details are described by Fellenius (1972). The distances to the points in the gages were measured with a tape inside the hose and related to elevation by means of surveyor's leveling of the uppermost point. The accuracy is about 1 mm for a single tape measurement and at best 2 mm in a comparison of two or more measurements.

A load cell was specially developed for the two test piles and was designed to measure the load in the pile directly,

Fig. 29. Load distribution at the Singapore Port site. Data from Leung et al. (1991).



eliminating the need for using the pile modulus to determine the load in the pile. Details about the load cell are available in Fellenius and Haagen (1969). The piles were made up of five 11.0 m long segments spliced in the field as each pile was driven. The load cells were manufactured as 0.6 m long pile segments and placed between the first and second, second and third, and third and fourth pile segments in both test piles. Pile PII was also supplied with a load cell at the pile toe. Pile PI was driven to a depth of 53.1 m, and pile PII to a depth of 55.0 m. After the driving, a toe telltale was inserted into each pile to measure pile shortening. Details about the piles are available in Fellenius (1972).

The testing activities consisted of the following steps: (1) taking the last "zero" (initial) readings of site gages on 24 June 1968, before driving the piles, and recording all gages at frequent intervals following the driving until 2 November 1969 (days 0–496); (2) forming and casting a concrete slab on the piles to provide a load of 440 kN (started on day 495, with the formwork removed on day 517); (3) taking the readings at all gages at frequent intervals until 31 October 1970 (days 518–859); (4) placing a 360 kN load of concrete blocks on the concrete slab to increase the pile load to 800 kN (day 860); (5) taking the readings at all gages at frequent intervals until 29 September 1973 (days 860–1923); (6) placing a 2 m high layer of fill in a circle with 42 m diameter around the piles (days 1924–1988); (7) taking the readings at all gages until the end of the test on 27 September 1982 (days 1988–5206); and (8) completing the last survey of the multipoint settlement gage on 8 August 1983 (day 5523).

Results

The driving of the two piles created large excess pore pressures in the clay. Figure 32 shows pore pressures measured at a depth of 23 m adjacent to the piles. At a depth of 23 m, the vertical effective overburden stress before the pile driving was about 120 kPa. Obviously, for the first few days after the pile driving, the piles are standing in a highly remolded soil with zero effective stress, essentially a liquid. The pore pressures required about 5 months to return to the pre-driving levels.

Fellenius and Broms (1969) present observations of soil movements during and immediately following the driving. The driving of the piles into the sand caused the sand to compact. Precision leveling immediately after the end of the driving (EOD) showed that the sand surface settled 9 mm at pile I and 7 mm at pile II during the driving. The simultaneous measurements showed that the clay layer expanded due to the driving of the displacement piles. As shown in Fig. 33, the upper 5 m portion of the ground next to the pile heaved about 20 mm relative to the bottom of the clay layer, resulting in a net heave of the ground surface. Below 2.5 m depth, the upward movement of the clay was less than the settlement of the boundary between the clay and the sand. Thus, except for the uppermost about 2.5 m, the net effect during the driving of the two single piles is a downward movement of the soil in relation to the piles. During the first few days following the driving, the concrete piles lengthened (swelled) about 1 mm from absorbing water from the soil. During the first 150 days following the driving, the clay compressed (settled) due to the reconsolidation.

The survey method of measuring settlement at the site was not accurate enough to record the settlement until the measurements had continued for a few years. As the monitoring proceeded, however, it was realized that a small continued settlement was occurring in the area that was equal in magnitude to the isostatic uplift of the land surface, i.e., about 1 mm per year. The settlement of the ground surface away from the piles was measured to be about 10–15 mm over the first 500 days. Next to the piles it was somewhat less (Fellenius 1971, 1972).

The load-cell measurements showed no change during the very first days when the measured pore pressures exceeded the calculated effective overburden stress. That is, the small heave and lengthening of the pile due to absorption of water did not coincide with any change of load in the pile. Thereafter, the loads measured in the piles started to increase, coinciding with the dissipation of pore pressures. The forces are built up from an accumulation of negative skin friction due to the relative movement between the pile and the clay. Figure 34 shows the development of pile forces with time after the pile driving.

The shortening of the piles due to the measured forces during the first 500 days before any load was placed on the pile head amounted to about 5 mm, and the pile toe movement (penetration) into the silty sand was about the same magnitude. Load cell M4 shows that the pile toe force increased from 80 kN to 200 kN.

At the end of driving (EOD), the forces in the pile were about equal to the buoyant weight of the pile. As the drag load from the clay increased, positive resistance was mobilized in the lower portion of the pile. The development is best seen in Fig. 35, which shows the vertical distributions of force in the piles. When placing the first 440 kN on the pile head (test activity step 2), the drag load was essentially eliminated. The small soil movements continued, and there-

Fig. 30. Bäckebol site: (A) distribution of Atterberg limits and water content; (B) distribution of vane shear strength, effective stress, and preconsolidation stress; (C) distribution of modulus numbers.

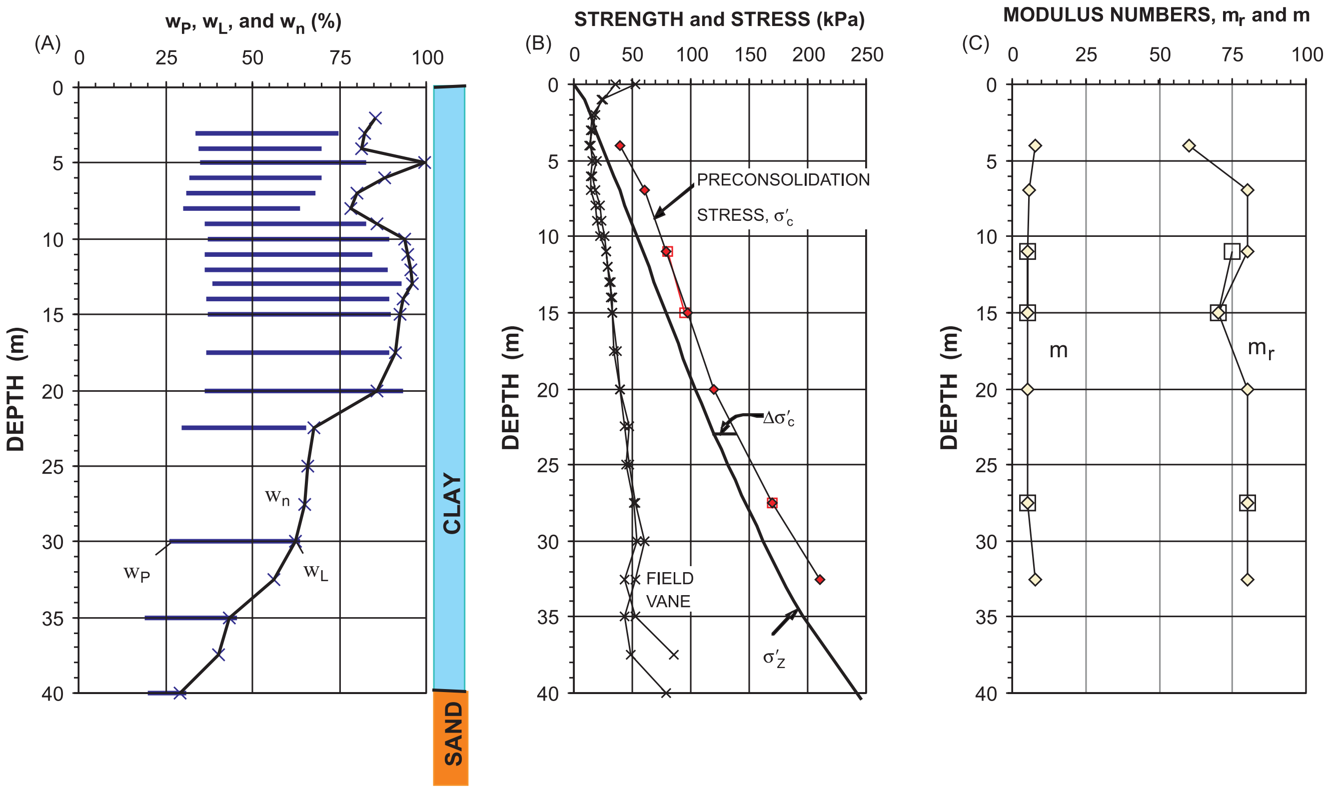


Fig. 31. Example of stress–strain curve from consolidometer test.

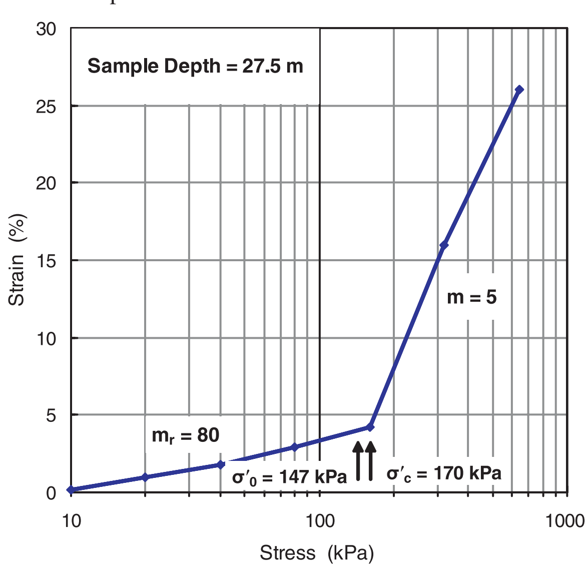
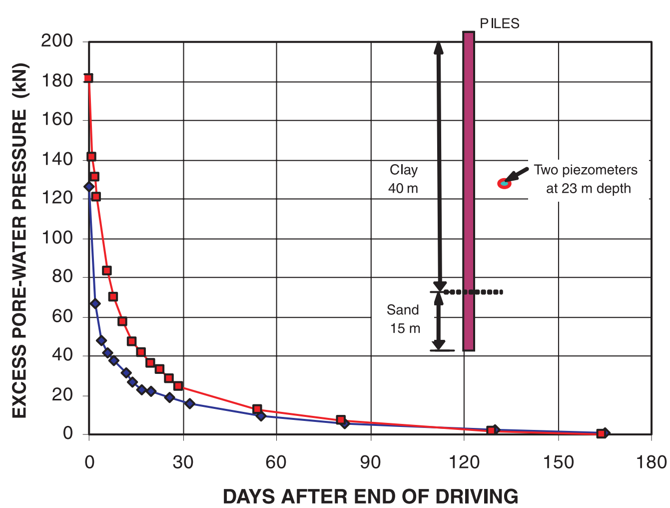


Fig. 32. Pore pressure measured at 23 m depth at the Bäckebol site.



fore the drag load continued to develop, as shown in Figs. 34 and 35. The rapid placement of the additional 360 kN on the piles (test activity step 4) temporarily eliminated the drag load and actually created a small positive shaft resistance along the piles. Each of the two applications of load to the pile heads resulted in a total pile shortening of about 2–3 mm over the length of the pile in the clay. The shortening at the 20 m mid-depth is about 1–2 mm. After the two loads had been placed on the pile heads, the total pile shortening since the pile driving was 9 mm, about equal to or marginally smaller than the settlement of the ground surface close to the piles at that time. That is, the forces and force changes in the pile are associated with relative movements between the pile and the soil of less than about 2 mm.

After adding the load to the pile heads and about a year of continued monitoring, a 2 m high layer of fill was placed

Fig. 33. Bäckebol site: (A) heave (negative numbers), and (B) settlement measured within the clay near pile PII during the first 150 days after driving (broken line shows approximate trend). EOD, end of driving.

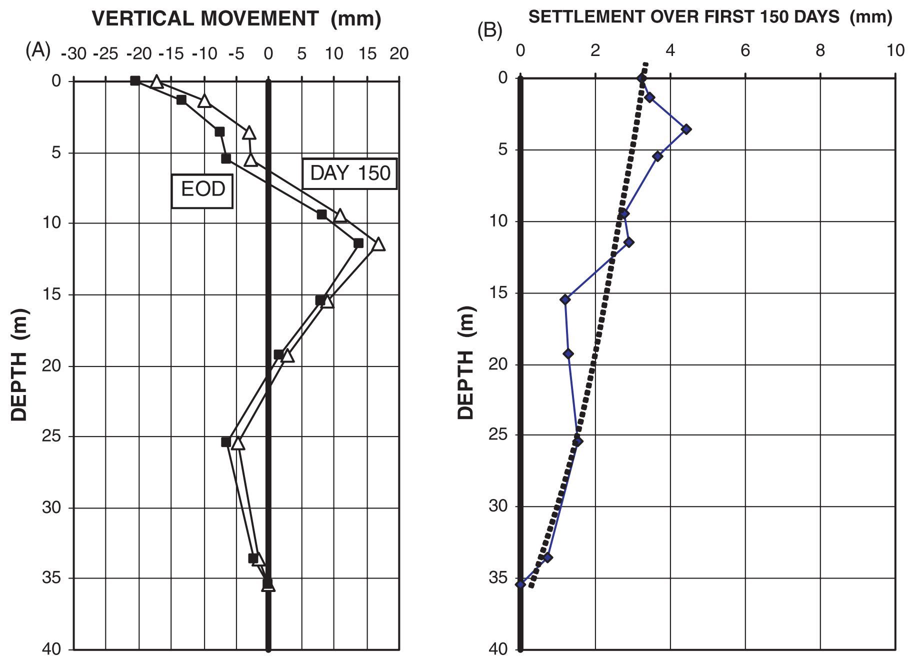
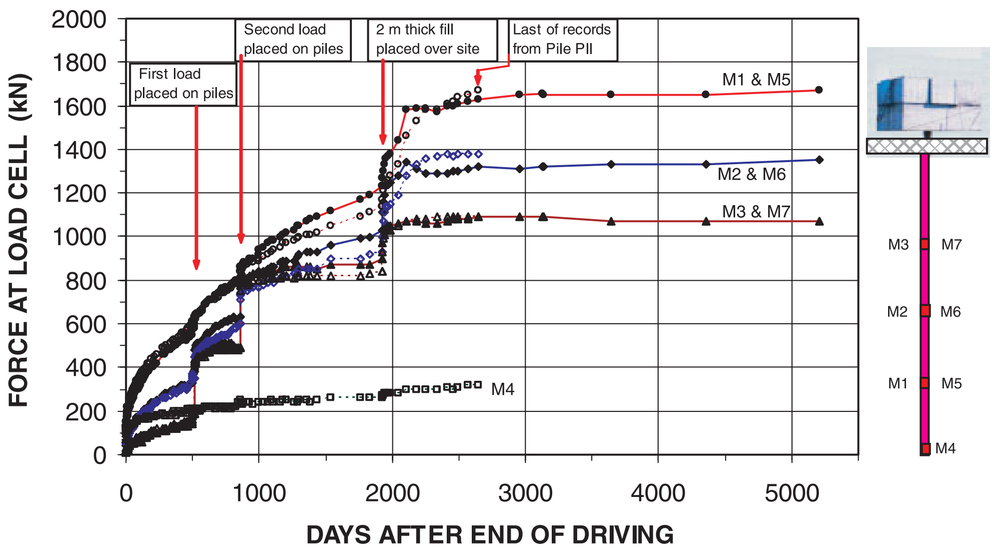


Fig. 34. Loads measured in the piles from the end of driving at the Bäckebol site. M1–M7, load cells.

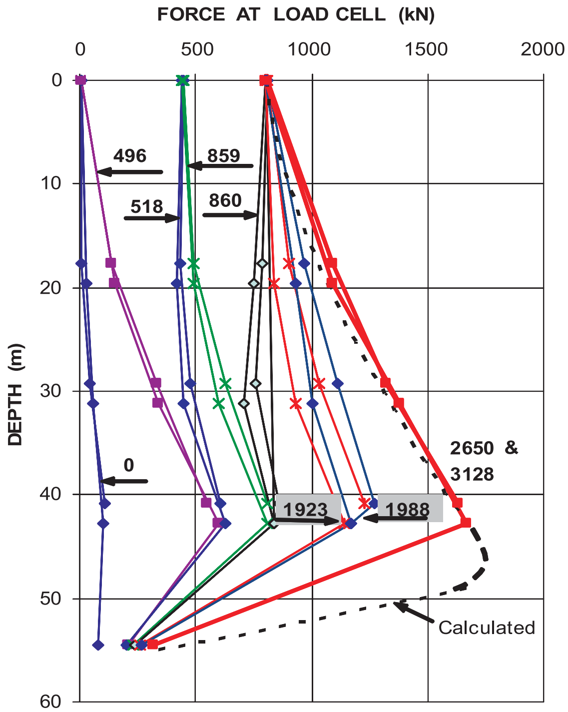


around the piles in a circular area with a 41 m diameter. The fill was placed during days 1923–1988. The fill accelerated the settlement and the development of drag load. Within about 3 months, the forces in the piles reached their maximum values and then remained constant for the duration of the following 3000 days of monitoring at the site. Pile PII was damaged above ground, breaking the cable connections to the load cells some time after day 2650, and no further records were obtained from the load cells in PII. Monitoring of the site and of pile PI continued until day 5206, almost 15 years after the end of pile driving.

The maximum load measured in load cell M5 was 1670 kN. Subtracting the load applied to the pile head, the maximum drag load recorded by load cell M5 is 870 kN. A calculation applying the vane shear strength as negative skin friction results in a calculated drag load of 3500 kN, about four times larger than the measured drag load. According to Swedish practice, described by Holtz and Wennerstrand (1972), a liquid limit of 85 necessitates reduction to 75% of the measured value, i.e., the calculated drag load would become about 2600 kN, still much larger than the measured drag load.

An effective stress analysis has been matched to the distribution of pile load measured on day 2650 in pile PII and day 3128 in pile PI. The beta coefficients in the clay for fitting the calculations to the measured loads between the ground surface and the first load cell level, between the first and second load cell levels, and between the second and third

Fig. 35. Distribution of load in the piles at the end of driving at the Bäckebol site immediately before and after adding the first load (days 496 and 518), immediately before and after adding the second load (days 859 and 860), immediately before and after placing the fill (days 1923 and 1988), and at final stabilized loads (pile PI at day 2650 and pile PII at day 3128).



load cell levels are 0.18, 0.15, and 0.14, which are low values. The broken line in Fig. 35 indicates the calculated distribution. The measured forces suggest that the force equilibrium lies below load cells M1 and M5. The broken line is drawn, somewhat arbitrarily, to reflect this. The slope of the broken line in the sand upward from the toe load cell is calculated using a beta coefficient of 0.60.

The average unit shear forces along the pile are obtained from differentiating the loads between the load cells and are presented in Fig. 36. For clarity, the values are only plotted up to day 3000. The values are almost constant beyond that date. The average shaft shear, with a maximum value corresponding to a beta coefficient of 0.33 recorded in pile PII between load cells M4 and M5 in the sand, includes a short distance of negative skin friction above the neutral plane and the positive shaft resistance below the neutral plane. Calculations using the mentioned beta coefficient of 0.60 in the sand place the location of the neutral plane about 4 m below load cell M, at a depth of 46.5 m.

To measure the pile shortening, new telltales were installed in pile PI at the start of the placing of the fill. Together with the survey of the pile head, the telltales determined the pile toe movement, as shown in Fig. 37. Figure 38 shows the pile toe load movement from the start of the test. The toe movement is accurately known only in pile PI, and the toe load only in pile PII (load cell M4). Figure 38 shows a combination of the two measurements: toe load versus toe movement. The buoyant weight of the pile has been subtracted from all load values. The starting point is the net toe load immediately after the end of the pile driving. As shown, the load–movement curve is approximately linear and does not show any sign of approaching an ultimate value. The maximum toe penetration corresponds to about 5% of the pile diameter.

The measurements and analysis indicate that the total ultimate shaft resistance in the clay and underlying sand was about 3000 kN. The mobilized toe resistance was 300 kN (including the pile weight), for a total toe penetration of about 13 mm. The total pile shortening for the 800 kN applied load plus drag load was about 18 mm.

Measurements showed that most settlement occurred in the upper layers. Figure 39 shows the settlement measured next to pile PII at different days after the start. The data clearly show that most settlement occurred above the 10 m depth where the stress from the fill exceeded or was about equal to the preconsolidation margin. The settlement data are inconsistent if the survey information would be taken as deciding for the long-term settlement of the ground surface. If so, all three multipoint settlement hoses would indicate that the settlement below the 32 m depth was three times larger than the 9 mm of settlement measured to have occurred between the depths of 10 and 32 m. Obviously, the casing supporting the upper portion of the multipoint settlement hose appears to have moved, probably as a result of seasonal influences. The individual measurements presented in Fig. 39 are plotted as if no settlement occurred below 32 m depth from the time that the fill was placed. Clearly, a few millimetres of settlement must have occurred also below the lowest point.

The measured settlements below 10 m depth are shown in Fig. 40 as measured on day 2044, the day all of the fill had been placed, and on day 4362, almost 7 years later when the last reading was taken. Below a depth of about 30 m, the difference between the curves is only 2 mm. Assigning the settlements to be equal at a depth of 32 m has probably cut a few millimetres off the true settlement. Still, it is clear that the settlement at depth was very small, and yet it fully mobilized negative skin friction between load cells M1 and M2 and M5 and M6 within a year after placing the fill, as shown in Fig. 36.

A comparison between the measured settlement and that calculated from the soil data is shown in Fig. 41. Figure 41A shows the effective stress, the stress from the fill as per the Boussinesq distribution, and the distribution of preconsolidation stress. Figure 41B shows the distribution of settlement calculated from the values plotted in Fig. 41A and the settlement distribution measured on day 4362 next to pile PII. Note the stress from the fill is so close to the preconsolidation margin that a change of 1 kPa or so, one way or the other, would significantly alter the calculated settlement distribution. The two curves show agreement, however, and support the conclusion that most of the settlement at the site from the placed fill occurred in the upper 10 m of soil and that only a very small relative settlement occurred between the pile and the clay below a depth of 10 m.

Conclusions

Several generally applicable conclusions of importance for the design of piled foundations are drawn from the reported full-scale tests and summarized as follows. Presenting

Fig. 36. Measured average shear forces between the load cells at the Bäckebol site.

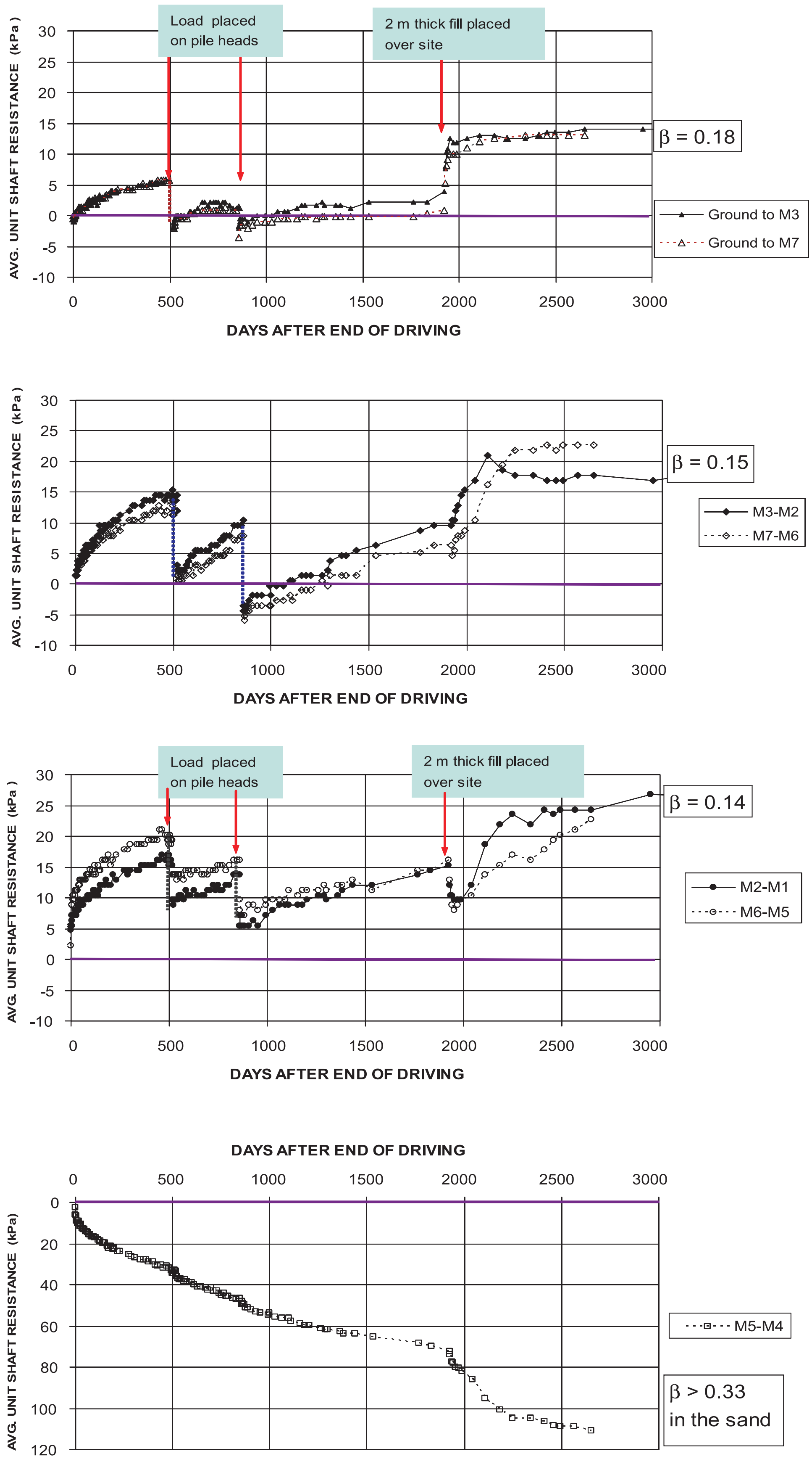


Fig. 37. Movement of pile heads and pile toe of pile PII since the start of placing fill at the Bäckebol site.

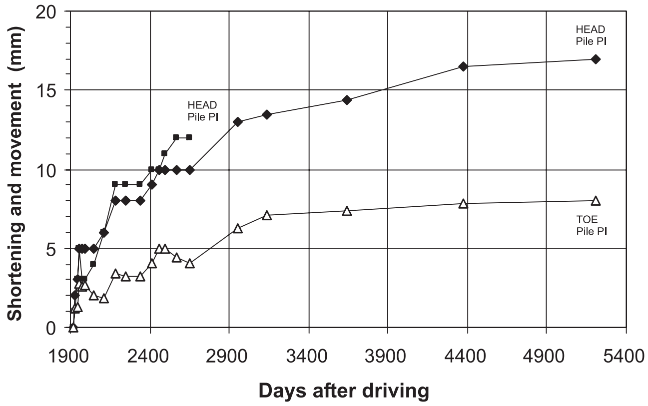
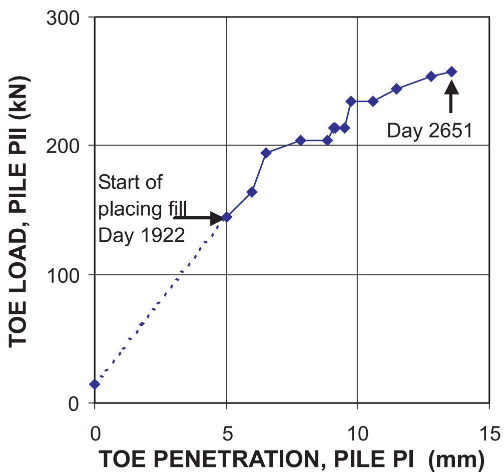


Fig. 38. Toe load of pile PII versus toe penetration of pile PI at the Bäckebol site.



design recommendations, however, is outside the scope of this paper. For design recommendations, see Fellenius (2004).

(1) Piles are usually installed to transfer loads through soft or loose soil layers to more competent soil. Under such conditions, negative skin friction will always develop along the piles and accumulate to a drag load.
(2) The case histories have shown that effective stress governs the load transfer from soil to pile and from pile to soil.
(3) Load distribution calculated from undrained shear strength values may occasionally agree with actual load distribution when no excess pore pressures exist, but this appears to be more a coincidence than a general rule.
(4) At all times, an equilibrium will exist between loads sustained on the pile head and the drag load acting downward and the positive shaft resistance and toe resistance acting upward. That is, a force-equilibrium point, called the neutral plane, will always develop. The location of the equilibrium will adjust to a change of sustained load or change of pore pressure.
(5) If the soil settlement is large and the pile toe response is stiff, the neutral plane will be located very close to the pile toe.

Fig. 39. Distribution of settlement measured in multipoint hose (A) next to pile PII and (B) 5 m away from both piles at the Bäckebol site.

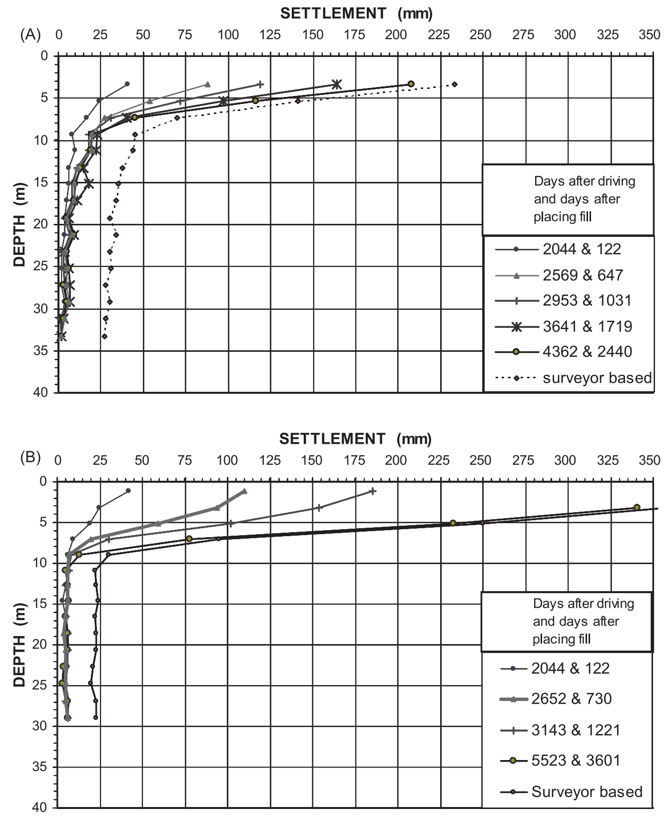
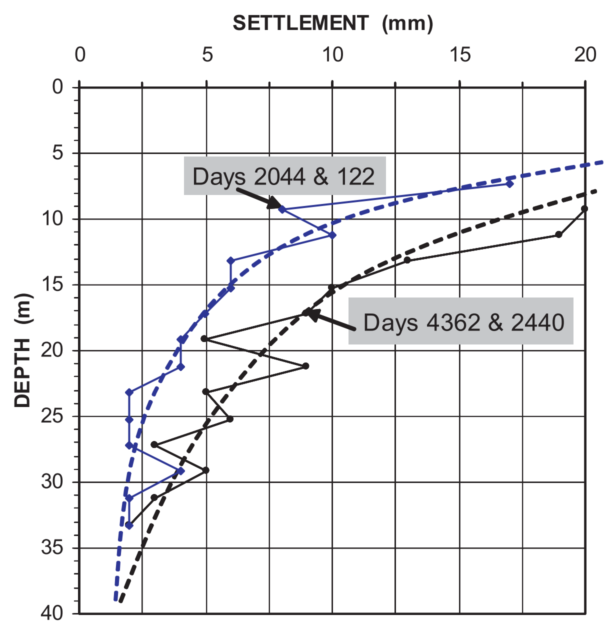
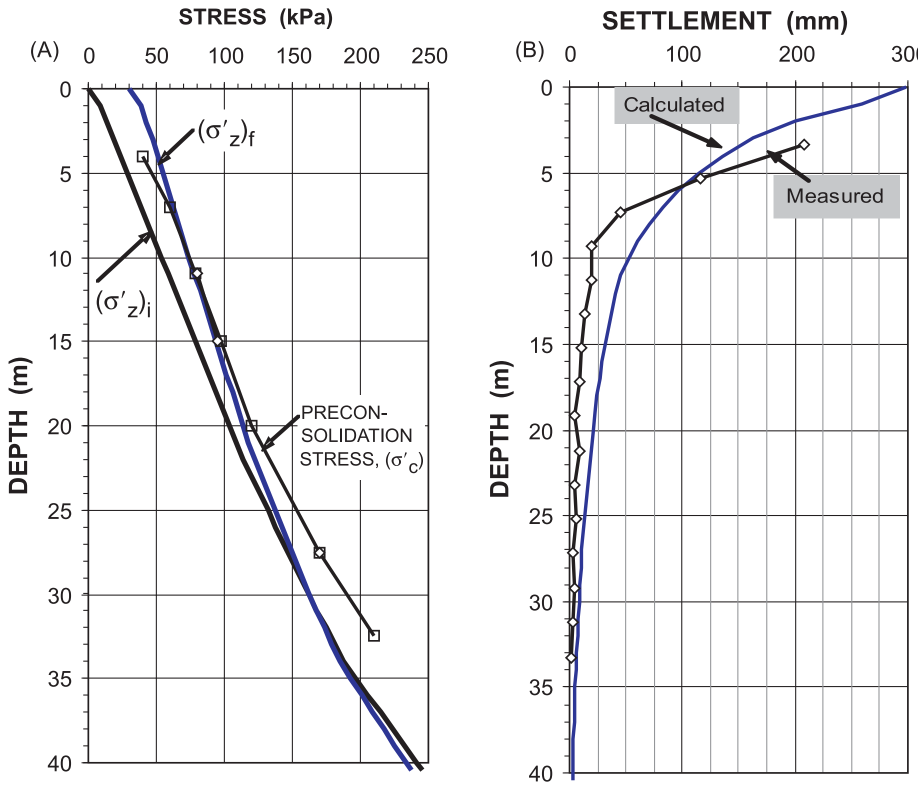


Fig. 40. Distributions of settlement measured in multipoint hose below 10 m depth at the Bäckebol site. Days after start (first number) and days after placing of fill (second number) are indicated. The broken lines indicate the trend of the curves.



(6) A few millimetres of movement between the pile surface and the soil can be sufficient to fully mobilize the shaft shear in the negative and positive directions.

Fig. 41. Bäckebol site: (A) distribution of effective stress ($(\sigma'_c)_i$), preconsolidation stress (σ'_c), and final stress ($(\sigma'_z)_f$); (B) distribution of calculated settlement measured at day 4362 next to pile PII.



(7) The length of the zone of transition from the negative-direction shear force to the positive direction is a function of the magnitude of the movement between the pile surface and the soil, or more precisely the relative settlement gradient. A small relative movement will result in a long transition zone, and a large relative movement will result in a short transition zone.

(8) The location of the neutral plane is the location of the force equilibrium, which is where the transition from negative to positive shear direction is completed. It is also the location where there is no relative movement between the pile and the soil. That is, at the neutral plane, the settlement of the soil and the settlement of the pile are equal, from which follows that the settlement of the pile head is the soil settlement at the neutral plane plus the pile shortening for the load.

(9) The load in a pile at the neutral plane is the maximum load in the pile, and it is the sum of the sustained load (dead load) and the drag load. A temporary load, such as a live load, will not add to the load at the neutral plane. The load at the neutral plane will not change (unless the live load is so large as to totally eliminate the drag load, that is, the live load has to be twice the drag load at the neutral plane, a rather unlikely design case).

(10) Two single piles of different lengths, or of the same length but having a different pile toe response, with the same dead load applied to the pile head can have a different depth to the neutral plane. They will therefore experience different downdrag (settlement) because settlement at the two neutral plane locations is different. It follows that two such piles connected to a common pile cap will have the same depth to the neutral plane if the cap is sufficiently rigid to ensure that the pile head movements are the same for both piles. Therefore, the two piles cannot have the same dead load but must attract different magnitude dead loads, as determined by the magnitudes necessary for the piles to develop that same location of the neutral plane. It also follows that the pile toe penetration must be of a similar magnitude, and, inasmuch as the pile toe stiffnesses are different, the pile toe resistances will be different in magnitude.

(11) A very thin — about a millimetre or two — coat of bitumen with a wide range of viscosity will significantly reduce the shear forces along a pile surface.

(12) The effect of the bitumen is, of course, to reduce the negative skin friction and the drag load. It will also move the neutral plane to a greater depth, possibly to a depth where the settlement is smaller, and therefore the downdrag will be smaller. The drag load will have changed to a lesser degree. Note that the bitumen coat will also reduce the pile capacity and the factor of safety of the pile in carrying the applied load.

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