RECENT ADVANCES IN THE DESIGN OF PILES
FOR AXIAL LOADS, DRAGLOADS, DOWNDRAG, AND SETTLEMENT

ASCE and Port of NY & NJ Seminar, April 22 and 23, 1998

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Introduction

Design of piles requires understanding of how load is transferred from pile to soil and, less obvious but equally important, from soil to pile. As long as the reference to actual behavior had to take the clue from testing of model piles and from full-scale tests where the only measurements were load-movement taken at the pile head, the understanding of the load-transfer mechanism left much to be desired. Starting in the 1960's, results became available from instrumented field tests resulting in a much improved state-of-the-art. Initially, however, the situation was marred by misinterpretation of the test results leading to the belief in the existence of a "Critical Depth" (Fellenius and Altaee, 1995). The "Critical Depth" appears when the "Residual Load" acting on the pile at the start of the static loading test is neglected. Residual load is a preloading of a pile by the soil, some of which develops during the driving and some is caused by the reconsolidation of the soil (set-up effect). A contributory reason to the rise and acceptance of the concept of a Critical Depth is that when the interpretation of the results of testing small-scale piles in the laboratory fails to properly model the stress scale for soil a Critical depth appears to exist. Early works addressing residual loads affecting the interpretation of load-transfer in piles were presented by Hunter and Davisson (1969), Fellenius and Samson (1975), Holloway (1976), and Holloway et al. (1978).

Load-transfer from a pile to the soil or from soil to a pile is primarily governed by the effective stress in the soil, and both shaft and toe resistances are proportional to the effective overburden stress. That is, the load transfer is best analyzed using the "beta method" of analysis. Contributing factors are the state of stress, preconsolidation, soil type, etc. This can be considered well established in the current state-of-the-art.

In pile foundation design, the allowable load is determined by dividing the pile capacity by a factor of safety applied to the pile capacity (be it established by testing or by analysis; the factor of safety is larger for the latter indicating the greater uncertainty of theoretical analysis as opposed to actual testing). The design load is the sum of sustained, permanent, or *dead load* and of temporary, transient, or *live load* and the design aims to ensure that the design load does not exceed the allowable load. The approach is quite straight-forward, although a particular case can sometimes be complicated by aspects of liquefaction and other seismic concerns, lateral loads acting together with the axial loads, uplift loading, and distribution of load between piles in a group, etc.

A pile foundation is the usual foundation alternative where a footing foundation would settle excessively. However, it is not usual to calculate settlement for a pile foundation. Instead, it is presupposed that if the factor of safety is all right, the foundation will not settle. Most engineers are familiar with the Terzaghi-Peck equivalent-footing approach to the calculation of settlement of a foundation placed on a group of shaft bearing piles in clay, a "floating pile foundation", analyzed as a virtual footing placed at a

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1) These notes are prepared as background to a presentation and are not intended as full coverage of the topic. The author apologizes for that lack of time has prevented conversion of foreign language text to English in some of the referenced figures.

depth equal to the lower third point of the pile embedment length. Most engineers would not use "floating piles" in a design, however. In other words, piles are means for bypassing weak and compressible soil layers and to transfer the loads to competent layers at depth.

The situation becomes a little more complex when the designer recognizes that the soil surrounding the piles will settle due to fill, groundwater lowering, or other causes. The soil will then tend to hang on the piles and transfer load to the pile due to "negative skin friction". The load is called "Dragload". Negative skin friction develops in the upper portion of a pile and the resulting dragload plus the applied dead load will always be in equilibrium with resistance (shaft resistance acting in the positive direction and some toe resistance) in the lower portion. The change over from one direction to the other is called "the neutral plane". This is where there is no relative movement between the pile and the soil—they move together. If the soil layer at the neutral plane settles, then, the pile will settle along with the soil and the pile will be "dragged down".

As will be shown in the following, "downdrag" is generally an undesirable occurrence. However, the "dragload" is not. The difference is not generally appreciated. Downdrag is a settlement problem and if downdrag occurs, the consequences can be very serious for a pile foundation. The dragload is of concern only for the pile structural strength and the designer must ensure the load can be accommodated without the pile experiencing structural distress. The dragload must not be added to the loads from the structure when checking that the design load does not exceed the allowable load. Neither should the capacity value be reduced by the dragload. Treating the dragload as a load similar to the loads from the structure is a very costly error. (It is unfortunate that the upward direction of shear forces along the pile is defined as the positive direction, and the downward direction is the negative, because there is nothing negative in dragload. In reality, it is a good load that prestresses the pile and has the effect of reducing the movements due to live loads. It is more akin to the load in a prestressed pile resulting from the pretensioning of the strands than a load from the structure supported on the piles. Also the latter, of course, is of no relevance to the bearing capacity of the pile).

Case Histories that Broke New Ground

The foregoing statements are best explained by looking at a few case histories that broke new ground, emphasizing what specifically was learned as reflected in current methods of analysis.

Bjerrum et. al., (1969) presented a case history of 300-mm steel piles driven to bedrock in Norway. The piles were driven closed-toe through about 20 m of silty clay on which an about 10 m thick fill had been placed. The piles were instrumented with telltales to measure pile compression at different depths. The compression was transferred to average load in the pile between the telltale ends. Fig. 1 shows a vertical view of the site and the piles. The key piles are Pile A and Pile B. The latter pile was supplied with a 1 mm thick coat of bitumen of penetration 80 - 100 to reduce the negative skin friction along the pile.

Fig. 2 shows the load distribution in the piles as evaluated from the telltale data after 18 months of observations. The diagram to the far right compares the load distribution evaluated for Piles A and B and demonstrates the success of the bitumen coating: Pile A shortened due to a dragload of about 110 tonnes, while the records from Pile B indicated only about a tenth of that load. Notice also the appearance of a neutral plane for Pile A. The maximum dragload was calculated in an effective stress analysis to correspond to a beta-coefficient on the effective overburden stress of 0.25. The fill introduced excess pore pressure into the clay and the "Δu" shown in the diagram to the left indicates the excess pore pressure remaining after 18 months. The ground surface at the site settled about 160 mm during the 18-month observation period.

Fig. 3 shows the compression of the piles during 13 months of monitoring. Pile A and B shortened about 10 mm and 1 mm, respectively. Notice the recovery of the shortening that occurred in early March 1967 coinciding with the driving of neighboring piles. The driving of the neighboring piles increased the pore pressures and the effective soil stress decreased, correspondingly. When the effective stress reduced, the side shear between the pile and the soil also reduced, that is, the negative skin friction reduced. These records clearly demonstrate that load transfer is governed by effective stress even in clay.

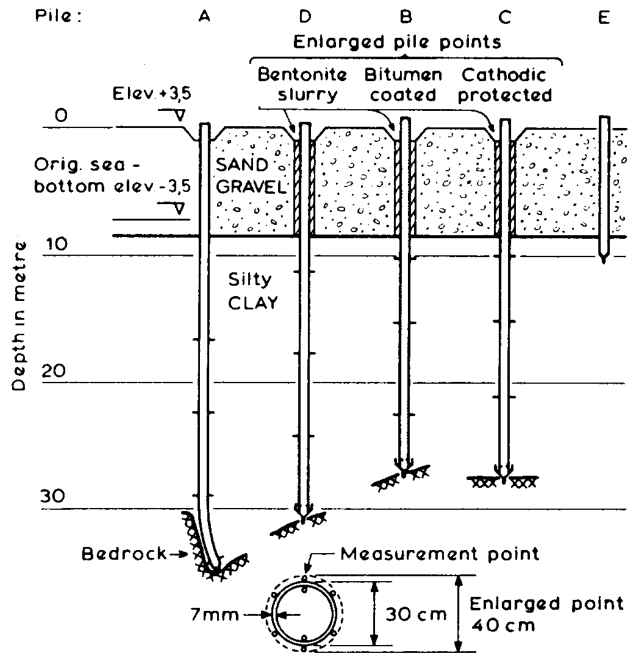


Fig. 1. Vertical view of test site and piles (Bjerrum et al., 1969)

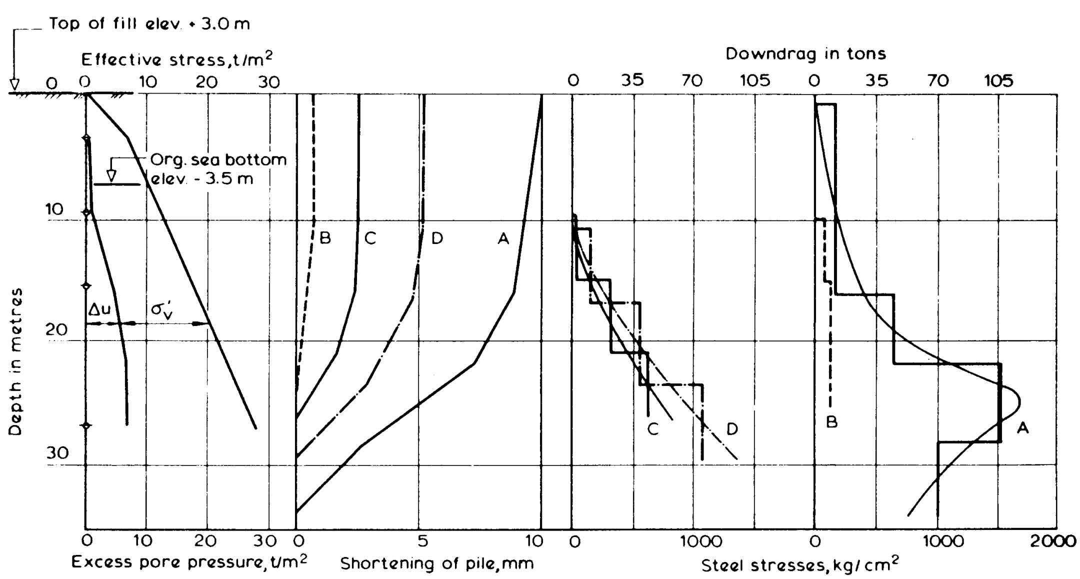


Fig. 2. Distribution of soil stress, excess pore pressure, shortening of piles, and load distribution (Bjerrum et al., 1969)

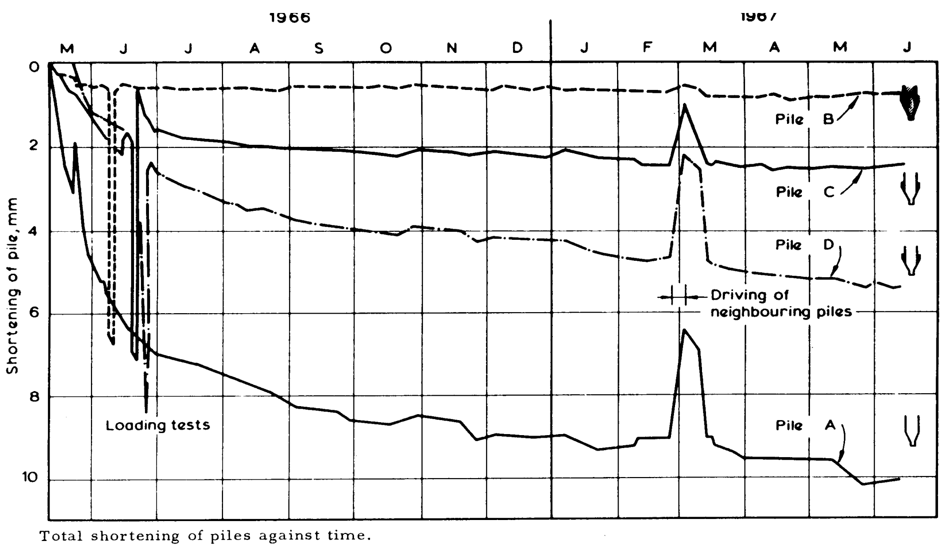


Fig. 3. Pile shortening versus days after initial driving (Bjerrum et al., 1969)

As stated by Bjerrum et al. (1969), because the piles were subjected to dragload, the usual design load (120 tonnes) would have to be reduced by 50 % or the number of piles be doubled. The solution adopted was to bitumen-coat the piles and to equip all piles with an enlarged pile shoe to protect the bitumen coat when driving the piles. The costs of the bitumen and the pile shoe amounted to 20 % increase over the cost of the same number of untreated piles. The total cost of the piling is indicated to have been more than 5 million Norwegian crowns, about one million us dollars (1966 prices) and, therefore, eliminating the dragload with the bitumen-coat saved about $800,000 for the owner. Without belittling this important case history, however, the measures to reduce the negative skin friction were only necessary if downdrag, i.e. excessive settlement, would occur, and/or if the pile structural strength would be inadequate to resist the dead load plus the dragload. In actual fact, the toe bearing piles would not settle and the strength of the piles was more than adequate. Therefore, by what we know today, the $200,000 expense was not necessary.

The author's paper includes a summary of the main results of other studies by the Norwegian Geotechnical Institute, showing that the maximum dragload for steel piles ranged from 20 tonnes through more than 400 tonnes for piles having embedment lengths of 30 to 60 m. Ground surface settlement ranged from zero to 2 m. The surprisingly large dragload values have overshadowed the observation that full magnitude dragload developed already at insignificant relative movement between the pile and the soil.

Endo et al. (1969) presented a very ambitious study in Japan on dragloads on four instrumented steel piles during a period of three years. The soil profile at the site consists of thick alluvium over a buried river: a 10 m thick layer of silty sand followed by silt to a depth of about 25 m followed by alternating layers of silt and sandy silt to a depth of about 44 m. The pore water pressure at the site was affected by pumping in the lower silt layer to obtain water for an industrial plant creating a downward gradient at the site. The ensuing consolidation of the soils caused the soil to settle and hang on the piles.

Fig. 4 shows a vertical view of the site and the piles, showing the instrumentation for measuring the vertical distribution of settlement and pore pressure. The bars to the right indicate four strain-gage instrumented test piles denoted ${}_{O}E_{43}$, ${}_{C}E_{43}$, ${}_{C}F_{31}$, and ${}_{C}B_{43}$. The piles were driven very lightly, the penetration for the last blow of the three vertical piles ranged between 22 mm to 25 mm and was 10 mm for the inclined pile. For the three longer piles, the pile driving stopped with the pile toe in the silt and fine sand layer where the SPT N-value was about 10 bl/0.3 m to 20 bl/0.3 m. Although the authors called the three longer piles toe-bearing piles, the data indicate that all test piles can be considered to be shaft bearing piles with only moderate toe resistance.

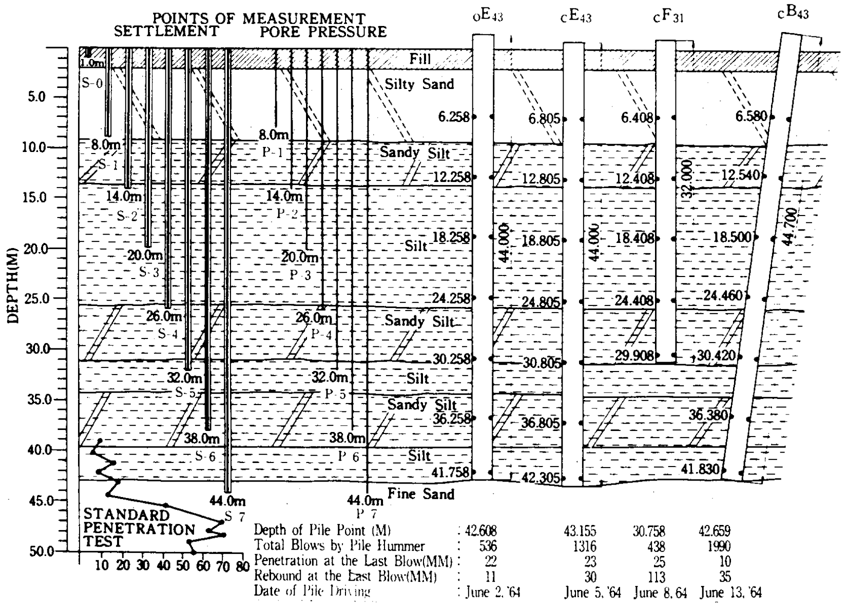
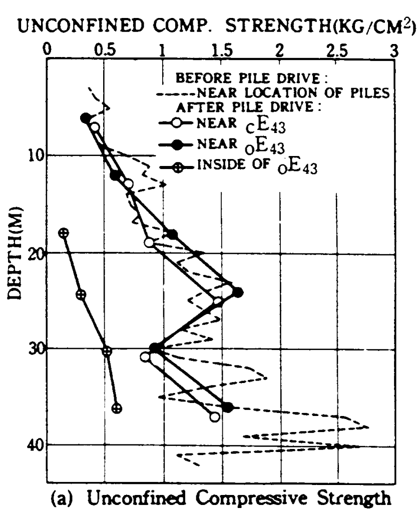


Fig. 4. Vertical view of test site and piles (Endo et al., 1969)

Fig. 5 shows a diagram of the distribution of the unconfined compression strength. Fig. 6 shows the pore pressure profile at the site indicating that the downward gradient is 32 m over 44 m.



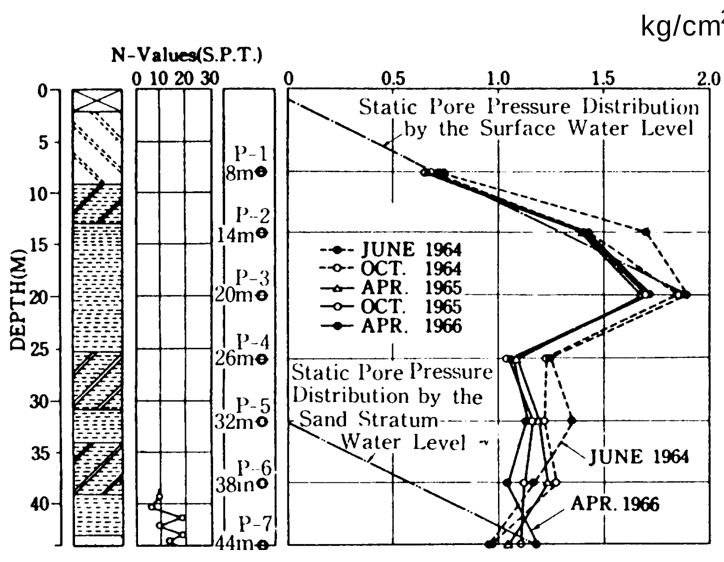


Fig. 5. Unconfined compression strength vs. depth (Endo et al., 1969)

Fig. 6. Pore pressure profile at the test site (Endo et al., 1969)

The study started in June 1964 and continued until March 967. Fig. 7 shows the stresses determined from strain gages located at seven levels in Pile $_{C}E_{43}$. As seen, most of the strain in the upper portion of the piles was recorded during the first few months of the observation period. For the gages located in the lower portion of the pile, very little additional strain occurred after April 1965.

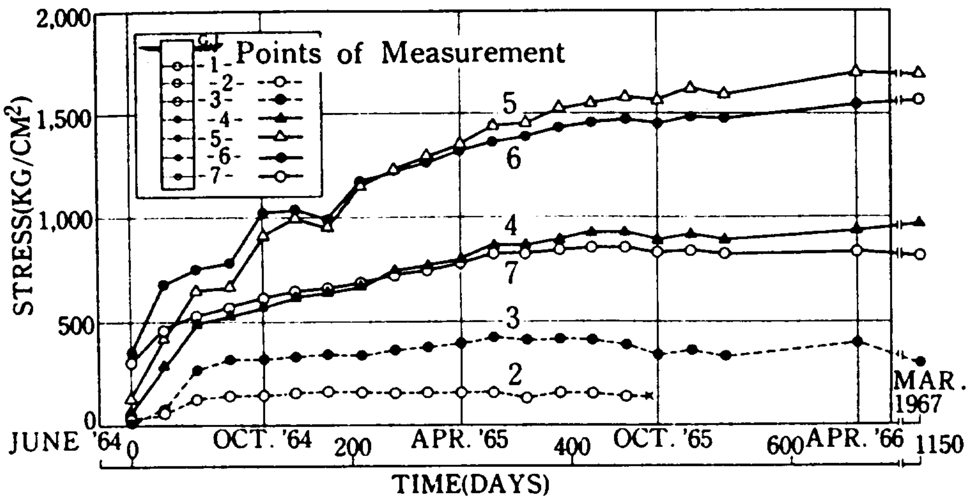


Fig. 7. Shortening of Pile $_CE_{43}$ measured during June 1964 through March 1967 (Endo et al., 1969)

Fig. 8 presents the distribution of load in the piles. The diagram to the left shows the development for Pile Pile $_CE_{43}$ at different times, indicating that the negative skin friction and the development of a neutral plane at a depth of about 30 m occurred early in the observation period. The diagram to the right shows the distribution for all four piles in April 1967. The toe resistance for Pile $_OE_{43}$ is significantly smaller than that for Piles $_CE_{43}$ and $_CB_{43}$. The diagrams show that the negative skin friction above the neutral plane is essentially equal for the piles as is the and the positive shaft resistance below the neutral plane.

As seen in the diagrams, the authors related the negative skin friction to the unconfined compression strength. In the text, however, they also mention that the negative skin friction followed the effective stress distribution corresponding to a beta-coefficient of 0.35. The toe coefficient is not given. The diagrams indicate that the mobilized toe resistance ranged from about 80 tonnes through 160 tonnes and the total shaft resistance extrapolated over the full pile length was about 500 tonnes to 600 tonnes.

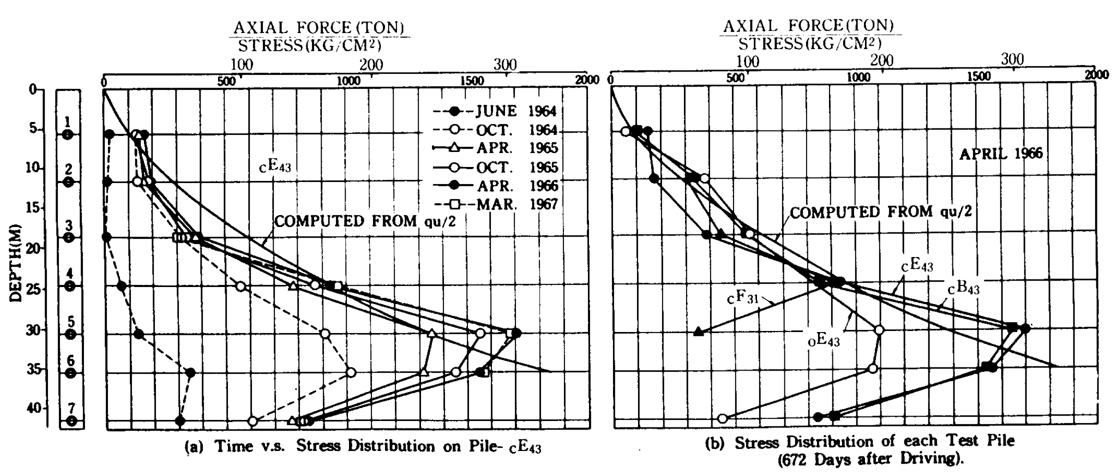


Fig. 8. Load distribution in the test piles (Endo et al., 1969)

Fig. 9 presents the results of observations of settlement at the site combined with the settlement observations for Pile $_CE_{43}$. The settlement of the ground surface after two years was about 130 mm and most of the settlement occurred below the depth of 20 m. As shown, the curves for soil and pile settlement intersect at a depth of about 30 m, which establishes the neutral plane at the same depth as found from the measurements of load in the piles. At the neutral plane, the pile settlement was about 40 mm. The pile head settled about 60 mm and the quoted case is a good example of downdrag. The pile toe penetrated about 20 mm into the silt and sand layer beyond the about 20 mm settlement occurring in that layer below the level of the pile toe.

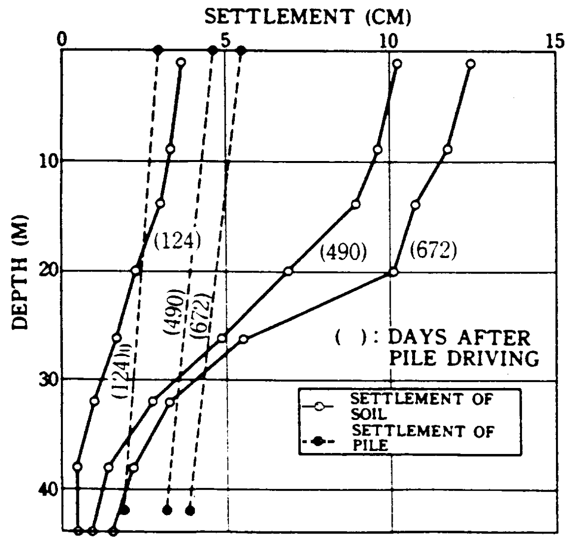


Fig. 9. Distribution of settlement of the soil and Pile $_{C}E_{43}$ (Endo et al., 1969)

Fellenius and Broms (1969) and **Fellenius (1972)** presented a case history from Sweden of 300 mm precast concrete piles driven through 40 m of clay and about 12 m into a sand deposit. Results obtained after 1972 were published by **Bjerin (1977)**. The soil profile is shown in Fig. 10.

The two test piles were equipped with special load cells for determining the load in the piles. Each pile had three load cells were placed at equal distances from the pile toe. One Pile, Pile II, also had a load cell at the pile toe. Pile II was driven 2.0 m deeper than Pile I causing the depth to load cells to differ by 2.0 m. Initially, the pile head had no load. One and a half year into the test, to model building a structure on the piles, a concrete platform was cast on the pile head imposing a load of 440 KN. One year later, an additional 360 KN load was applied. Three years later again, a fill was placed on the ground around the piles.

Fig. 11 shows the measured load during seven years after the pile driving. The sketch in the top left corner shows the location of the load cells. The agreement of the load development for the two piles is very good. Initially, the load built up rapidly in the piles, coinciding with the dissipation of the induced excess pore pressures and the reconsolidation of the soil after the driving. The continued build-up of load was at first a source of bewilderment at the site as no settlement was observed. With time, however, it was established that the ground surface at the site was settling at a rate slightly smaller than about 1 mm per year. The relative movement between the soil and the pile at the mid-level Load Cells M2 and M6 during the first year was smaller than 1 mm and, yet, it was sufficient to cause a substantial negative skin friction to develop.

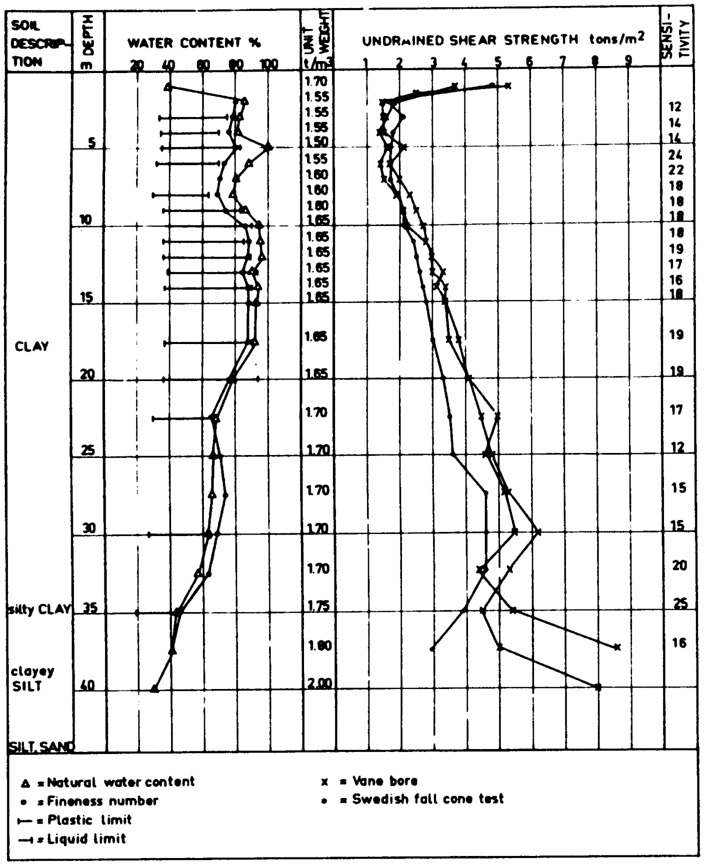


Fig. 10. Soil profile at the Bäckebol site (Fellenius 1972)

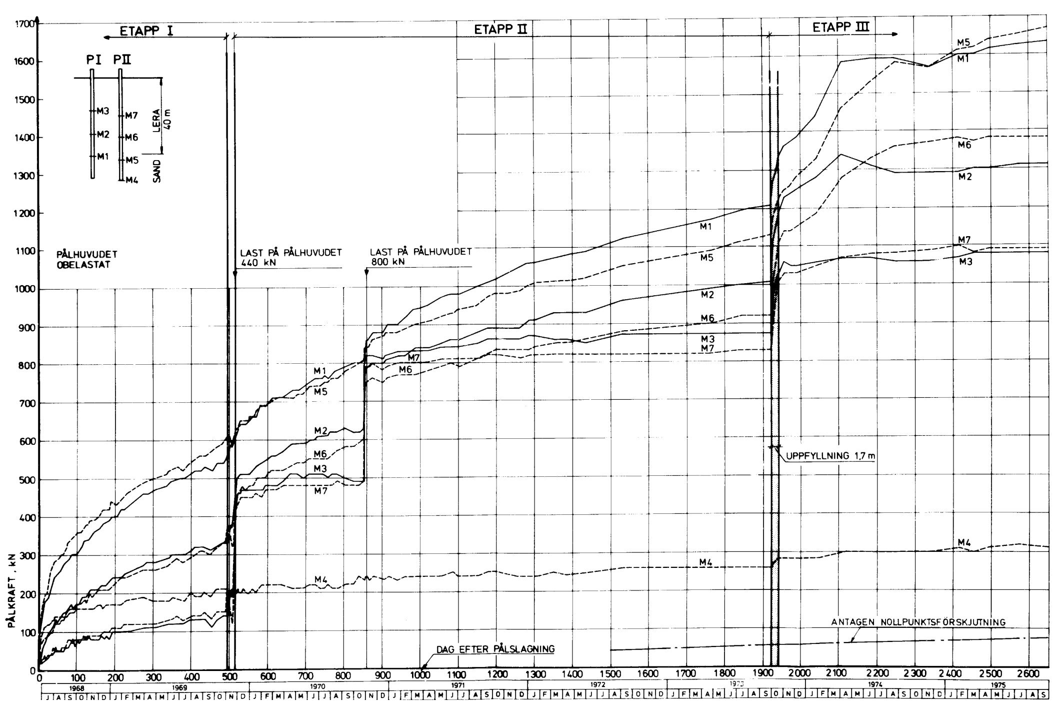


Fig. 11 Measured loads versus time after driving (Bjerin, 1977)

Fig. 12 presents the vertical distribution of load in the piles over the observation period for Pile II. The load curve marked "0" was measured immediately after the pile had been driven and the load distribution is about equal to the buoyant weight of the pile. Notice that when the load was applied to the pile head, it did not add to the load down in the pile, but the dragload was reduced by the amount of the applied load. Indeed, the rapid raising of the load at the pile head from 440 KN to 800 KN at Day 859 reversed the direction of shear from negative to positive. An obvious conclusion is that live loads and dragloads must not be combined, i. e., be assumed to act at the same time.

The Fig. 12 diagram also shows that no or only little increase of the load in the pile (Load Cells M6 and M7) occurred beyond the first year after the placing of the fill.

The load on the ground imposed by the fill started a period of more significant settlement. The ground surface settled about 100 mm over the two-year observation period. Full consolidation was expected to take many years and be about 1 m. At the level of the uppermost load cells, M3 and M7, the measured soil settlement was about 10 mm or smaller. Fig. 13 presents the measured settlement and also shows the calculated final consolidation settlement. The observation of the pile movement indicated a pile penetration into the sand layer of about 4 mm. The pile compression caused by the increase of load in the pile caused about an additional 4mm downward movement. Therefore, the balance, about 2mm is the relative movement between the pile and the soil during the two years after the placing of the fill occurring between the levels of Load Cells M6 and M7.

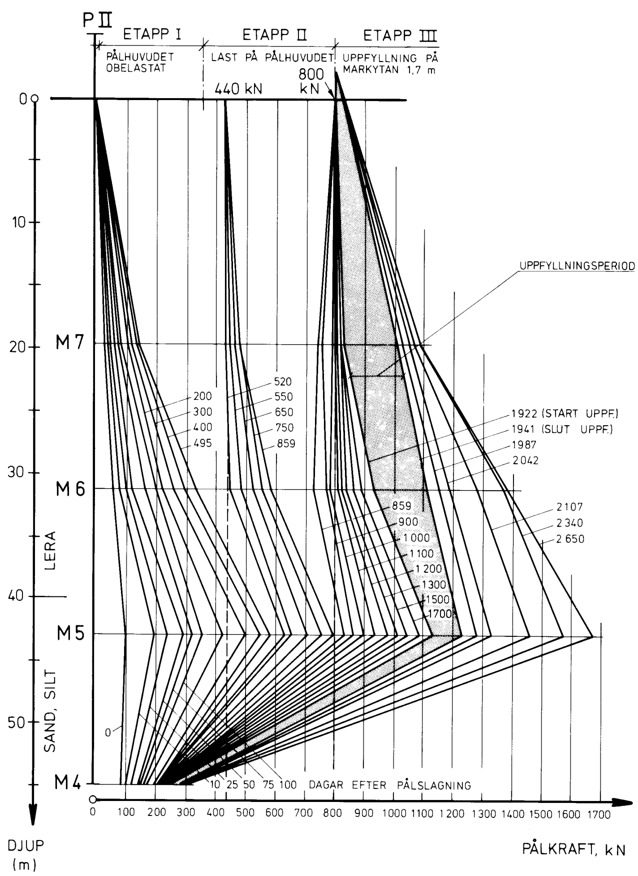


Fig. 12 Vertical distribution of load in Pile II (Bjerin, 1977)

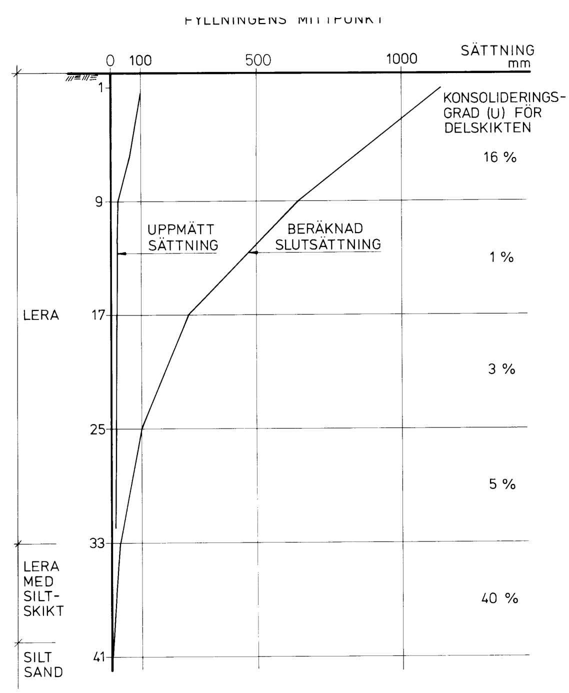


Fig. 13 Vertical distribution of measured and calculated consolidation settlement (Bjerin, 1977)

Fig. 14 presents the average shear resistance calculated between Load Cells M2 and M3 in Pile I and M6 and M6 in Pile II. The curves show that the loading at the pile head reduced and eliminated the negative skin friction. With time, it built up again and after the fill had been placed, the rate of build up increased to a maximum that then stayed constant. This maximum negative skin friction corresponds to about 25 % of the effective overburden stress and, as mentioned, required only a very small relative movement, about 2 mm, between the pile and the soil.

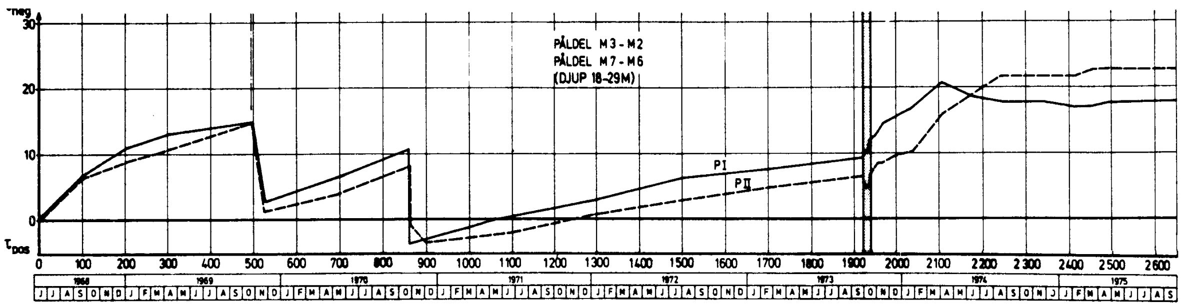


Fig. 14 Average shaft shear in Piles PI and PII between Load Cells M2 and M3, and M6 and M7 (Bjerin, 1977)

Walker et al. (1973), reported a study in Australia of negative skin friction on two 30 m, 760 mm, strain-gage instrumented, open-toe pipe piles driven through a 5 m sand layer over an about 15 m of preconsolidated silty clay deposit on silt and sand. The clay water content was 60 % - 80 % and the undrained shear strength was 40 KPa - 80 KPa. The preconsolidation stress in the clay was larger than the stress exerted by a 2 m fill placed on the ground surface. During a 200f-day period, 10 mm of surface settlement was observed. One test pile was provided with a 1 mm thick bitumen and one was uncoated. Fig. 15 presents the load distribution for the two piles, showing the very small loads in the coated pile as opposed to those in the uncoated piles. The diagram indicates that no toe resistance developed for the piles.

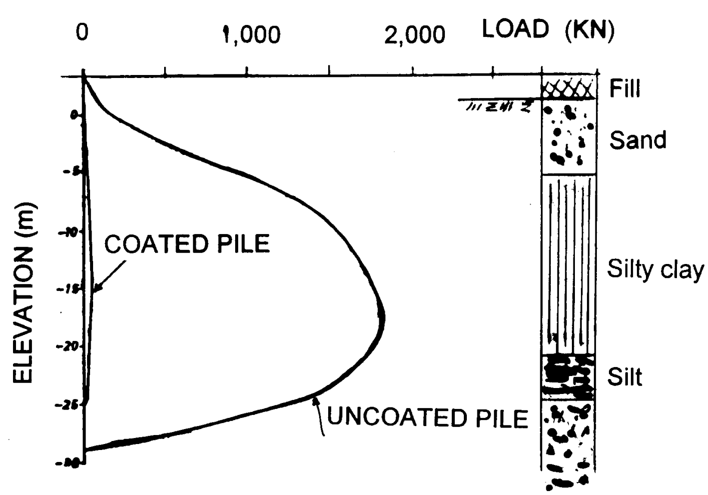


Fig. 15 Load distribution on two pipe piles, one bitumen-coated and one uncoated (Walker et al., 1973)

Bozozuk (1981) presented a case history of static loading on a 49 m long instrumented pile in Canada ten years after its initial driving during which period substantial negative skin friction had developed. The initial application of static load eliminated the negative skin friction. When the loads were increased to a value equal to the dragload, the pile failed in plunging. No increase of resistance was observed to occur below the neutral plane. This case demonstrates that the dragload had no influence on the bearing capacity of the pile and confirms that live load and dragload must not be assumed to occur at the same time in a pile.

Clemente (1979; 1981) reported a case history in the USA on 50 m, 430 mm prestressed concrete piles in soft clay and compared loads for piles having a thin coat to piles uncoated piles. The dragload was caused by settlements induced by a fill on the ground surface. The negative skin friction increased to a neutral plane at a depth of about 30 m. The bitumen-coating was efficient in reducing the negative skin friction.

What was learnt

The mentioned case histories are pioneering papers and important reference to the understanding of long-term behavior of piles. In summary, the case records established the following:

1. Load transfer along the pile shaft, be it by negative skin friction or by positive shaft resistance, follow effective stress principles, as does pile toe capacity. Knowledge of the pore pressure distribution, not just the location of groundwater table, is vital for correct design of pile foundations.
2. Shaft shear—negative skin friction or shaft resistance—develops for very small movement often no larger than the compression of the pile caused by the design load. Negative skin friction can develop from secondary compression alone. In essence, regardless of whether or not the settlement of the ground surface is of a noticeable magnitude, with time, all piles will develop negative skin friction and dragloads. At a level called neutral plane equilibrium will develop between the applied dead loads and the dragload above the neutral plane and positive shaft resistance below the neutral plane.
3. Dissipation of pore pressures induced by pile installation and reconsolidation of the soil, i. e., “set-up”, will introduce negative skin friction and residual loads in a pile.
4. Live loads will reverse the direction of relative movement between the pile and the soil and in the process eliminate an equal amount of the dragload. Live load and dragload do not act at the same time over the same length of pile.
5. The maximum load in the pile occurs at the neutral plane and is the sum of the dead loads and the dragload. It is important that this load does not exceed the structural strength of the pile (with an appropriate margin).
6. The dragload will not have any influence on the pile bearing capacity and the dragload must not be subtracted from the pile capacity when determining the allowable load. Neither must the dragload be added to the design loads when checking that the loads from the structure do not exceed the allowable load.
7. From a geotechnical view, the dragload is a beneficial force prestressing the pile and reducing the deformation that occurs from live loads. As long as the pile structural strength is not exceeded, the larger the dragload, the stiffer and better the pile foundation. Beyond the name, there is nothing negative in negative skin friction.

8. Historically, negative skin friction is associated with large settlement. However, the problem is not that of dragload, but of downdrag. Downdrag occurs where the pile settles with the soil and is associated with a small dragload. The neutral plane is where the pile and the soil settle equally. Therefore, the problem of downdrag has to be addressed in an analysis that determines the location of the neutral plane and the soil movement at the neutral plane.

9. Downdrag and dragloads are important aspects to include in a geotechnical design and they are neglected at one's peril. Excessive settlement may develop for pile foundations on shaft bearing piles with pile toes in compressible soils, where settlement develops due to a combination of the load on the piles (dead loads) and increase of effective stress caused by other factors, such as lowering the groundwater table or placing a fill on the ground. (Notice, dragload does not cause settlement). Long toe-bearing piles may experience excessive stress at the neutral plane. "Long" in this context means piles longer than about 100 pile diameters.

10. When conditions warrant and associated costs are acceptable, the negative skin friction can be substantially reduced by applying a thin coat of bitumen to the pile surface. The bitumen will reduce the maximum load in the pile (useful, if the structural strength otherwise would be inadequate) and lower the neutral plane(useful, if lowering the neutral plane to a less compressible layer would be desirable). However, the bitumen will also reduce the pile capacity, which is an undesirable effect. Moreover, while applying a bitumen coat to a pile is a simple, as well as an assured and proven engineering solution to the problem of downdrag and excessive dragload, it is costly and its implementation can be awkward and result in a prolonged pile driving and development of damaging tension in a concrete pile. Therefore, bitumen coating should not be introduced lightly. The selection of bitumen is not the problem. A thin coat (about 1 mm to 2 mm) of commercially available Asphalt Grade 85 - 105, heated and brushed onto the pile surface, will be sufficient for most cases. The main problem with bitumen coating, apart from high costs, is making sure it adheres to the pile surface (a primer is normally necessary) and that it is not scraped off in surficial coarse-grained soil layers.

Fig. 16 will demonstrate the design approach in three diagrams. The diagram to the left shows the distribution of negative skin friction and positive shaft resistance along a pile in a uniform soil. The unit shaft shear increases with the effective overburden stress and the change from negative to positive direction occurs linearly along a transition zone, a "neutral plane zone".

The middle diagram shows two curves. One indicates the ultimate resistance of the pile, R^u, which is the sum of fully mobilized toe resistance R_t^{ult} plus fully mobilized shaft resistance, R_s^{ult}. This curve would be established from a static loading test on an instrumented pile or from static analysis. The second curve shows the long-term load-transfer for the pile, indicating a partially mobilized toe resistance, the load due positive shaft resistance over the length between the pile toe and the neutral plane, the load over the length above negative skin friction to the pile head, and the dead load applied to the pile. Except for iside the transition zone, the shaft shear below and above the neutral plane is fully mobilized.

The diagram to the right presents the distribution of settlement for the soil and pile. At the neutral plane, the pile and the soil move equally and the pile settlement is the settlement at the neutral plane plus the compression of the pile above the neutral plane.

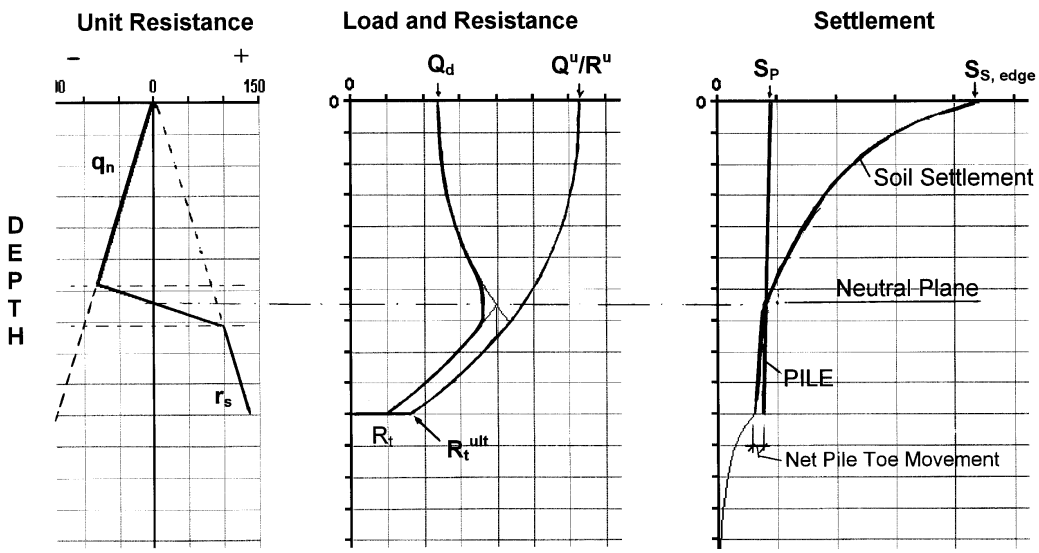


Fig. 16 Distribution of unit shaft shear and load-transfer curves

The two major unknowns indicated in Fig. 16 are the length of the neutral plane transition zone and the magnitude of mobilized toe resistance. If the soil movement near the neutral plane is small relative to the pile, the zone would be long and, conversely, if the settlement is large, the zone is short. Rephrased, the smaller the angle between the two intersecting curves, the longer the zone. The amount of toe resistance is governed by "net toe movement", which can be determined by means of analysis using appropriate "t-z functions".

For pile groups with fewer than about a dozen piles, the settlement of the pile group is suggested to be taken as the net toe movement plus the compression of the pile. For larger groups, it is suggested that the settlement be taken as the settlement for an equivalent footing placed at the neutral plane considering both the dead loads (the dragload must not be included) on the pile group and the changes to the effective stress due to other actions at the site. The soil reinforcing effect needs to be included, which means that the analysis can be made with the equivalent footing placed at the level of the pile toes. The t-z function and net movement analysis should not be combined with the equivalent footing analysis of settlement of settlement.

In summary, a pile foundation design is carried out in three "unified" steps, as follows:

Allowable Load and Design Load: The allowable load is a function of the bearing capacity with no reduction for dragload. The design load to be checked against the allowable load includes dead and live load, but no dragload.

Maximum Load and Structural Strength: The maximum load in the pile occurs at the neutral plane and is dead load plus dragload. Live load must not be included. The structural strength of the pile is what determines what maximum load to allow in the pile at the neutral plane.

Settlement of a Pile Group: The settlement of a pile group is a function of stress increases in the soil due to fills, embankments, and excavations, change of groundwater table, *and* the load on the pile group. Estimation of settlement requires knowledge of the location of the neutral plane and the soil settlement at the neutral plane. The settlement of a small group of piles is best analyzed in terms of t-z functions for the pile toe. A large group can be analyzed by means of an equivalent footing approach.

Example Project

The following will serve as an illustration to the author's "unified design approach". The project involves the driving of about 2,000 precast concrete piles for a power plant in Arecibo, Puerto Rico. The soils consultant was GMTS in Puerto Rico, Hector Lavergne, P. E., the Owner was Cambalache Partnership, Mark Bellamy, P. E., Project Manager, and the Contractor was Fuentes Concrete Pile Co. Inc., Jorge L. Fuentes, P.E.

The soil site is located in a former ocean bay that was infilled by alluvial soils washed down from the mountains. The soil profile consists in summary of about 6 ft of recent fill (serving as working platform) placed on about 25 ft of clay followed by about an equally thick layer of silty sand. At a depth of about 60 ft, the soil profile changes to alternating layers of silt and clay continuing to a depth of 100 ft to 110 ft, where again a layer of sand exists. The soil borings show fine to medium sand with occasional shell fragments alternating with a few feet thick zones of coarse sand, gravel, or silty clay. Testing on samples of the clay at this depth showed it to be a slightly overconsolidated, OCR given as 1.1, with a water content varying from 20 % through 30 %, organic content of 2 % to 5 %, and a modulus number (m) of about 15 or better and reloading modulus, (m_r) of about 100. The sands below 100 ft showed N-values ranging between 20 bl/ft to 30 b/ft. Below a depth of 140 ft, the soil again consists of silt followed by a thick layer of dense sand at 170 ft.

Although the stress from the structures was no more than 0.7 ksf, with the slab providing 0.3 ksf of this stress, because the area was expected to settle due to the fill, the foundations had to be placed on piles. The piles chosen were the 12-inch Fuentes spliced precast concrete pile to be driven with an ICE 520 double-acting diesel hammer. The pile design load was 75 kips for dead load plus 150 kips for live load. The allowable load, therefore, had to be at least 225 kips, which coupled with a minimum factor of safety of 2.0, gave a required axial capacity of 450 kips. The structural limit for axial load applicable to the load at the neutral plane was also 450 kips.

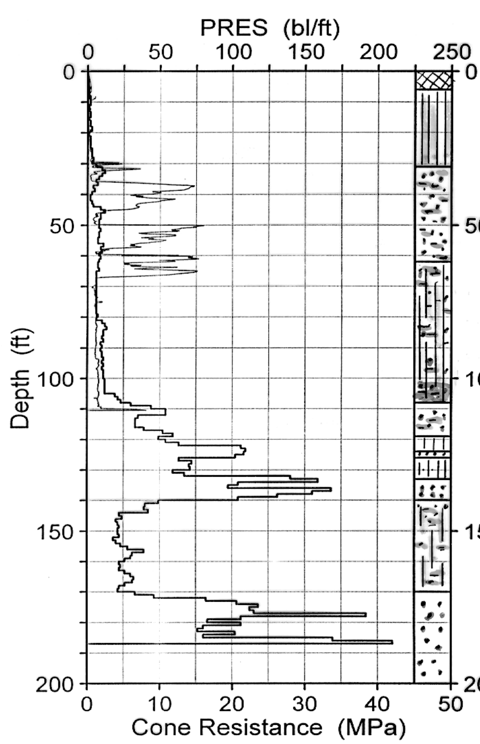


Fig. 17 Driving diagram of test pile to 197 ft and cone resistance diagram to 110 ft

Prior to the author's involvement and GMTS's soil exploration, a pile testing programme had been carried out at the site. The testing consisted of driving a series of test piles until they reached a penetration resistance of about 200 bl/ft, which was attained at depths below the fill of about 160 ft to 190 ft. The piles were then subjected to a static loading test to twice the design load. Fig. 17 shows a driving diagram from one of these piles and, also, a diagram from a piezocone test, CPTU, stopping in the surface of the sand layer at 110 ft.

The long piles raised concerns for costs and length of time to complete the construction. The static tests showed that the pile capacity was clearly greater than the twice design load, but gave little other information. The testing programme included dynamic testing during initial driving and restriking, but other than providing useful information on driving stresses, hammer performance, the analysis (CAPWAP) of blow records could only indicate the obvious, that is, that at end-of-initial-driving the soil resistance was larger than the hammer could mobilize and that soil resistance increased due to set-up with time after initial driving. However, the magnitude of the set-up could not be determined.

The initial programme had only been directed toward determining the pile capacity. Once it became clear that the piles could be driven through the sand layer at into the silty layers below the depth of 140 ft, capacity was obviously not the problem, because such long piles would have more than adequate capacity. Also shorter piles installed to some toe bearing in the sand layer at about 110 ft would have adequate capacity, and probably no more dragload than they could resist. In contrast, piles installed to the dense sand layers below 170 ft would probably be so stiff in the response to the movements above this depth that the maximum load in the pile would exceed the structural strength. Therefore, the problem to address at this site was the load-transfer of piles installed into the sand layer and the settlement for the structure placed on these piles.

A testing programme was introduced consisting of driving piles to three different shallow depths and test them to determine the load-transfer. In addition, the soil exploration was supplemented with two new boreholes to recover Shelby samples from the clay layers in the sand below 100 ft depth and testing these in the laboratory.

Six piles were driven in pairs to depths of 83 ft, 93 ft, and 103 ft (Piles T1 and T2, T3 and T4, and T5 and T6, respectively). The testing programme included dynamic monitoring of the initial driving of the piles, static loading tests after a three week for set-up to develop, and dynamic monitoring during restriking the piles.

Figs. 18 and 19 present wave traces from the End-of-Initial-Driving (EOID) and Beginning-of-Restrike (BOR) recorded for Pile T3. The traces show that there was considerable soil set-up at the site. At EOID, the penetration resistance (PRES) was 10 blows/foot. At BOR, the equivalent PRES was 25 blows/inch.

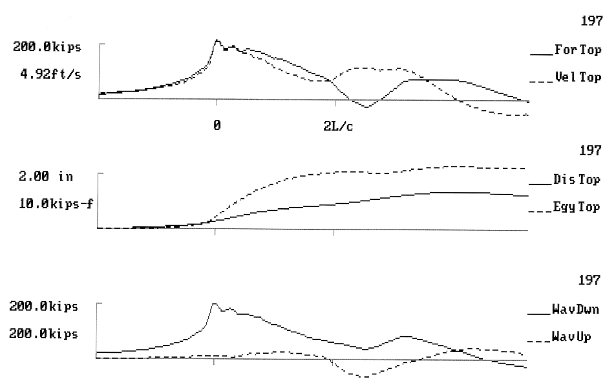


Fig. 18 Wave traces from Pile T3 at EOID

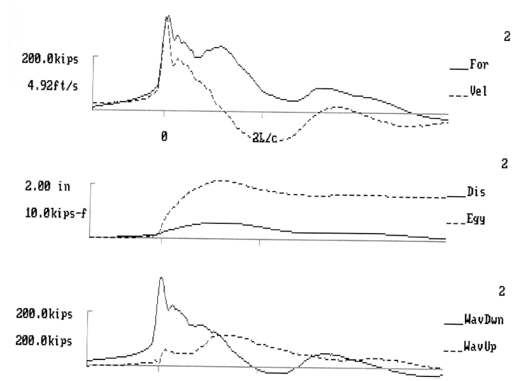


Fig. 19 Wave traces from Pile T3 at BOR

The dynamic blow records were analyzed by means of CAPWAP signal-matching to determine capacity and distribution of resistance along the pile, and to produce a simulated static load-movement curve. One of each of the three pairs of piles was subjected to a static loading test. Figs. 20 through 22 present the results of the static loading tests and the CAPWAP simulations. Notice that the CAPWAP analysis of the blow record from Pile T6 indicates that the hammer did not fully mobilize the pile capacity, notably the toe capacity.

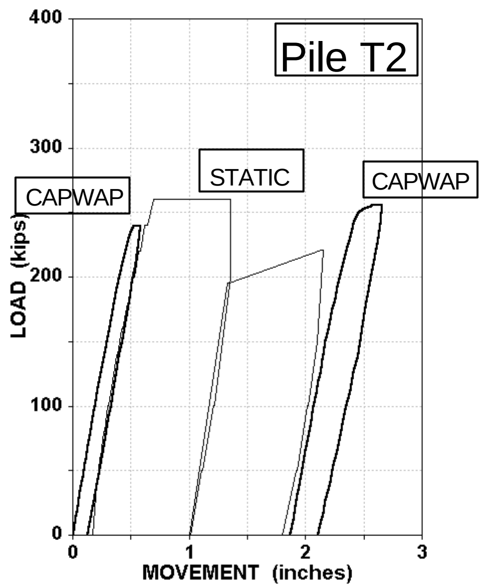


Fig. 20 Test Pile T2. Static tests and CAPWAP simulated static test

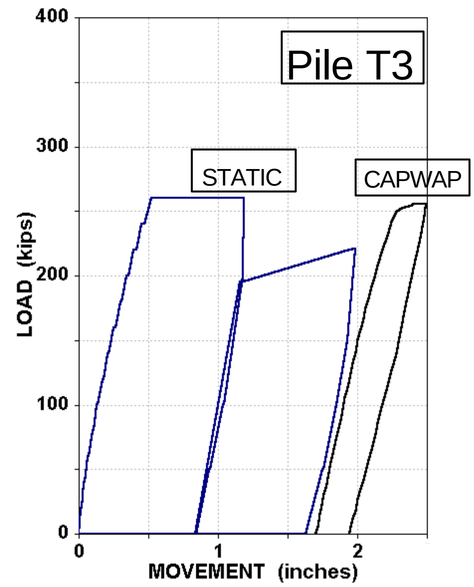


Fig. 21 Test Pile T3. Static tests and CAPWAP simulated static test

The static tests demonstrated that the pile load-transfer could be correlated to effective stress analysis and, also, served to verify that the CAPWAP signal-matching analysis on restrike blows was a reliable tool for determining the pile capacity at the site.

The results showed that the piles could be designed to be driven to moderate seating into the sand layer starting at 105 ft. After set-up, the pile would have more than adequate capacity and the maximum load would be smaller than the limit governed by structural strength. Settlement analysis indicated that the settlements would be no more than about 0.5 inch.

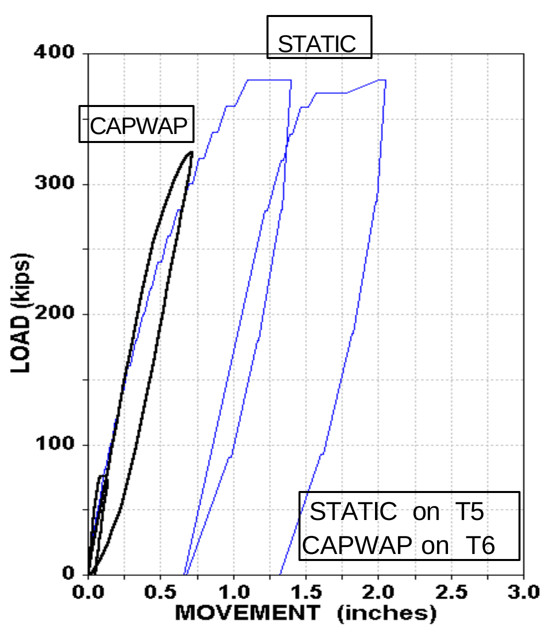


Fig. 22 Test Pile T5 Static tests and Test Pile T6 CAPWAP simulated static test

The analysis results are compiled in Fig. 23 and show the long-term load distribution in a pile as estimated from the data. In Fig. 24 the a similar analysis is used to show what would have been the distribution had the project gone ahead with the deeper installation as originally intended.

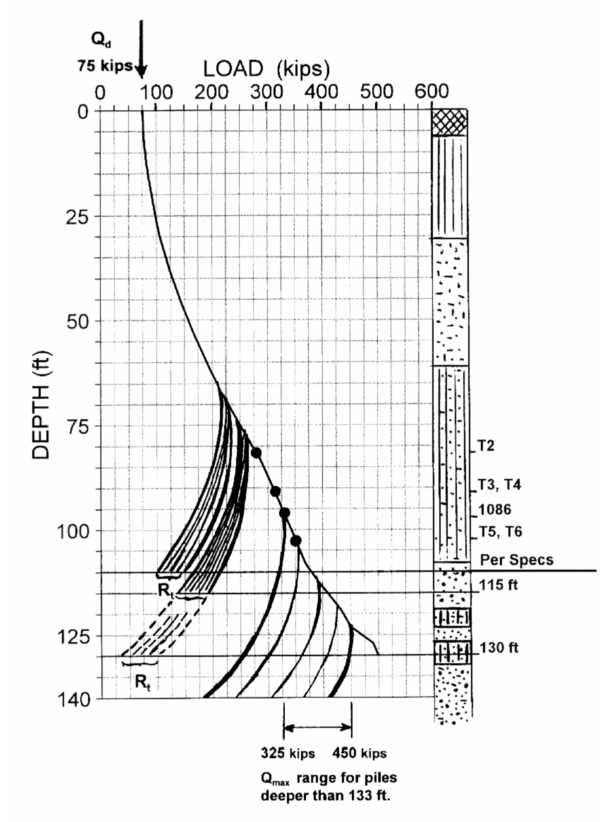


Fig. 23 Long-term Load Distribution
per Final Design

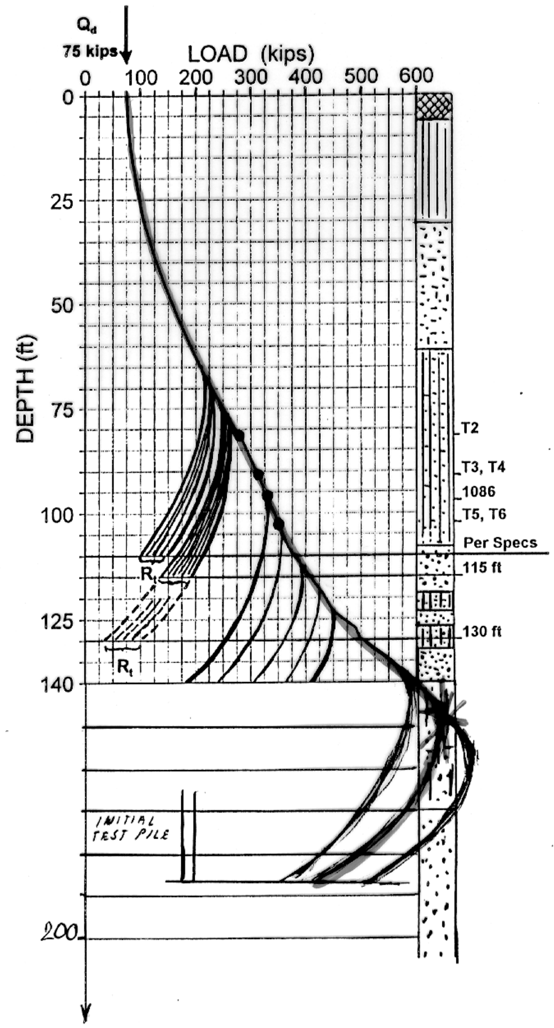


Fig. 24 Long-term Load Distribution
per Initial Design

The foregoing is only a brief account of the project and leaves out much detail. The installation of the construction piles was carefully monitored to verify that the piling proceeded as postulated in the design. No more static tests were performed. However, the driving was checked by frequent dynamic testing at Initial Driving and Restrike. Because the driving was easy and fast, on occasions, the piles happened to be driven deeper than intended. Fig. 25 presents a compilation of the driving logs for a row of ten piles. Three piles of these piles were inadvertently driven too deep, but not to the point that they were considered unsuitable for use.

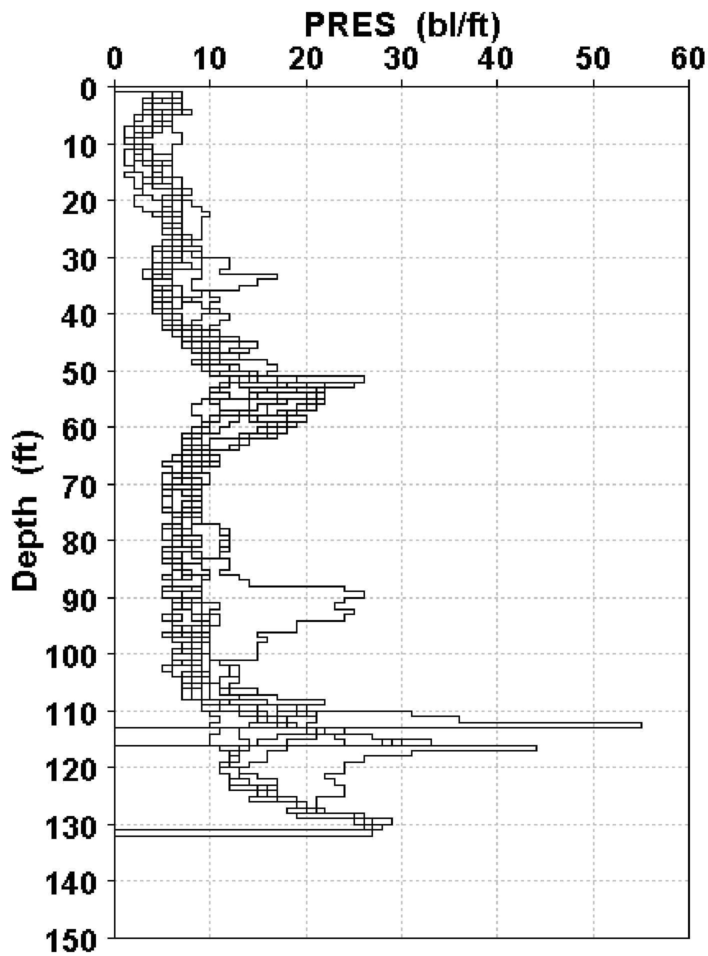


Fig. 25 Driving diagram (ten construction piles)

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