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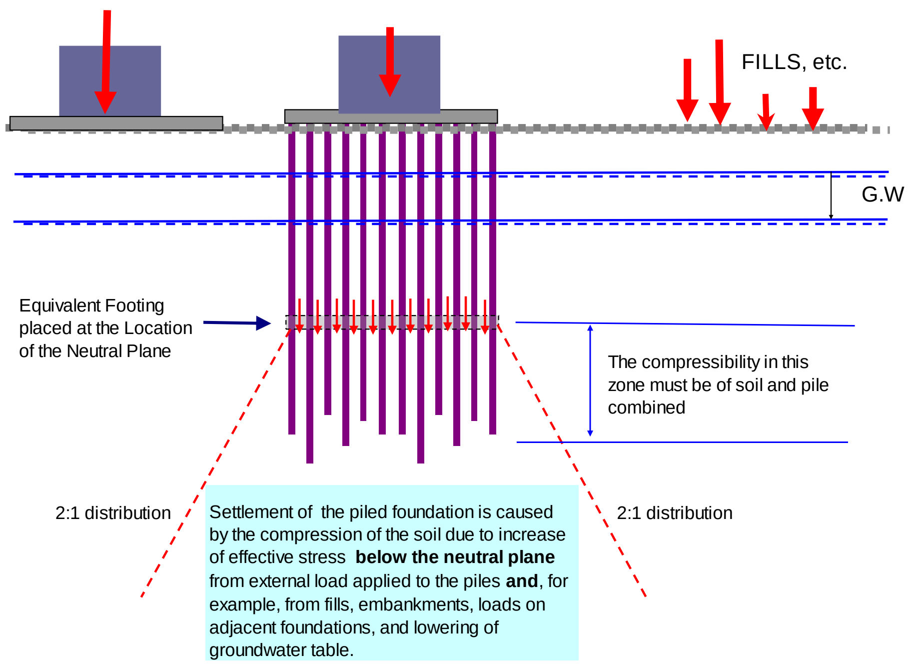

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UNIFIED DESIGN OF PILED FOUNDATIONS WITH EMPHASIS ON SETTLEMENT ANALYSIS

Bengt H. Fellenius[1]

ABSTRACT Design of a piled foundation rarely includes a settlement analysis and is usually limited to determining that the factor of safety on pile capacity is equal to an at least value. This approach is uneconomical and, sometimes, unsafe. Every design of a piled foundation should establish the resistance distribution along the pile, determine the location of the force equilibrium (the neutral plane), estimate the magnitude of drag force from accumulated negative skin friction at the neutral plane, evaluate the length of the zone where the shear forces change from negative to positive direction, establish the load-movement relation for the pile toe and the load distribution in the pile at the time that settlement becomes an issue for the design, and, finally, perform a settlement analysis. The settlement analysis of a piled foundation must distinguish between settlement due to movements caused by external load on the piles and settlement due to causes other than the load on the piles. A fundamental realization of such design approach is that pile toe capacity is a misconception. Each of the mentioned points is addressed in the paper, and a design approach for the design of piled foundations and piled rafts is presented. Examples and case histories are included showing the distribution of measured and calculated resistance distribution along the piles and settlement of soil and piles.

INTRODUCTION

The most common reason for placing foundations on piles as opposed to on spread footings, rafts, or other types, is to minimize foundation settlement. Yet, the design of a piled foundation rarely includes a settlement analysis. Of old, the common notion is that, if capacity is safe, nature takes care of the rest. This "design-by-faith" approach is frequently uneconomical and wasteful—neither is it always safe. In addition to determining capacity, settlement analysis must be a part of every design of a piled foundation. For piles bearing in rock or glacial till, this may merely be an assessment

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of the fact that no adverse settlement will occur. For other conditions, settlement assessment requires detailed analysis. Similar to the design of any type of foundation, a proper settlement analysis necessitates that soil profile and pore water regime are well established and that influence of fills, loads from other foundations, excavations, and changes in groundwater table are included in the calculations. For piled foundations, however, it is necessary to take into account additional factors, such as the distribution of pile shaft and toe resistances at long-term equilibrium between loads at the pile head, drag force at the location of the neutral plane due to accumulated negative skin friction, the length of the zone above and below the neutral plane within which the shear forces along the pile shaft change from negative direction to positive direction, the load-movement relation for the pile toe, and the axial force distribution in the pile. Moreover, a settlement analysis must distinguish between settlement due to movements caused by external loads from the supported structure and settlement due to causes other than the sustained load.

PILE CAPACITY AND RESIDUAL FORCE

Pile capacity is a basic aspect of the pile design and analysis. Capacity is the ultimate resistance of the pile, the load beyond which movement becomes excessive or progressive for little increase of load, as observed, for example, in a static loading test. The capacity is easy to determine in the case of a pile having no toe resistance and a shaft resistance with elastic-plastic response to loading, such as the typical pile load-movement curve presented in Figure A. The curve is determined in a simulation of a static loading test on a 300 mm diameter, 15 m long closed-toe steel pipe pile in uniform soil. The capacity value is obvious from the plunging behavior of the shaft-bearing pile, i.e., the continuous movement for no load increase. As indicated in Figure 1B, however, once toe resistance comes into play, the load-movement curve no longer demonstrates a plunging behavior.

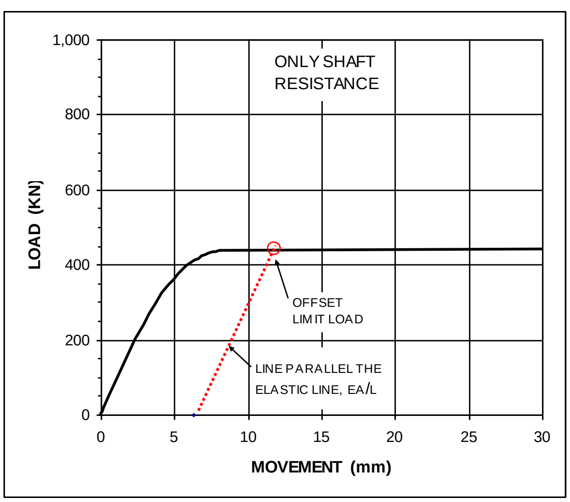


Fig. 1A Load-movement for a 100 % shaft-bearing pile

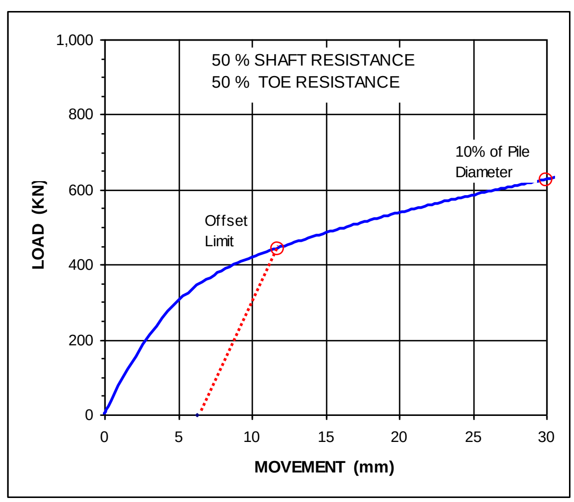


Fig. 1B Load-movement for a pile with equal shaft and toe resistances

The load-movement curve shown in Figure 1B is representative of the same pile when the resistance is assumed to be 50 % from shaft resistance and 50 % from toe resistance. The soil parameters for the calculations behind Figure 1B are chosen so as to have the Offset Limit equal to the capacity of the shaft-bearing pile (Figure 1A). The load-movement curve in Figure 1B does not show any tendency toward “plunging” or any obvious load value that could be considered to be the capacity of the pile. For such cases, the practice is either to simply consider the capacity to be the load that generated a movement equal to certain percentage of the pile diameter, or to select a value of pile capacity by a definition applied to the curvature of the load-movement. Several such definitions are in use, the most common in North America is the mentioned Davisson Offset Limit Load, which is the load at the intersection with the load-movement curve and a line parallel with the elastic line of the pile rising from the movement axis at a value equal to 4 mm plus the pile diameter divided by 120. The multitude of failure definitions is a consequence of the futility of forcing an ultimate resistance theory onto a situation where it does not apply. Obviously, there is more to determining pile capacity than selecting an arbitrarily defined point on a curve.

The load-movement response of a pile is the combination of the results of three developments. First, the shaft resistance, which in most cases does develop an ultimate (plastic) resistance and a failure mode. Second, the shortening of the pile, which is a more or less linear response to the applied load. Third, the toe response, which does not display an ultimate resistance. The latter statement can be understood on realizing that a pile toe is but a footing supporting a long column, and the load-movement behavior of a pile toe is similar to that of footings, as discussed next.

The concept of ultimate resistance was developed many years ago from observations of large scale footings in clay and model footings in sand. Loading tests on small-scale or large-scale footings on clay were performed at rates of loading such that pore pressures developed increasingly as the test progressed, causing the effective strength to reduce to the point that a failure resulted. This does not happen when the loading rate is so slow that excess pore pressures dissipate as fast as they develop. Moreover, with regard to the small-scale tests in sand, it was not realized at the time that the soil response of tests on small footings placed on the surface of a sand is always in a dilative mode: the sand expands, loses density, and loses strength as the test progresses (Altaee and Fellenius 1994). In contrast, no failure has been observed for buried footings, small or large.

For example, Ismael (1985) performed footing tests in fine compact sand on square footings with sides of 0.25 m, 0.50 m, 0.75 m, and 1.00 m at a depth of 1.0 m, 2.8 m above the groundwater table. The results in terms of measured stress versus movement as a percentage of the footing widths are shown in Figure 2A. Notice that the curves are gently curving with no break or other indication of failure despite the movements being as large as 10 % to 15 % of the footing side. Similar results were presented by Briaud and Gibbens (1994) for footings placed well above the groundwater table in a slightly preconsolidated, silty fine sand. The natural void ratio of the sand was 0.8. The footing sides were 1.0 m, 1.5 m, 2.0 m, and 3.0 m. Two of the footings were 3.0 m wide. The results of the test are presented in Figure 2B. It is noteworthy that no indication of failure is indicated despite the large movements.

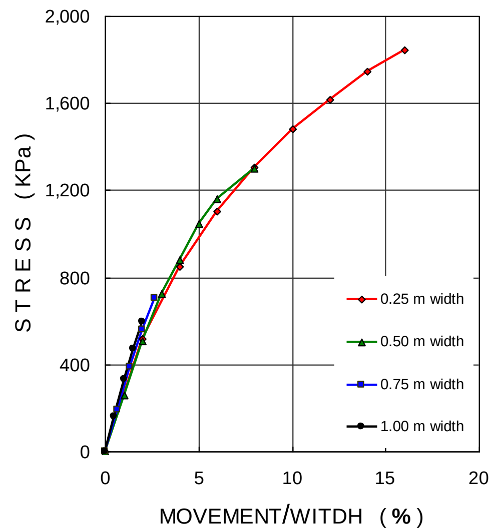


Fig. 2A Stress vs. movement for four footings (data from Ismael 1985)

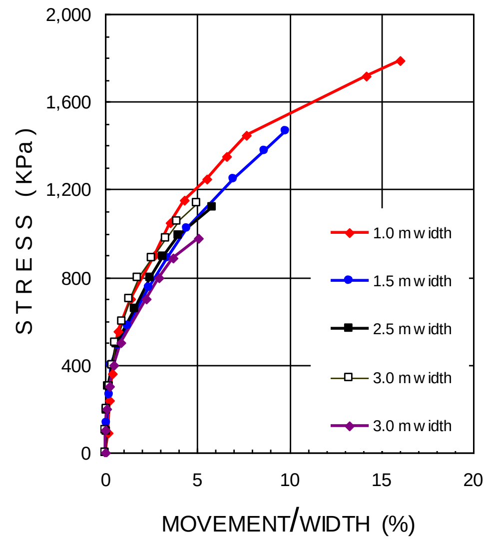


Fig. 2B Stress vs. movement for five footings (data from Briaud and Gibbens 1994)

The recently developed bidirectional test (Osterberg 1998) has enabled direct observations of the response of a pile toe to increasing load and shown that the virgin load-movement response of the pile toe is in the shape of a gentle curve gradually bending over and displaying no kinks or sudden changes of slope; very much similar to that of a footing. Plainly, bearing capacity as a concept does not apply to the response of a pile toe to load.

That the concept of pile bearing capacity is specious does not mean that the application of the concept of a bearing capacity of a pile would be wrong. The approach is well-established in engineering practice. However, the practice would do well to recognize the fallacies involved as demonstrated in the following.

Figure 3 presents two load-movement curves produced by simulation of a static loading test using identical pile and soil input. The only difference between the calculation of the two curves is that no residual force was assumed to be present in the pile represented by the lower curve, whereas for the upper curve a residual load amounting to one-third of the “ultimate” toe resistance was assumed. (The term “residual forced” refers to the force present in a pile—locked-in—immediately before the start of a static loading test). The Offset Limit line (“Davisson line”) has been added to show more clearly how much difference — in this case 20% — the presence of residual force can have on the interpretation of the results of a test.

To calculate the load-movement curves, the shaft and toe resistance responses were input expressed in “t-z” and “q z” curves as indicated in Figure 4. The q-z curve (used for the Figure 1B simulation) is in the shape of a gentle curve gradually bending over without reaching a peak—typical for a pile toe response. Both of the two t-z curves have a distinct peak, however. The “no-strain-softening” t-z curve used for the simulations in Figures 1A and 1B shows a shear resistance having a slight trend to strain hardening beyond the peak. The second t-z curve shows a strain-softening response beyond the peak, which load-movement behavior is typical for pile shaft resistance in most soils. The results of a simulation employing this t-z curve are presented in Figure 5. As in Figure 3, the upper load-movement curve in Figure 5 includes the effect of residual force and the difference between the two curves is similar to those of Figures 1 and 3 (to facilitate the comparison, the curves shown in Fig. 3 are indicated also in Figure 5). The curves show that the gradual increase of toe resistance is compensated by the simultaneous gradual decrease of shaft resistance. Notice, an eye-balling of the curves would now suggest that the Offset Limit would be a reasonable "failure load" to interpret from test results.

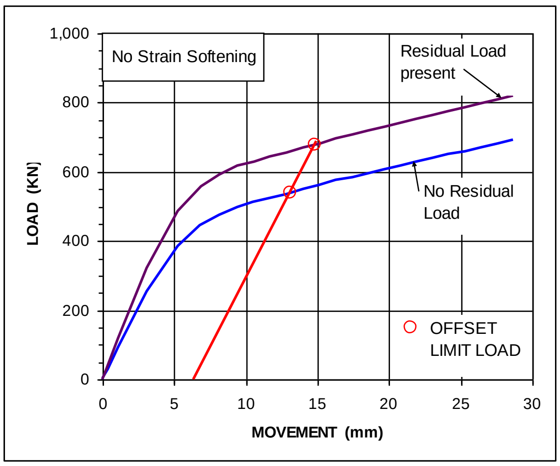


Fig. 3 Load-movement curves for pile unaffected and affected by presence of residual force

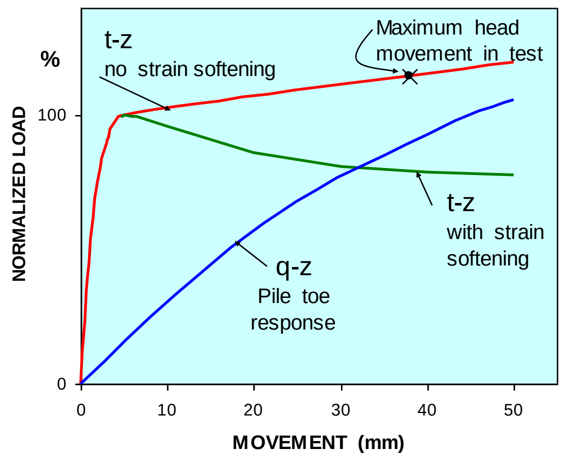


Fig. 4 Shaft and toe response t-z and q-z curves (normalized load)

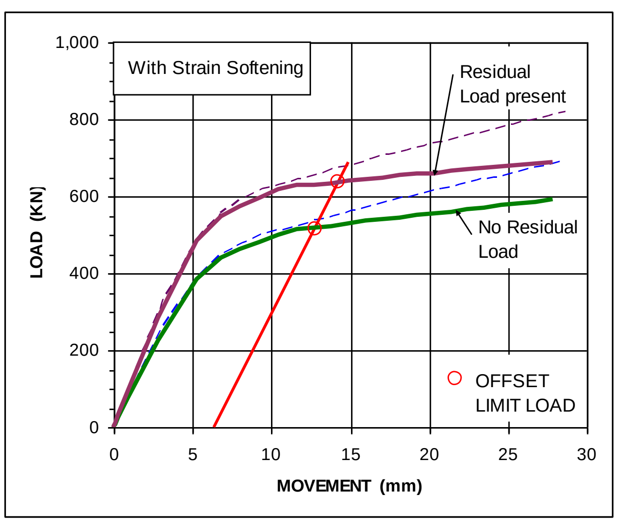


Fig. 5 Load-movement curves for using the strain-softening t-z curve, unaffected and affected by residual force

Despite the fact that the existence of residual force has been observed and reported several times (e.g., Hunter and Davisson 1969; Gregersen et al. 1973; Fellenius and Samson 1976; Holloway et al. 1978), many are under the fallacious impression that its effect is marginal and, anyway, limited to piles in clay. Thus, they are led to think that residual force can be neglected in the analysis of the results from loading tests on instrumented piles. The effect is far from marginal, however, nor is it limited to piles in clay. Figures 6 and 7 show results of a static loading test on an instrumented 280 mm diameter precast concrete pile driven 16 m into sand.

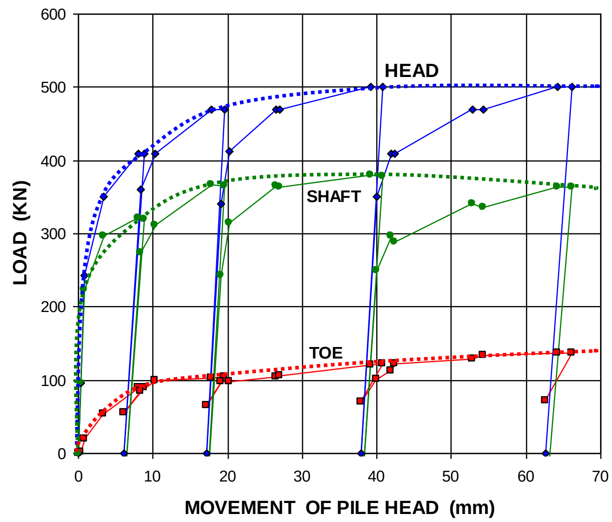


Fig. 6 Load-movements for head, toe, and shaft of an instrumented pile in sand (data from Gregersen et al. 1973)

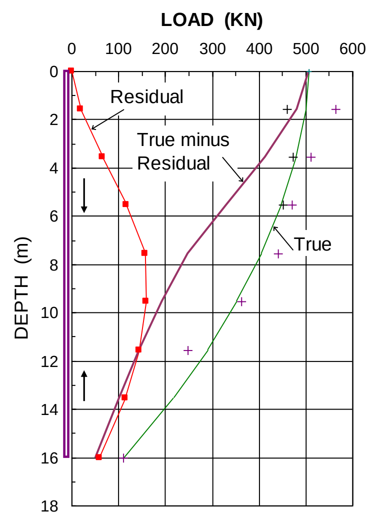


Fig. 7 Measured distributions of residual load and true load (data from Gregersen et al. 1973)

Note the gradual, almost linear increase of the toe load and the reduction in shaft resistance for pile head movements beyond 40 mm shown in Figure 6. The initial stiffer shape of the toe curve is an effect of the driving having densified the sand immediately below the pile toe. The increase of toe resistance beyond the 10 mm pile head movement is about equal to the simultaneous decrease of shaft resistance and results in the appearance of plunging failure for the load-movement curve of the pile head at a 500 kN maximum load. Note also that each time the pile head was unloaded, the load remaining increased, showing that the preceding load cycle had increased the residual force in the pile.

The pile was instrumented at several levels and Gregersen et al., (1973) measured the load distribution in the pile immediately before and during the static loading test. Figure 7 presents the distribution measured immediately before the test, i.e., the "residual force", and the load distribution at the 500 kN maximum load, i.e., the "true load". The difference between the two curves, (the curve marked "True minus Residual") is the measured imposed increase of force in the pile at the maximum test load. This is what would have been mistakenly considered the "true" force distribution the presence of residual force had not been considered, and all gage readings at the start of the test had been assigned to be "zero readings", that is, the readings for zero load in the pile.

The residual load in the pile can be explained as partially introduced by the driving of the pile and partially be due to recovery (re-consolidation) of the soil from the disturbance caused by the pile driving. Note that residual load for piles in sand is not restricted to driven piles, which is illustrated in Figure 8, presenting the load distribution in a 0.9 m diameter, instrumented bored pile in sand.

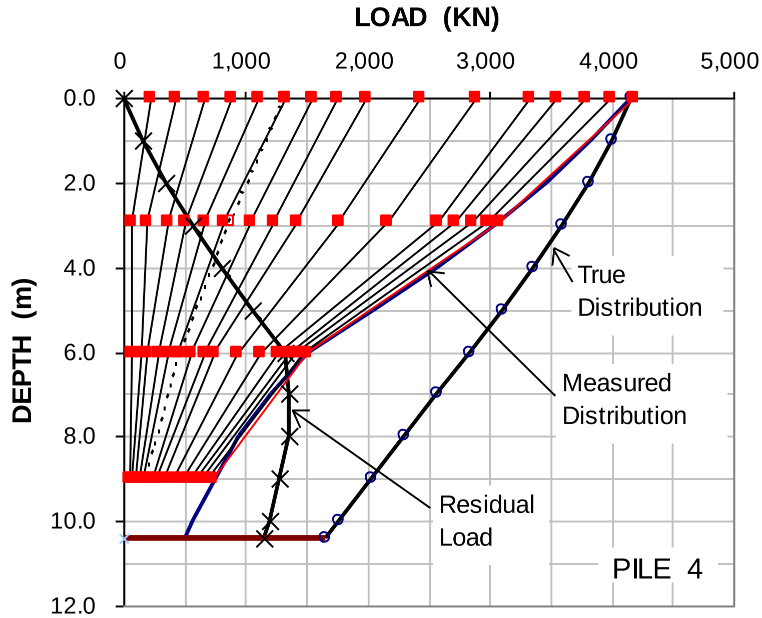


Fig. 8 Measured load increase from the start of a static loading test in sand with evaluated distributions of residual load and true load. (data from Baker et al. 1993, re-analyzed by Fellenius, 2001)

Measurements of residual force is mostly obtained from observation of force distribution for piles in clay with the objective of studying the development of drag force. Usually no static loading test is performed. However, the term "drag force" is just the term for "residual force" when no static loading test is performed. The mechanics are identical.

For example, Fellenius and Broms (1969) and Fellenius (1972) measured the force distribution in two 300 mm diameter, 53 m long, instrumented piles driven through about 40 m of clay and into sand. The area was virgin, untouched by construction since it rose from the sea after the end of the Ice Age. The load distribution was measured immediately and during a long time following the driving. The measured distributions are presented in Figure 9 and show that, immediately after driving, the force in the pile was about equal to the own weight of the piles themselves. The dissipation of the pore pressures induced in the driving over the next 154 days resulted in the build-up of load in the pile. Somewhat surprisingly, the build-up of force continued also after the induced pore pressures had dissipated. Figure 9 includes also the distribution measured at 496 days after the end-of-driving, 342 days later. The continued force increase is considered due to a small regional settlement of about a millimetre per year coinciding with the isostatic land heave of about the same magnitude. Had a static loading test been performed at any one of these times after driving, the measured drag forces would have been the residual force in the piles.

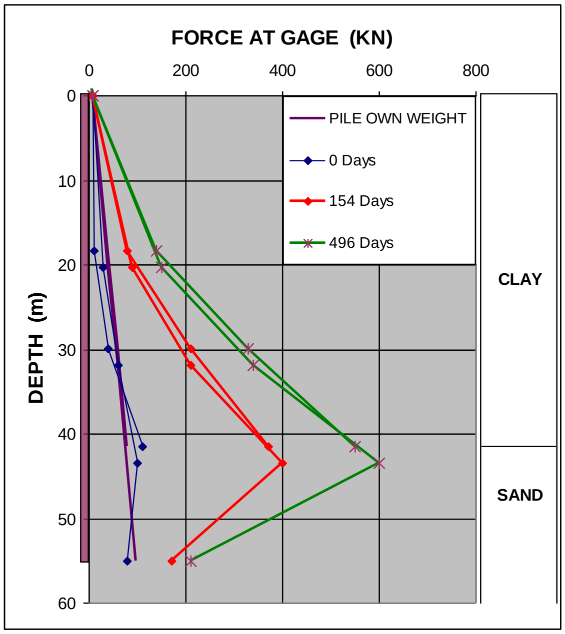


Fig. 9 Load distributions measured in two piles immediately, 154 days, and 496 days after the end of driving. (Data from Fellenius and Broms 1969; Fellenius 1972)

An additional example is presented in Figure 10, which shows results of a static loading test on a driven, 45 m long, 406 mm diameter, instrumented steel pipe pile in soft clay (Fellenius et al. 2004). The test was performed 46 days after the pile was installed when the induced pore pressures had dissipated. Note that if the residual force had not been considered in the evaluation of the test data, that is, if the gages had been "zeroed" at the start of the test, the forces measured during the tests would have been thought to be representative for the true load distribution. Then, the evaluation would have mistakenly concluded that there was no shaft resistance along the lower 12 m length of the pile.

Already a small movement will be sufficient to develop shear forces along a pile. Such movement is the result of a large number of influences associated with the driving of piles, drilling and grouting of bored piles, curing of the concrete grout, reconsolidation of the soil around the pile, etc., as well as effect of environmental events at the site, such as ongoing settlement. It is not the purpose of this paper to dwell on what causes the development of residual load, only to indicate that if residual load is disregarded in the analysis of the results of a static loading test, the results of the analysis will be in error.

Residual force is built up by accumulation of negative direction shear forces along the upper part of the pile that is in equilibrium with accumulation of positive direction shear forces along the lower part. The length along the pile (the shear transition zone) where the shear direction changes from negative to positive direction can be short or long.

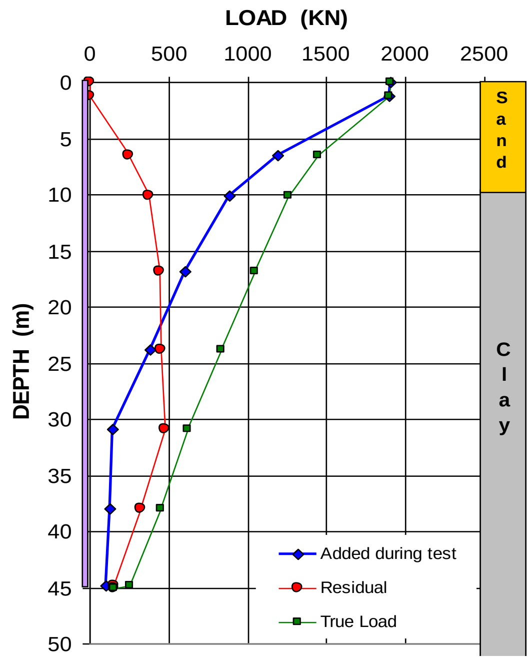
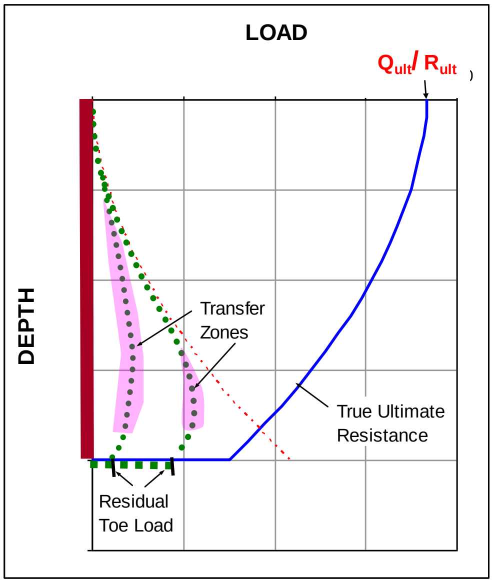


Fig. 10 Load distributions measured in a static loading test on a 45 m long pile in clay. (Fellenius et al. 2004)

Shear resistance along a pile is governed by effective stress and is approximately proportional to the overburden effective stress. Therefore, the ultimate shaft resistance distribution in a homogenous, uniform soil will be in the shape of a curve similar to the ultimate resistance curve shown in Figure 11A, which shape can be determined in a test performed on an instrumented pile.

The toe resistance portion of the "ultimate" resistance, shown level with the pile toe in Fig. 11A, is the resistance mobilized in the static loading test, typically at a toe movement of about 10 mm. As made clear in the foregoing, before the start of the test, the pile will have become subjected to a residual force—sometimes a significant amount, sometimes negligible. To illustrate the statement, the figure indicates two distributions of residual force. The first curve represents the case of small relative soil movements, where full residual shaft shear is only mobilized near the ground surface (negative direction) and near the pile toe (positive direction). The shear forces are not fully mobilized along the middle portion of the pile, the zone of shear direction change— the "transfer zone". The second curve is typical of where larger relative soil movement have caused the shear forces to become fully mobilized over a longer length of the pile, leaving a shorter transfer zone.

If the gages in the pile had been "zeroed" immediately before the start of the static loading test, and only the test-applied loads considered in the final reporting of the distribution, the so-determined distribution may be grossly in error. Figure 11B shows the "true distribution" curve, which combines the residual force and test applied loads, and the "false distribution", which appears when the residual force is neglected. Published case histories on results of loading tests on instrumented piles have frequently neglected to include residual force, which has given rise to fallacies such as the "critical depth" (Fellenius and Altaee 1995).



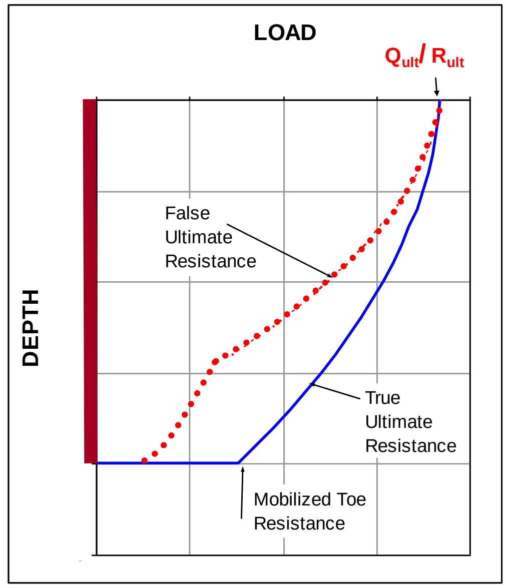


Fig. 11A Distributions of ultimate resistance and residual load

Fig. 11B True and false ultimate resistance distributions

NEGATIVE SKIN FRICTION, DRAG FORCE, AND DOWNDRAG

Several important papers have published presenting measurements of load distribution in instrumented piles with emphasis on drag force, e.g., Bjerrum et al 1965 and 1969; Walker et al. 1973; Endo et al. 1969; Fellenius and Broms 1969; Fellenius 1972; Clemente 1979 and 1981; Bozozuk 1981, Leung et al. 1991).

The case history paper by Endo et al. (1969) is a comprehensive field study of drag force and downdrag on piles and demonstrates the interaction between the forces in the pile, the settlement, and the pile toe penetration. The paper presents the results of measurements on instrumented, 610 mm diameter, 43 m long, single, driven steel piles, as well as settlements of the piles and surrounding soil during a period of almost two years (672 days). The soil profile at the site consisted of thick alluvium over a buried aquifer: a 9 m thick layer of silty sand followed by silt to a depth of about 25 m underlain by alternating layers of silt and sandy silt to a depth of 41 m followed by sand. Two piles were driven closed-toe and one was driven open-toe. The end-of-driving penetration resistance was light, about 20 mm for the last blow.

The groundwater table was located about 1 m below the ground surface. The pore water pressure at the site was affected by pumping in the lower silt layer to obtain water for an industrial plant, which created a downward gradient at the site. The difference in terms of head of water between the groundwater table and the sand layer at the depth of 40 m was about 30 m. The ensuing consolidation of the soils caused the soil to settle and hang on the piles, creating drag force and downdrag.

Figure 12A presents a compilation of the load distributions measured in three piles 672 days after the driving, denoted Piles oE43, cE43, and cB43. All three piles developed a neutral plane shortly below 30 m depth. The force distributions in the two closed-toe piles are very similar. For these two, the drag force is about 3,000 kN, in equilibrium with the sum of positive shaft resistance of about 1,500 kN and toe resistance of about 1,500 kN. Had a static loading test been included in the study, the shaft resistance of the full length of the piles would have been about 4,500 kN.

The load distributions are particularly interesting when related to the settlement distributions measured both in the soil and for the piles. Figure 12B presents settlement measurements taken 124 days, 490 days, and 672 days from Pile cE43, after the end of driving. Note that the point where the relative movement between the pile and the soil is zero (i.e., where the soil and the pile settlement curves intersect) is approximately level with the depth of the force equilibrium — the neutral plane. Figure 12B shows that the settlement of the ground surface 672 days after the end of driving was 120 mm. At this time, the settlement (of soil and pile) at the neutral plane was about 40 mm, and the settlement of the pile head was 53 mm, consisting of the settlement at the neutral plane plus about 13 mm of pile shortening between the pile head and the force equilibrium plane due to the drag force.

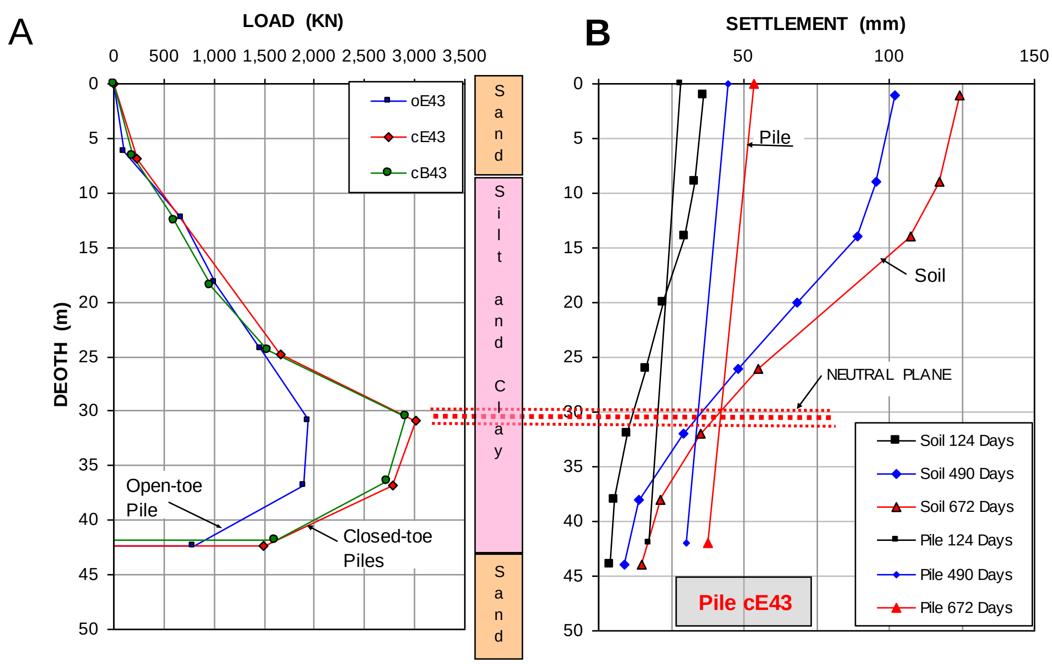


Fig. 12 Load distributions in three pipe piles 672 days and settlement distributions at 124 days, 490 days, and 672 days after driving (data from Endo et al. 1969).

During the 672 days of measurements, the relative difference between the settlement of the soil at the pile toe and the pile toe increased slightly, that is, the net pile toe penetration into the sand increased. The records also show that the pile toe load increased. In Figure 13, the measured pile toe forces have been plotted versus net pile toe penetration, as measured on the 124 day, 490 day and 672 day after the driving of the piles. The first measurements taken at the end-of-driving, indicate that the pile toe was subjected to an initial residual toe force corresponding to an initial toe movement of approximately 8 mm.

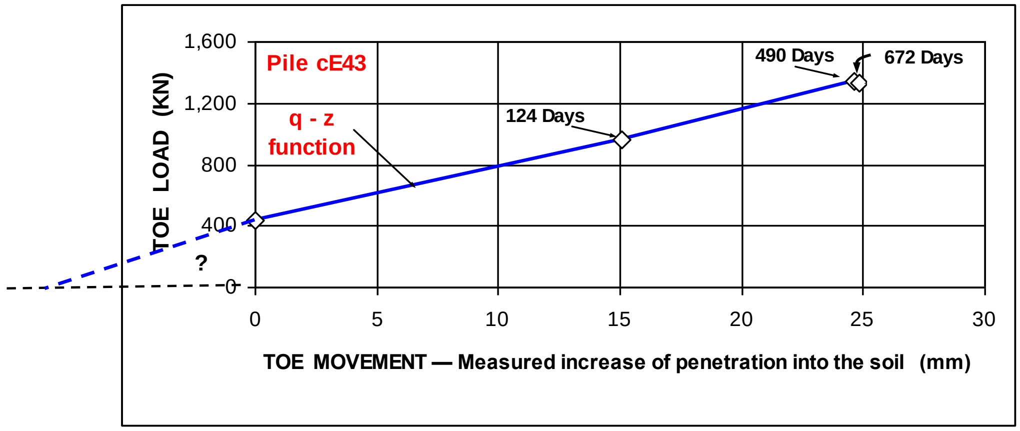


Fig. 13 Pile toe force-movement — the q-z function. (data from Endo et al. 1969)

The load-movement curve indicates a practically linear relationship between the toe-force and the toe penetration. Obviously, had the settlement been larger, the pile net penetration would have been larger, too, which would have mobilized a larger pile toe resistance, which, in turn, would have lowered the location of the force equilibrium.

Collectively, the case records established that

1. Shaft shear (negative skin friction as well as shaft resistance) is governed by effective stresses and requires very small soil movement relative the pile surface to become mobilized. Indeed, Bjerrum et al. (1969) reported that about the same magnitude drag force was measured for a few millimetre of settlement at the ground surface as for 2 m of settlement.
2. For high capacity toe-bearing piles, large drag forces were measured and, for these piles, the observed pile settlement consisted mainly of pile shortening for the loads.
3. A pile subjected to drag force has a load distribution consisting of negative skin friction accumulating in the upper portion of the pile in equilibrium with positive shaft resistance along the lower portion of the pile plus toe resistance. The zone where the shear forces transfer from negative to positive direction can be short or long depending on the magnitude of the soil movements and the relative stiffness of the pile and the soil. The picture is the same as that for the distribution of residual force shown in Figure 11A. Indeed, had Endo et al. (1969) polished off their superb study with a static loading test, the drag force would have acted as the residual force for the test.
4. The location where the downward acting forces are equal to the upward acting forces is where there is no movement between the pile and the soil. The location is called “force equilibrium plane” or “neutral plane”. At this location, the pile and the soil settle equally (“settlement equilibrium plane”), which is a very important insight for pile group design.

5. If appreciable soil settlement occurs at the neutral plane, the pile(s) will be subjected to downdrag, an undesirable condition for most piled foundations.
6. The location of the neutral plane is entirely a function of the conditions for the force equilibrium and is not a function of the magnitude of settlement in any other regard than the force equilibrium is a function of the pile toe penetration into the soils at the pile toe, which in turn is governed by the magnitude of settlement at the neutral plane.
7. Where soil settlements are small, the transition zone is long and where the soil settlements are large, the transition zone is short. All other things equal, the drag force is larger in the second case.
8. Live loads (transient loads) will reduce or eliminate the drag force.
9. Negative skin friction will develop whether or not an external load is applied to the pile head. The external dead load and the drag force will combine and the maximum load in the pile will occur at the neutral plane.
10. The larger the pile toe resistance, the lower neutral plane.
11. A thin coat of bitumen will drastically reduce the shear force between the pile surface and the soil and reduce the negative skin friction (and reduce the positive shaft resistance and pile bearing capacity).

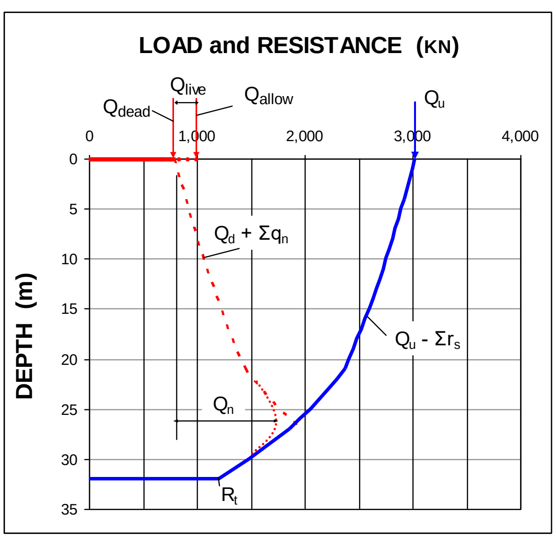
By using the term “drag load” in lieu of "drag force", some are led to believe the drag force to be just another load similar to the loads applied from the structure supported on the piles. However, the drag force is only of concern for the structural strength of a pile. In contrast to the external loads (the loads from the supported structure), the drag force is of no consequence for the bearing capacity or settlement of the pile or pile group. Simply, the drag force is no more a negative aspect for a pile than the prestress force is for a prestressed concrete pile. Indeed, a pile subjected to considerable drag force is stiffer than a pile that is not subjected to much drag force and will display smaller deformation for variations of the load applied to the pile head. Downdrag, on the other hand, is an important settlement problem that has to be carefully addressed in a design. The author has termed the design approach “The unified design of piled foundations for capacity, settlement, drag force, and downdrag” (Fellenius 1984; 1989).

THE UNIFIED DESIGN

Design of a piled foundation must consider three major aspects: capacity, drag force, and settlement, as recognized in the Canadian Foundation Engineering Manual (CFEM 1992), and other authoritative texts, such as the CSA and Ontario Highway Bridge Design Code (OHBDC 1991), the NCHRP-FHWA Report 343 (Barker et al. 1991), and the ASCE and US Army Corps of Engineers Technical Engineering Design Guides No. 1 and 7 (ASCE 1993 1994). (Unfortunately, poor use of terminology in the latter two texts frequently make the therein suggested procedures unclear). Most other guides, text books, codes, and standards limit the recommendations to the capacity aspect, causing many practicing engineers to make the usually safe, on occasions unsafe, and almost always overly costly decision to leave out the other two aspects from the design. Some designers, who do become aware that settlement will occur at a site, and, therefore, conclude that drag force will act on the pile, deal with this by adding the drag force to the external loads (the loads from the structure) or by reducing the pile capacity with the amount of the drag force before dividing the capacity with the factor of safety to obtain the allowable load. While the decision may have the positive effect of lowering the depth of the neutral plane and, therefore, reducing the calculated downdrag, it is often not a very cost effective method, and it does not correctly consider the factors involved. Sometimes, a designer decides to reduce the negative skin friction by means of a bitumen coat without realizing that this in equal measure also reduces the capacity of the pile.

Pile capacity is the sum of the positive shaft resistance along the <u>entire</u> length of the pile plus the toe resistance for a certain toe movement. As indicated in the foregoing, it is usually not recognized that the pile toe does not exhibit an ultimate resistance but is a function of the pile toe q-z response. However, the engineering practice has for a long time been served well by choosing a pile capacity and establishing an allowable load by dividing that capacity with a factor of safety. Capacity can be determined from empirical calculation methods employing data obtained by means of full-scale field tests, such as static loading tests or impact tests, or by using in-situ tests, such as cone penetrometer tests and standard penetration tests in combination with well-calibrated analytical procedures, such as effective stress analysis (beta-method) or, but less reliably, stress-independent analysis (alpha method). Practice is to employ a factor of safety on the results of the field test of about 2.0 and between 2.5 and 3.0 on the in-situ tests. *Nota bene*, all test results must be combined with a static analysis and correlated to the overall site conditions and their variations, or the mentioned factors of safety may be too low.

The approach to determining the allowable load, consisting of dead load and live load, is illustrated in Figure 14 for a pile with a capacity of 3,000 kN coupled with a factor of safety of 3.0, which determines the allowable load to 1,000 kN of which 800 kN is dead load and 200 kN is live load.



Q_d = Dead Load
Q_l = Live Load
Q_q = Allowable load = $Q_d + Q_l$

Q_u = Ultimate resistance = R_u

q_n = Unit negative skin friction
Q_n = Accumulated q_n

r_s = Unit positive shaft resistance
R_s = Total shaft resistance

R_t = Toe resistance

Fig. 14 Illustrative example of the approach to allowable load, dead load and live load, and drag force

The location of the neutral plane is indicated in the figure as the load distribution starting from the dead load and increasing with shaft shear in the negative direction until intersection with the load distribution starting from the ultimate resistance (pile capacity) and reducing with shaft shear in the positive direction. In this analysis, the live load must not be included—drag force and live load cannot coexist.

Note, in determining the allowable load from the pile capacity, drag force must not be included. Drag force is to be combined with the dead load and the sum be considered only when evaluating the structural strength of the pile (at the neutral plane).

Structural Strength. As indicated in Figure 14, the drag force for the example is determined to 900 kN, which means that the maximum force in the pile, consisting of dead load and drag force, becomes 1,700 kN. The axial structural strength of the pile must be such that this force can be resisted. However, piles are often installed so that the structurally allowable load at the pile cap will be about equal to the geotechnically allowable load (capacity divided by the factor of safety). Unless the live load on the pile is an unusually large portion of the allowable load, this means that the load at the neutral plane (dead load plus drag force) will be larger than the load allowed structurally at the pile cap. However, the structurally allowable load at the pile cap has to include considerations off-location placement, bending and lateral shear, which aspects do not apply to the structurally allowable load at the neutral plane, and the two should not be the same. The axial force at the neutral plane must be allowed to be larger than that at the pile cap. The author has applied the condition that at the neutral plane, usually, a strain of at least 0.001 can be accepted. The strain compatibility requirement applies in particular to composite piles, e.g., a concreted pipe pile or a reinforced concrete pile. The axial load limit cannot be determined from “allowable stress” on concrete and on steel, but must be based on a strain limit.

Because the transition from negative to positive direction of shaft shear requires some length, the maximum force in the pile is not the force at the curve intersection (calculated for fully mobilized negative skin friction and positive shaft resistance), but a smaller force, which magnitude is governed by the length of the transition zone (see below), which in turn is determined by the magnitude of the relative movement between the pile and the soil. In most cases, it can be assumed that the change starts and is finished where the relative movements in the negative and positive directions are both about 2 mm. In soils with high organic content, the full mobilization of the shear forces may require a larger relative movement.

Settlement Analysis. When evaluating the settlement of a piled foundation, the location of the neutral plane governs the results of the analysis. Whatever the magnitude of the settlement at the neutral plane, the settlement of the pile head is that value plus the shortening of the pile for the loads (dead load plus drag force) above the neutral plane. However, the location of the neutral plane is not constant. As the pile is forced down, the net penetration of the pile toe into the soil increases, and, therefore, the pile toe resistance increases, which in turn causes the location of the force equilibrium (neutral plane) to move down. The main scenario is that, if the final location of neutral plane is in non-compressible soil layer where the soil settlement is small, downdrag is also small and, probably, negligible. However, if the settlement at the final location of the neutral plane is large, downdrag may be large and critical for the foundation.

The settlement calculation cannot be made without an assessment of the interdependence of length of the transition zone, location of the neutral plane, and magnitude of pile toe resistance as a function of net toe penetration, which in turn is a function of the magnitude of soil settlement. This is illustrated in Figure 15. Case 1 is for a case where the soil settlement is small, the transition zone is long, the toe resistance is small, the neutral plane "lies high", the drag force is small, and the pile head settlement (A) is small. Case 2 is for the identical soil resistance distribution, only, the soil settlements are larger, which results in a larger mobilized pile toe resistance, a lower location of the neutral plane, a larger drag force, and a larger pile head settlement (B). Should the soil settlement increase further, the toe resistance could become larger than the value determined in the static loading test (presumed to be how the distribution of ultimate resistance was determined). The neutral plane would then lie close to or level with the pile toe. This only occurs where the load-movement response of the soil is very stiff.

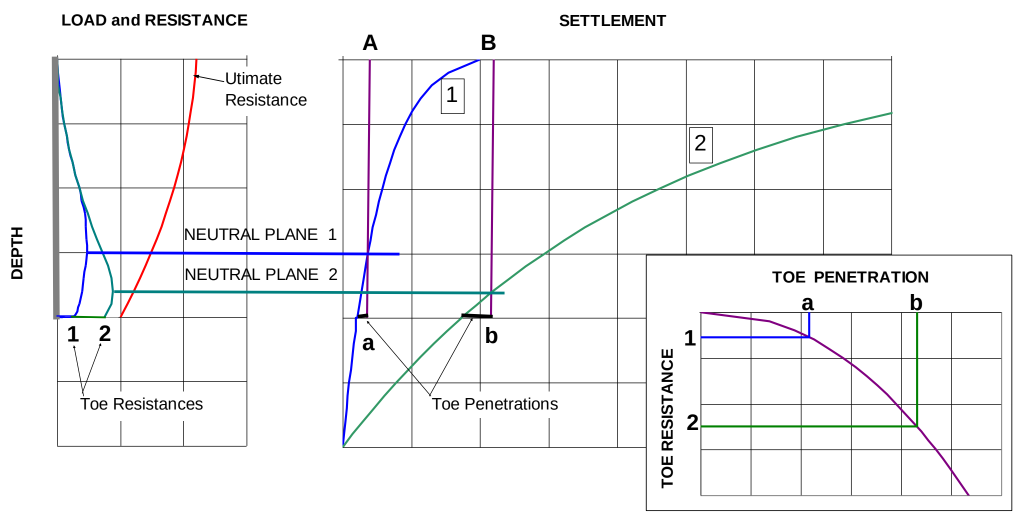


Fig. 15 Example of the interdependence of length of the transition zone, location of the neutral plane, and magnitude of pile toe resistance as a function of net toe penetration in a homogeneous soil. Case (1) small settlement and Case (2) large settlement.

A settlement analysis starts with determining the location of the force equilibrium. In many cases, perhaps most cases, once it has been established that the location is where the soil settlements are small, the analysis is essentially completed. The analysis presupposes that all influencing factors are considered, such as the load on the pile foundation and other changes to the effective stress in the ground since the completion of the construction of the structure supported on the piles. The latter is of primary importance. Note, settlement is caused by increase of effective stress in the soil, and drag force cannot contribute to the settlement of a piled foundation.

Where there is no obvious non-compressible soil layer, the key to determine the location of the force equilibrium lies in finding the magnitude of the toe resistance, which is determined by the particular toe response function, the q-z function. A simple q-z function is provided by the following relation (Fellenius 2002).

$$\frac{R_{mob}}{R_{ult}} = \left(\frac{\delta_{mob}}{\delta_{ult}}\right)^e$$

where R_{mob} = mobilized resistance
R_{ult} = ultimate resistance
δ_{mob} = movement mobilized at R_{mob}
δ_{ult} = movement mobilized at R_{ult}
e = an exponent usually ranging from a small value through unity

If, for example, a test has mobilized a toe resistance of 1,000 kN at a toe movement of 10 mm and the e-exponent is 0.6, then, the toe resistance for a toe movement of 8 mm is 875 kN. Or, if the toe resistance is 750 kN, then, the toe movement is 6.2 mm. Finding the toe resistance that fits the movement is a simple iterative procedure between toe resistance values deciding the location of the force equilibrium, which determines the toe movement value, which determines the toe resistance. As the q-z function movement exponent is an estimated value and the process depends on the exercise of good judgment, it does not make sense to look for decimals in the results.

For a single pile, or a pile group consisting of just a few piles, the load on the pile group will not cause significant settlement of the pile foundation. The settlement is almost entirely governed by the “environmental aspects”, i.e., increase of effective stress due to other causes than the loads applied to the piles. Only for a large group of piles will the loads on the piles contribute significantly to the settlement. As illustrated in Figure 16, increase of stress in the ground due to the loads on the pile group (pile cap) can then be calculated by transferring the loads to a virtual footing, called “equivalent footing” or "equivalent raft" placed at the pile toe level and having the same footprint as the pile cap. The settlement of the piled foundation is then assumed to be equal to the settlement calculated for the equivalent footing. An important aspect of the settlement calculation by means of the equivalent footing approach is that it allows a calculation that can incorporate all the environmental factors external to the pile group, as indicated in the figure.

When calculating the settlement for the equivalent footing, the reinforcing (stiffening) effect of the piles must be taken into account. This can be done quite simply by proportioning the soil modulus and the pile modulus to the respective portions of the “footing” area of soil and piles and assigning the combined modulus to the soil layer between the pile head and the pile toe. In most cases, the combined modulus is so large that the settlements calculated for the soil between the

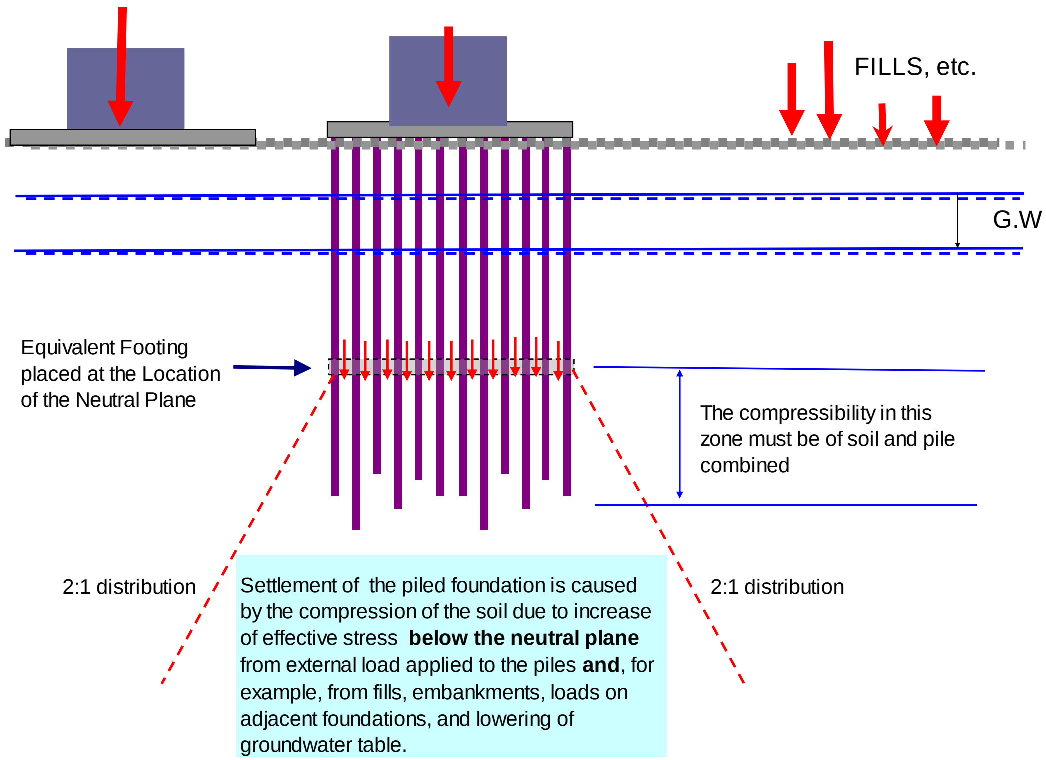


Fig. 16 Principle of analysis of settlement for a pile group

level of the equivalent footing and the pile toe depth are negligibly small. For small pile groups, say, four to twenty piles, it may be more realistic to place the equivalent raft at the neutral plane and distribute the stress over the raft footprint by 1(H):5(V). When considering the stiffening effect of the piles, no significant settlement due to the loads on the piles will develop above the pile toe.

The total capacity of a pile can be established by simplified, empirical approaches, such as stress-independent analysis (provided that they are referenced to reliable sources applicable to the case). However, distribution of shaft resistance is better if employing methods which recognize that the shaft resistance is proportional to effective stress and, therefore, in homogenous soil, shaft resistance always increases with depth. An incorrect placement of the neutral plane will have a considerable effect on the value of settlement calculated for the piled foundation.

The foregoing is demonstrated in Figure 17. The example involves 54 m long piles “floating” in a homogenous, settling soil (the example is based on an actual case, but somewhat adjusted to the message intended). The settling soil layer is about 65 m thick and is followed by a non-settling soil of low strength. The pile capacity is about 8,500 kN and consists mostly of shaft resistance (toe resistance is only about 300 kN).

The results of two different calculations, “A” and “B”, are shown. In “A”, the capacity is assumed represented by a constant unit shaft resistance, which results in the linear shaft resistance distribution and a neutral plane at a depth of 24 m. When instead, the capacity is assumed represented by a unit shaft resistance proportional to the overburden effective stress, as shown in “B”, the shaft resistance distribution becomes curved, and the neutral plane lies at a depth of 36 m. Due to a small fill on the ground surface and a lowering of the groundwater table, a settlement at the ground surface of 200 mm is anticipated. Two settlement distributions are indicated, "A" and "B". As sometimes is done, settlement distribution "A" is assumed linear through the settling depth. For distribution "B", the calculated settlement values are plotted as calculated for intermittent layers of soil compressibility parameters matched to give the same 220 mm settlement at the ground surface. Distribution "B" reflects the fact that for the same actual increase of stress, the relative increase becomes smaller with depth and the calculated settlement, therefore, will reduce exponentially with depth.

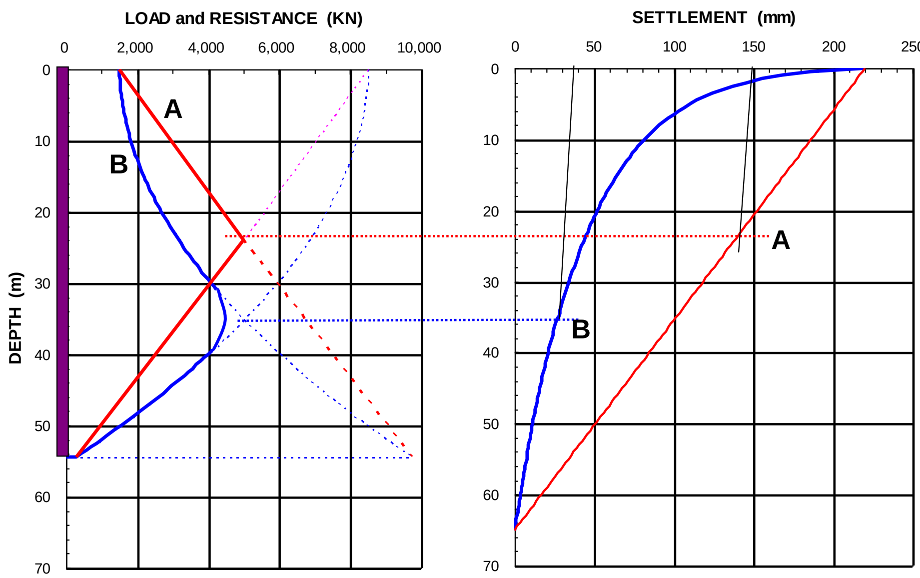


Fig. 17 Comparison between total stress results and effective stress results

The consequence for the results of the two simplifications—linear distribution of shaft resistance and of settlement—as opposed to the results of the more realistic distributions, is obvious: the calculated settlement for the piled foundation reduces from 150 mm to smaller than 40 mm, making the difference between a design that may have to be rejected to one that could be accepted.

Piled raft. As indicated in the case history by Endo et al. (1969) presented in Fig. 12B and in the typical settlement distributions shown in Figs. 15 and 17, at the level of the pile cap, there is no contact stress between the underside of the pile cap and the soil, because the soil will always settle more than the pile cap. It is therefore incorrect to allow any contribution from contact stress. The exception to this is in the case of a piled raft, which is a term referring to a piled foundation designed with a factor of safety for the piles of close to unity, or better expressed, where the neutral plane is designed to be located close to or at the underside of the raft. Only if the external loads are equal to or larger than the combined pile capacities will there be a contact stress.

The emphasis of the design for a piled raft is on ensuring that the contact stress is uniformly distributed across the raft. The piled-raft design intends for the piles to serve both as soil reinforcing (stiffening) elements reducing settlements and as units for receiving the unavoidable concentrated loads on the raft. This condition governs the distribution across the raft of the number and spacing of the piles. The design first decides on the depth and number of piles (average spacing and lower boundary number of piles) necessary for reinforcing the soil so that the settlement for the raft is at or below the acceptable level. This analysis includes all loads to be supported by the raft. Thereafter, the spacing and number of piles to carry load concentrations are designed as to depth and locations. An iterative procedure of these steps may be required.

SUMMARY

Only for piles deriving most of the capacity from shaft resistance and subjected to residual force will the results of a static loading test be interpreted to show a capacity value (ultimate resistance) that can be intuitively perceived and accepted as such. For most pile tests, therefore, the capacity is a crude engineering concept that requires a specific definition. The reason is that the pile toe does not exhibit an ultimate resistance, but has a curved load-movement without sudden change of curvature or other indication of ultimate resistance. While the concept of pile capacity is still useful, to advance, the engineering design practice needs to put more emphasis on settlement analysis and consider that pile response to load is primarily a movement response to the load coupled with the soil settlement due to environmental factors.

The design of a piled foundation 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 drag force. The allowable load includes dead and live load, but not drag force.

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

Settlement. The settlement of a piled foundation is caused by stress increase in the soil due to fills, embankments, and excavations, change of groundwater table, and the sustained load on the pile group from the supported structure. Estimation of settlement requires knowledge of the location of the force equilibrium and the soil settlement at that depth, the settlement equilibrium. The settlement of a small group of piles is best analyzed in terms of q-z functions for the pile toe in response to the load applied to the pile and incorporating the aspects of necessary conformity with the movement at the neutral plane. A large group of piles will also be affected by the stress increase in the soils below the pile toe due the external loads supported on the piles. This can conveniently be analyzed by means of an equivalent footing placed at the location of the neutral plane or at the pile toe.

APPENDIX I — REFERENCES

Altaee, A. and Fellenius, B.H., 1994. Physical modeling in sand. Canadian Geotechnical Journal, (31)3, 420-431.

ASCE, 1993. US Army Corps of Engineers Technical Engineering Design Guide No. 1 Pile Foundations, 142 p.

ASCE, 1994. US Army Corps of Engineers Technical Engineering Design Guide No. 7 Bearing capacity of Soils, 99 p.

Baker, C.N., Park, G., Braid, J.L, Drumright, E.E., and Mensah, F. (1993). Drilled shafts for bridge foundations. Federal Highway Administration, FHWA, Washington, Report No. FHWA RD 92 004, 335 p.

Barker, R.M., Duncan, J.M., Rojiani, K.B., Ooi, P.S.K, Tan, C.K., and Kim, S.G., 1991. Manual for the design of bridge foundations. National Cooperative Highway Research Programme, Report 343, Transportation Research Board, Washington, D. C., 308 p.

Bjerrum L. and Johannessen, I.J., 1965. Measurements of the compression of a steel pile to rock due to settlement of the surrounding clay. Proceedings 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, September 8 - 15, Vol. 2, pp. 261 - 264.

Bjerrum L. Johannessen, I.J., and Eide, O., 1969. Reduction of negative skin friction on steel piles to rock. Proceedings 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, August 25 - 29, Vol. 2, pp. 27 - 34.

Bozozuk, M., 1981. Bearing capacity of a pile preloaded by downdrag. Proc. 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, June 15 -, 19, Vol. 2, pp. 631 - 636.

Briaud J-L and Gibbens R.M., 1994, Predicted and measured behavior of five spread footings on sand — Tests and Prediction Proceedings of a Symposium sponsored by the Federal Highway Administration at the, 1994 ASCE Conference Settlement ’94. College Station, Texas, June 16 - 18, pp., 192 - 128.

Canadian Foundation Engineering Manual, CFEM, 1992. Third Edition. Canadian Geotechnical Society, BiTech Publishers, Vancouver, 512 p.

Clemente, F.M., 1981. Downdrag on bitumen coated piles in a warm climate. Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, June 15 -, 19, Vol. 2, pp. 673 - 676.

Clemente, F.M., 1979. Downdrag. A comparative study of bitumen coated and uncoated prestressed piles. Proceedings, Associated Pile and Fittings 7th Pile Talk Seminar, New York, N.Y., pp. 49 - 71.

Endo M., Minou, A., Kawasaki T, and Shibata, T, 1969. Negative skin friction acting on steel piles in clay. Proc. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, August 25 - 29, Vol. 2, pp. 85 - 92.

Fellenius, B.H., 1972. Downdrag on piles in clay due to negative skin friction. Canadian Geotechnical Journal, 9(4), pp. 323 - 337.

Fellenius, B.H., 1984. Negative skin friction and settlement of piles. Proceedings of the Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18 p.

Fellenius, B.H., 1989. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75 - 82.

Fellenius, B.H., 2001. Determining the true distribution of load in piles. American Society of Civil Engineers, ASCE, International Deep Foundation Congress, An International Perspective on Theory, Design, Construction, and Performance, Geotechnical Special Publication No. 116, Edited by M.W. O'Neill, and F.C. Townsend, Orlando, Florida, February 14 - 16, 2002, Vol. 2, pp. 1455 - -1470.

Fellenius, B.H., 2002. Basics of foundation design. Electronic Edition. www.Geoforum.com, 250 p. (*and later editions*).

Fellenius, B.H. and Broms, B.B., 1969. Negative skin friction for long piles driven in clay. Proc. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, August 25 - 29, Vol. 2, pp. 93 - 97.

Fellenius, B. H. and Samson, L., 1976. Testing of drivability of concrete piles and disturbance to sensitive clay. Canadian Geotechnical Journal, 13(2), 139-160.

Fellenius, B.H. and Altaee, A., 1995. The critical depth – How it came into being and why it does not exist. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering Journal, London, No. 113-2, pp. 107 - 111. Discussion and Reply in No. 119-4, pp. 244 - 245.

Fellenius, B.H., Harris, D., and Anderson, D.G., 2004. Static loading test on a 45 m long pipe pile in Sandpoint, Idaho. Canadian Geotechnical Journal, 41(4) 613-628.

Gregersen, O.S., Aas, G., and DiBiagio, E. (1973). Load tests on friction piles in loose sand. Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, ICSMFE, Moscow, August 1973, Vol. 2, Paper 3/17, pp. 109 – 117.

Leung, C.F., Radhakrishnan, R., and Tan, S.A., 1991. Performance of precast driven piles in marine clay. American Society of Civil Engineers, ASCE, Journal of Geotechnical Engineering, 117(4) 637-657.

Holloway, D.M., Clough, G.W., and Vesic A.S. (1978). The effects of residual driving stresses on pile performance under axial load. Proceedings of the 10th Offshore Technology Conference, Houston, TX., Vol. 4, pp. 2225 - 2236.

Hunter, A.H. and Davisson, M.T. (1969). Measurements of pile load transfer. Proceedings of Symposium on Performance of Deep Foundations, San Francisco, June (1968, American Society for Testing and Materials, ASTM, Special Technical Publication, SPT 444, pp. 106 - 117.

OHBDC, 1991. Ontario Highway Bridge Design Code, 3rd. Edition, Code and Commentary, Min. of Transp., Quality and Standards Division, Toronto.

Osterberg, J.O., 1998. The Osterberg load test method for drilled shaft and driven piles. The first ten years. Great Lakes Area Geotechnical Conference. Seventh International Conference and Exhibition on Piling and Deep Foundations, Deep Foundation Institute, Vienna, Austria, June 15 - 17, 1998, 17 p.

Walker, L.K., Le, P., and Darvall, L., 1973. Dragdown on coated and uncoated piles. Proc. 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, August, Vol. 2, Paper 3/41, pp. 257 - 262.