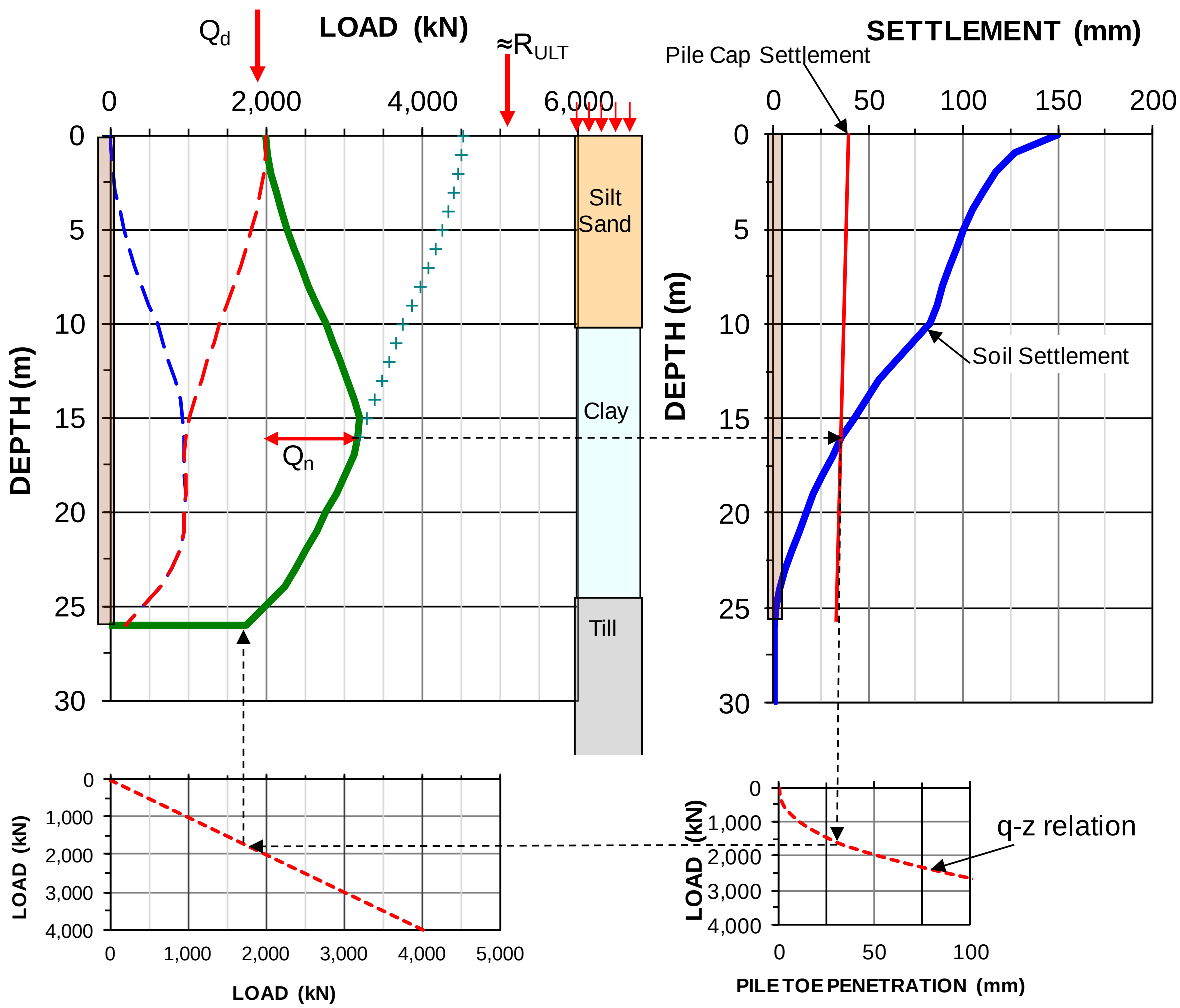

341. Fellenius, B.H., 2014. Piled foundation design as reflected in codes and standards. Proceedings of the DFI-EFFC International Conference on Piling and Deep Foundations, Stockholm, May 21-23, pp. 1013-1030.

PILED FOUNDATION DESIGN AS REFLECTED IN CODES AND STANDARDS

Bengt H. Fellenius, Dr.Tech., P.Eng.
Consulting Engineer, 2475 Rothesay Ave., Sidney, BC, Canada, V8L 2B9. <Bengt@Fellenius.net>

ABSTRACT

An example from a guidelines document for the Eurocode is discussed that addresses design for geotechnical strength of piled foundations in settling soil, generating drag force and downdrag. A modified example of geotechnical response is then analyzed in a failure mode, Ultimate Limit States (ULS), and stationary long-term state, Serviceable Limit States (SLS). It is shown that the Eurocode principles, when applied to correctly determined forces, results in an irrational and costly design. The results of the analysis of the modified example are applied to eight different codes pertaining to piled foundations from USA, Canada, Europe, and other countries. Several of the codes require that the drag force be included as a load in a ULS design, which an absurd and costly requirement, in some codes, in one case made even worse by combining drag force with live load. A major weakness of almost all codes is their minimal treatment and sometimes total absence of aspects of settlement of piled foundation. In contrast, my main argument is that design for capacity, whether in working stress approach, or by LFRD or ULS approaches, is not always safe unless a thorough settlement analysis shows that the design also meets the requirements of the SLS.

1. INTRODUCTION

First, there were analysis, design, and construction establishing a State-of-Practice, SOP. Then, came codes and specifications summarizing the SOP, as interpreted by a committee. When the state-of-art, SOA, was not understood by that SOP-interpreting committee, the codes and specifications are uneconomical and, sometimes, also unsafe.

The paper first presents an example quoted from two commentaries on the Eurocode 7 on design for geotechnical strength by the Eurocode (1997). The example is then modified to a more realistic case and analyzed in failure mode (ultimate limit states, ULS) and in the context of settlement (serviceability limit states, SLS). The analysis results are then applied to eight codes addressing piled foundation design and changes necessary for the referenced codes and specifications to be closer to the SOA are identified.

The referenced codes employ different terms. In the interest of uniformity, I have applied the term "load" to denote what some codes call "action", "design action effect", or demand". Similarly, I use the term "dead load" to denote "permanent", "sustained", or "long-term" loads. I use "live load" to denote "transient", "temporary", "persistent", or "short-term" loads. I use the term "drag force" to denote the accumulated negative skin friction along a pile having its maximum value at the neutral plane (below which depth, the total axial force in the pile reduces due to positive shaft resistance acting along the pile). Some codes here use terms such as "drag load", "dragdown force", "N.S.F.", "negative friction", or even "downdrag". "Downdrag" is not a synonym to drag force, however, but the term for the settlement of a pile when dragged down by the soil settlement.

Conventional working stress design (WSD) applies a global factor of safety (F_s) defined as a ratio between the sum of the resistances and the sum of the loads. ULS design (also called Strength Limit State in the Load and Resistance Factor Design, LRFD) applies factors to each component or sum of similar components. These factors are called "load factors" when applied to load, and "resistance factors" when applied to resistances. Obvious terms, it would seem. However, a few codes, e.g., the Eurocode, use the term "partial factor of safety" to apply to either a strength parameter or load component. To denote "resistance factor", the Australian code uses the term "strength factor" and the Canadian code uses the term "performance factor". I will use the US and Canadian term "load factor" for loads and forces, which factors are equal to or larger than unity and US term "resistance factor" for resistances, which factors are equal to or smaller than unity.

2. EUROCODE 7, EXAMPLE 7.4

Two commentaries (guidelines) on the Eurocode 7 (Simpson and Driscoll 1998, Frank et al. 2004) present a design example comprised of a 300-mm diameter bored (circular) pile installed to 16.5 m depth through 5 m of soft clay above a thick layer of stiff clay (the two documents use the same example). The unfactored dead load assigned to the pile is 300 kN. Live load is not included.

The example information is summarized in Figure 1. The guidelines state that the pile shaft resistances are determined in an effective stress analysis that results in an average unit shaft resistance in the "soft clay" of 20 kPa and in the "stiff clay" of 50 kPa (50 kPa is right at the borderline between firm and stiff consistency). The toe resistance is assumed to be zero. The shaft resistances in the two layers are 94 kN and 543 kN, respectively, combining to a total capacity of 637 kN. A surcharge will be placed over the site, generating consolidation settlement. The specific surcharge stress is not mentioned. Nor is the location of the groundwater table indicated.

A back calculation for the condition of the guidelines example (long-term; full consolidation has developed from the surcharge placed on the ground surface), applying the stated unit shaft resistance values, shows that the surcharge stress is 30 kPa, the groundwater table lies at the ground surface and the pore pressure is hydrostatically distributed with depth, the total density of the soft clay layer is 1,800 kg/m^3 (w_n = 40 %; e_0 = 1.09) and 1,960 kg/m^3 (w_n = 28 %; e_0 = 0.74) in the stiff clay layer, and the effective stress proportionality coefficient, ß, is 0.40 in both clay layers. A beta-coefficient of 0.40 is very large for a soft clay and large for a stiff clay unless it would be a clay till or similar. However, as the stated purpose of the example is to demonstrate the Eurocode handling of negative skin friction, selecting realistic coefficients is not essential to the example.

It is likely that the piles are constructed before the surcharge is placed or before any appreciable consolidation from the surcharge has developed, which represents a short-term condition. Applying the same beta-coefficients, the effective stress calculation show the shaft resistance along the full length of the pile to be 450 kN for the short-term condition.

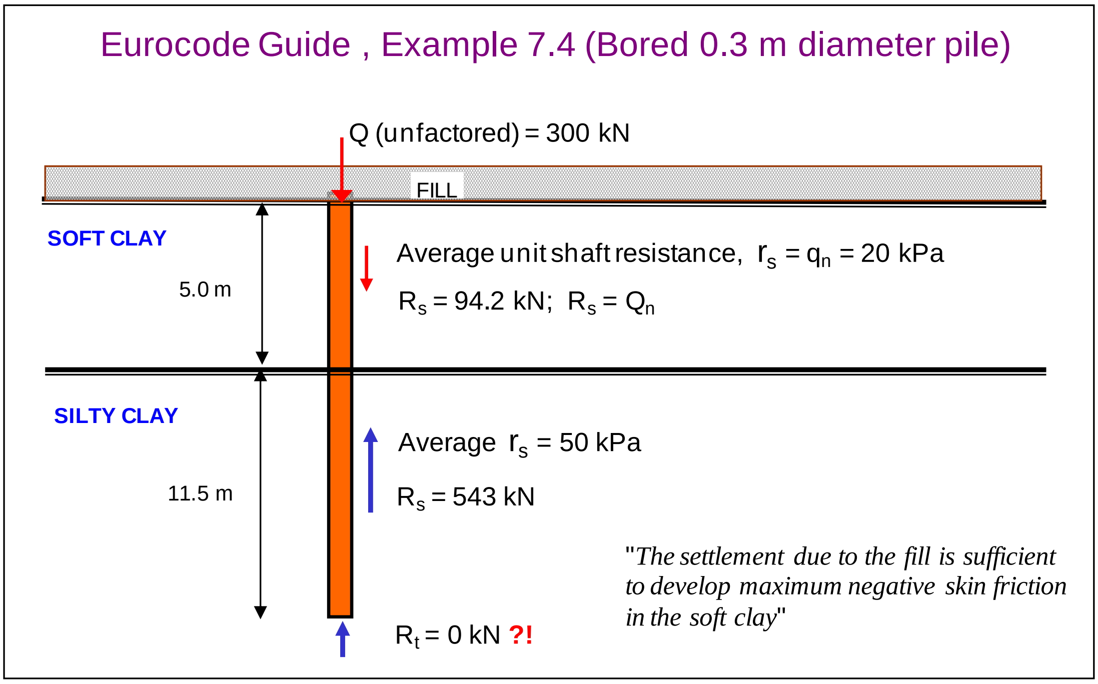

Fig. 1 Example 7.4 according to Simpson and Driscoll (1998) and Frank et al. (2004)
R_s = Accumulated unit shaft resistance, r_s Q_n = Accumulated negative skin friction, q_n

Checking the conditions for a conventional global factor of safety design, the factor of safety on the long-term capacity shows to be 2.1. Conventionally, a factor of safety of 3.0 applies to the design calculations based on theoretical static analysis. Thus, the "design" appears to be short on capacity. Had the 637-kN capacity been determined in a static loading test, the 2.1-factor of safety would have been acceptable in a conventional global factor of safety design. The factor of safety for the short-term condition shows to be 1.5, which by the conventional approach would be inadequate even if the capacity would have been determined in a static loading test.

According to Frank et al. (2004), the Eurocode considers the drag force to be a permanent load acting on the pile much the same way as the load applied to the pile head. Moreover, the assumption is made that the settlement due to the surcharge only causes negative skin friction in the soft clay (94 kN drag force) and no negative skin friction and no settlement develops in the lower layer, the stiff clay—but full positive shaft resistance does develop in that layer. The Eurocode disregards the contribution from the shaft resistance in the soft clay layer allowing support only from the 543-kN shaft resistance in the stiff clay layer (as mentioned, the toe resistance is assumed to be zero).

The Eurocode applies the principles of ultimate limit states, ULS, for analysis of capacity (geotechnical strength), that is, factoring resistances and loads separately, requiring the sum of the factored resistances to be equal to or larger than the sum of the factored loads.

The guidelines apply two approaches to the design of the example pile. According to the Eurocode DA-1, Combination 2, ("normally considered first"), the load and resistance factors applicable to the design calculations for the dead load applied to the pile is 1.00 and the load factor for the drag force is 1.25. The resistance factor on the shaft resistance ("design resistance") is 0.77 (actually, this is the inverse of the partial factor safety, 1.30, that the Eurocode applies to shaft resistance). For the long-term condition, the sum of the factored loads is 1x300 +1.25x94 = 417 kN and the factored resistance is 0.77x543 = 418 kN. According to the Eurocode, therefore, the long-term condition is acceptable.

In the alternative approach, the Eurocode DA-1, Combination 1, the load factor for the dead load applied to the pile is 1.35 and the same load factor is applied to the drag force. The resistance factor on the shaft resistance is 1.00. Per the guidelines, the factored load is 1.35x(300 + 94) = 532 kN, and the factored resistance is 1.00x543 = 543 kN. Thus, also for this approach, according to the guidelines, the long-term condition is acceptable.

For the short-term condition, it can be assumed that no drag force would have developed and, therefore, the guidelines would employ shaft resistance acting along the full length of the pile. With no surcharge effect on the effective stress distribution, short-term pile capacity is 450 kN and the short-term factor of safety is only 1.5. According to Eurocode DA-1, Combination 2, the factored load and the factored resistance are 1.00x300 = 300 kN and 0.77x450 = 347 kN, respectively. Thus, the Eurocode would find the pile design results acceptable also for the short-term condition. According to Combination 1, the factored load and the factored resistance are 1.35x300 = 405 kN and 1.00x450 = 450 kN, respectively, again showing the short-term condition to be acceptable.

The foregoing is how the design approach is presented in the commentaries (I have added the aspects of the short-term condition). In my opinion, the Eurocode approach, as presented in the commentaires, is quite wrong—on the dangerous side. As mentioned, the guidelines state that negative skin friction only develops in the soft clay and imply that no settlement will develop in the stiff clay. This is hardly realistic. Why would negative skin friction not develop in the stiff clay? Numerous full-scale tests in different soils have shown that fully mobilized shaft shear—in the negative as well as in the positive direction—requires only a very small movement between the pile shaft and the soil. Possibly, the authors of the example had in mind that the settlement in the stiff clay is much smaller than in the soft clay and such small settlement might be of negligible concern for the structure supported on the pile(s), but that is an issue for the settlement of the foundation supported on the pile(s) and not for the development of negative skin friction and drag force. If positive direction shaft shear along the pile can be relied on during the development toward the long-term (ongoing consolidation), then, surely, the same "ability" must be assumed to be available also for the negative direction shaft shear.

In my opinion, typical and reasonable compressibility parameters for the two clay layers would be Janbu virgin modulus numbers, m, of 15 (optimistically) and 40, and re-loading modulus numbers, m_r, of 150 and 400, respectively. The virgin modulus numbers correlate to virgin compression indices, C_c, of 0.32 and 0.10, respectively (the corresponding void ratios are mentioned above). Moreover, it would be reasonable to assume that both layers are somewhat overconsolidated, and I have assumed preconsolidation margins of 5 kPa and 20 kPa, respectively. These values characterize the soft clay as compressible and the stiff clay as a soil of low compressibility. I also assume that the stiff clay layer is 15 m thick and deposited on a firm layer of minimal compressibility, e.g., a very dense glacial till.

Figure 2A shows the condition for the more realistic load distribution for the long-term condition when the consolidation process has developed an equilibrium between the downward acting forces and the upward acting resistances. No toe resistance is indicated because the guidelines state that this is the case for the example.

The calculations of load distributions and settlement for the guidelines example and the modified example are performed using the UniPile program (Goudreault and Fellenius 2013). The analysis follows generally accepted principle is a in a in a c in an effort s of effective stress analysis as detailed in Fellenius (2014).

Fig. 2B shows the calculated distribution of the long-term settlement of the soil and the pile. I have assumed that the pile is a single pile for which, then, the load applied to the pile will not cause any appreciable consolidation settlement below the pile toe. Some pile head movement (settlement) will develop due to load transfer of the 300 kN dead load to the soil during the construction of the structure. It will be limited to the compression of the pile for the imposed axial load and the small load-transfer movement of the pile element nearest the pile toe. It is not included in the 35-mm long-term settlement of the pile, which is due to downdrag, i.e., settlement due to imposed pile toe movement and a small amount from additional compression as the axial load in the pile increases.

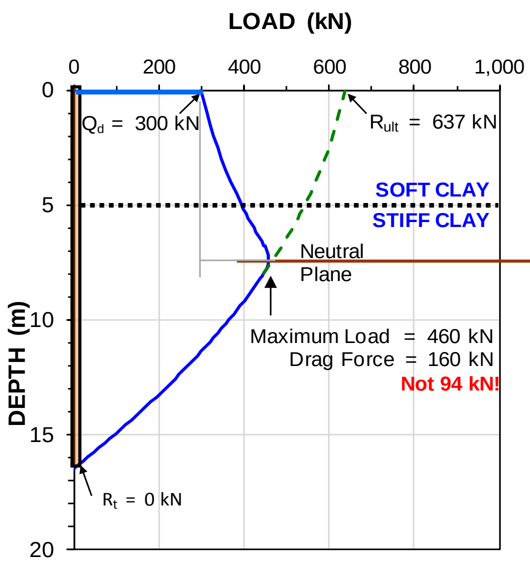

Fig. 2A Load distribution

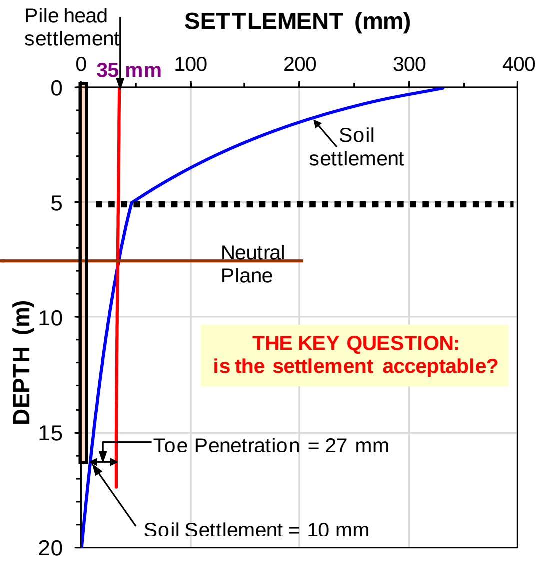

Fig. 2B Settlement distribution

The settlement distribution shown in the figure is that assumed developed at 90-% degree of consolidation, say, 30 years after placing the surcharge. Secondary compression would add about 10 to 20 mm of settlement to the 30-year value and then increase slightly with time. I would expect that the settlement after the first about 20 years after construction will be about 80 % of the values shown in the figure.

In the long-term, the soil settlement will result in negative skin friction along the pile that will accumulate to a drag force. The drag force plus the dead load from the structure supported on the pile will always be in equilibrium with the positive direction forces. Eventually, a stationary force equilibrium will develop at a depth called "neutral plane" ("equilibrium plane" might be the better term). For the guidelines example, as illustrated in Figure 2A, the neutral plane will be at a depth of 7.4 m. There is always a transition zone from negative to positive direction of shear along the pile and a small transition zone is indicated by the curved change from increasing load to decreasing. When the soil settlement relative to the pile is large, the height of the transition zone is small, when the settlement is small, the height is large. In the latter case, the drag force is smaller than in the former case. However, the location of the neutral plane is approximately the same, be the settlement small or large.

As indicated in Figure 2A, the drag force is 160 kN and the maximum load will be 460 kN. Below the neutral plane, in the what some call the "stable zone" (the soil is no less stable above, however), the accumulated positive shaft resistance is equal to the dead load plus the drag force, i.e., 460 kN—of course, this is what the force equilibrium means. However, the total ultimate shaft resistance is 637 kN, and after subtracting the 160-kN drag force, the remaining shaft resistance, the resistance below the neutral plane, is 477 kN, not 460 kN. The explanation to this discrepancy lies in the assumed transition zone. For example, if the transition zone is longer than the length indicated in Figure 2A, the drag force might reduce to, say, 100 kN, and the maximum load would become 400 KN. It would then seem as if the shaft resistance below the neutral plane, because of the equilibrium condition would be 400 kN, significantly smaller than 477 kN. The incongruity is due to comparing two mechanically conflicting conditions: when the pile responds to changing movement—is in flux—and when it is in a stationary condition.

The location of the force-equilibrium neutral plane is always the same as the location of the settlement equilibrium neutral plane, which is where there is no relative movement between the pile and the soil, i.e., where the soil and the pile(s) settle equally. This condition determines the settlement of the pile head after due consideration of the compression of the pile for the load between the pile head and the neutral plane. As mentioned, when the pile settlement is due to the soil dragging the pile down, it is termed "downdrag".

If one would argue that my assumed values of compressibility of the stiff clay are too conservative, and, quite optimistically, apply values resulting in much smaller consolidation settlement than shown in Figure 2B, then, the long-term soil settlement would still be sufficient to mobilize fully the negative skin friction and the positive shaft resistance. Indeed, were the piles to be constructed after the full consolidation had taken place, the distribution of load would still be the same as illustrated in Figure 2A, the final state would just take a longer time to develop. The long-term settlement would be small, of course. The transition height would therefore be longer.

Now, were the Eurocode principles applied with the correctly determined distribution of forces along the pile, the analysis would result in a factored load of 1x300 + 1.25x160 = 500 kN versus a factored resistance of 0.77x460 = 354 kN, and the design would no longer be acceptable according to the Eurocode.

For a real case, it is likely that the stiff clay would provide some toe resistance. For example, if a 100-kN toe resistance would be included in the analysis, I think most would agree that the margin against failure of the pile would have improved. However, improvement would not be recognized in an analysis applying the Eurocode principles, because the location of the neutral plane would have moved down, the drag force would have increased by about 50 kN, and the positive shaft resistance, below the neutral plane would have decreased with the same amount. Despite the increase of capacity, the factored resistance would have become smaller by the amount of 0.77x50 = 38 kN, and the increase of drag force would have added 1.35x50 = 68 kN to the factored load. In effect, providing toe resistance to the pile would actually have made the Eurocode indicate that the adequacy of the pile design had gone down!

I strongly disagree with the Eurocode design principles. The magnitude of the maximum axial force in the pile (consisting of the dead load plus the drag force) is only of concern for the axial structural strength of the pile. In contrast, when assessing a design for bearing capacity (geotechnical strength), the drag force must not be lumped in with the load from the structure. The main requirement or premise of design for bearing capacity is the adequacy of the margin against the possibility of the loads applied to the pile could exceed total resistance of the pile, i.e., resistance acting along the entire length of the pile. The safety factors are chosen to ensure a margin against that possibility. Drag force will develop only when the chosen factors are successful in providing that margin. If the factors are inadequate, the pile will start to fail, and, then, there is no negative skin friction and no drag force—nonetheless, the pile, most undesirably, fails. To avoid this misfortune, a proper design applies margins to the load and resistances. When considering the margin against failure—against the geotechnical response, i.e., capacity—the design must not add-in the drag force, which is a load that *à priori* assumes absence of failure. Indeed, the larger the drag force on a pile, the larger the margin against failure of the pile (provided the axial strength of the pile is not exceeded).

Consider a pile, similar to the guidelines example, installed in a uniform soft soil that is undergoing consolidation and has minimal toe resistance. (Such piles are often called floating piles). Assume further that the shaft resistance is about two to three times larger than the load to be applied to the pile—which would seem to be an adequate design. Eventually, an equilibrium will develop between the downward direction forces (dead load plus drag force), and the upward direction shaft resistance with a neutral plane located somewhere below the mid-point of the pile. However, applying the Eurocode principles, the factored loads would be larger than the factored resistance. Actually, even if no dead load would be applied, the Eurocode would show that the pile would not even be adequate to support its own drag force. Indeed, when the geotechnical response is correctly analyzed, a mainly shaft bearing pile can never meet the requirements of the Eurocode.

Assume now that the pile would have a significant toe resistance, say, just about equal to the total shaft resistance and the capacity would be doubled to four to six times the applied load. Now, the neutral plane would lie deeper and if the provision would be applied that all contribution to "bearing" above the neutral plane would be disregarded and instead be applied as load (drag force), then, this would show the pile to be inadequate to support any load according to the Eurocode! In effect, the Eurocode lumping-in the drag force with the loads from the structure in assessing geotechnical pile response is absurd and leads to large unnecessary foundation costs.

I must point out that my criticism is for the Eurocode and not for the authors of the guidelines, who simply report how the code treats the design example, claiming it neither to be right nor wrong.

The guidelines example only includes a permanent load. The Eurocode recognizes that live load and drag force do not act together—a pile element cannot have shaft shear in both the negative and positive directions at the same time. I assume that as long as the factored live load is smaller than the factored drag force, the Eurocode just leaves out live load from the analysis.

As mentioned, the objective of the guidelines example was to illustrate the principles of the Eurocode for analysis of a pile subjected to negative skin friction from settlements of the surrounding soil. It is, however, of interest to compare the settlement for the single pile to that of a group of piles. Let's assume that the example pile is one of a group of 64 piles in circular configuration at a center-to-center spacing of 4 pile diameters with a footprint of 130 m^2. Because of the large pile spacing, the piles inside the group would not have any appreciably reduced drag force compared to the single pile. (Drag force is limited by the buoyant weight of the soil in-between the piles. Therefore, a small spacing means that full drag force is reduced compared to that of the single pile). Moreover, because the piles reinforce the soil, the downdrag (settlement at the neutral plane) is significantly reduced. However, the load (64 x 300 kN) on the pile group will add stress (about 150 kPa) to the soil below the pile toe level, which will result in consolidation settlement between the pile toe level and the bearing, non-settling layer at 20 m depth. Calculations show that the pile group will settle close to about 80 mm in the long-term (the pile group calculation method is detailed in Fellenius 2014).

3. A MODIFIED EXAMPLE

The guidelines example can be turned into an more realistic case by modifying it to a 400-mm diameter, bored pile installed to or into the bearing, non-settling layer at 20 m depth and allowing for a more typical load to support from the structure, say, 800 kN. For good measure, I assume that this load includes a 200-kN live load. The soil parameters given for the guidelines example are applied also to the modified example and a similar surcharge will be placed. However, I will allow for a toe resistance commensurate with a dense soil layer at the 20-m depth.

The analysis will address the conditions both before and after settlement of the soft and stiff clay layers due to the surcharge have resulted in "negative skin friction effects" on the pile—drag force and downdrag. I assume that the downdrag-imposed toe-penetration into the bearing layer will follow the relation defined by the q-z function called Ratio Function (Fellenius 2014) expressed by Eq. 1.

$$R = R_u \left(\frac{\delta}{\delta_u}\right)^{ӿ} \qquad [1]$$

where	R	=	resistance
	δ	=	movement at "R"
	R_u	=	"ultimate" resistance
	δ_u	=	movement at "ultimate" resistance
	ӿ	=	an exponent

I further assume that the structure supported by the pile is sensitive to settlement. If so, a conventionally imposed pile-head settlement limit is a "half-inch", let's say close to 15 mm. To satisfy this requirement, the neutral plane must be located at the depth of about 14 m. The pile toe penetration imposed by the downdrag will then be about 10 mm, which I assume will result in a 5 MPa toe stress; about 600 kN.

A typical toe response for a pile toe is a doubling of the toe resistance for a tripling of the movement. Thus, a pile toe having a 5 MPa resistance for a 10-mm movement will have a 10 MPa resistance for a 30-mm movement. The corresponding total toe resistances for the example pile are 630 kN and 1,260 kN, respectively. Inserting the two pairs of values into Eq. 1 results in an exponent of 0.63. Despite the fact that ultimate toe resistance does not exist in the real world (Fellenius 1999; 2014), and, therefore, somewhat arbitrarily, I define that the 10-MPa unit toe resistance at 30-mm movement will represent an "ultimate" condition for the pile toe response of the modified example.

The long-term is defined as the conditions after 90-% consolidation of the soft and stiff clay layers has developed. Two states must be analyzed. The ULS condition, which is the state where soil failure occurs and the pile is in flux—is moving—and the SLS condition which is the stationary state—the distance to the neutral plane and the load distribution stay essentially unchanged over time.

Note, for the stationary condition—SLS—, the "assumed" values of toe resistance and toe penetration cannot be chosen independently of each other because they are interconnected by their q-z function. If the toe resistance would be assumed to be zero, the calculation would result in a neutral plane at 8.9 m depth. However, this would only be possible if the q-z relation states that the toe resistance is zero regardless of toe penetration (such as for the guidelines example—a floating pile with no toe resistance). Similarly, it would seem that the upper limit of toe resistance would be to assume that the neutral plane is at the pile toe level, which would result in a large toe resistance (1,260 kN for the example pile). However, that would require a large toe penetration (30 mm), which is not possible—the downdrag is too small to generate this toe movement. In fact, for every distribution of settlement and every q-z relation, there is only one location of the neutral plane and only one value of mobilized toe resistance. That is, three interdependent parameters govern the condition and any two of them determine the third.

The objective of serviceability limit states design, SLS, for a piled foundation is to combine the geotechnical response to the dead load placed on the pile (load distribution) and the settlement distribution around the pile. This will determine the stationary conditions for the pile.

Piled foundation design needs to consider both the ultimate limit states, ULS, and the serviceability limit states, SLS. For ULS conditions, neither negative skin friction, drag force, nor downdrag exist. The ULS design applies load and resistance factor to ensure a reasonable margin to the undesirable event of failure.

In contrast to the ULS design, SLS is design for deformation—settlement—of the piled foundation, and it applies neither load factors nor resistance factors. The designer assesses the calculated settlement in relation to the settlement that can be tolerated by the structure. Of course, there has got to be a suitable margin between the calculated settlement and the maximum settlement the foundation can tolerate. This margin is not achieved by imposing a certain ratio between the two settlement values. Instead, in calculations using unfactored loads and resistances, the designer exercises judgment applies realistically chosen, but upper and lower boundary values, for loads and assumes upper and lower boundary values for resistances, a conservative q-z relation, conservative values for compressibility parameters, etc. to produce results of neutral plane location, downdrag amounts in a settlement estimate that then must not exceed the maximum tolerable for the particular foundation. The analyses must include a realistic depth to the location of the neutral plane. The upper boundary settlement will then represent sufficiently improbable outcome of the design; "improbable", yes, but still mechanically possible.

The parameters used in the analysis of the modified example were combined with a pile E-modulus of 30 GPa and an assumed t-z function for the shaft resistance response—a Hansen 80-% function with a slight post-peak softening relation was assumed representing the shear-movement relation along the pile shaft, same for pile elements in both clay layers—with the peak occurring at a 10-mm movement. Figure 3 shows the results from a static loading test simulated using the UniPile program: load-movement curves for the pile head, pile toe, pile shaft, and pile compression. The figure represents the condition existing in a conventional design case: the test is performed before the surcharge has been placed—before any appreciable change has occurred in the effective stress distribution. Because it is the rare test that is continued until a pile head movement beyond about 40 mm (regardless of pile diameter), the lengths of the curves beyond 40 mm are dashed.

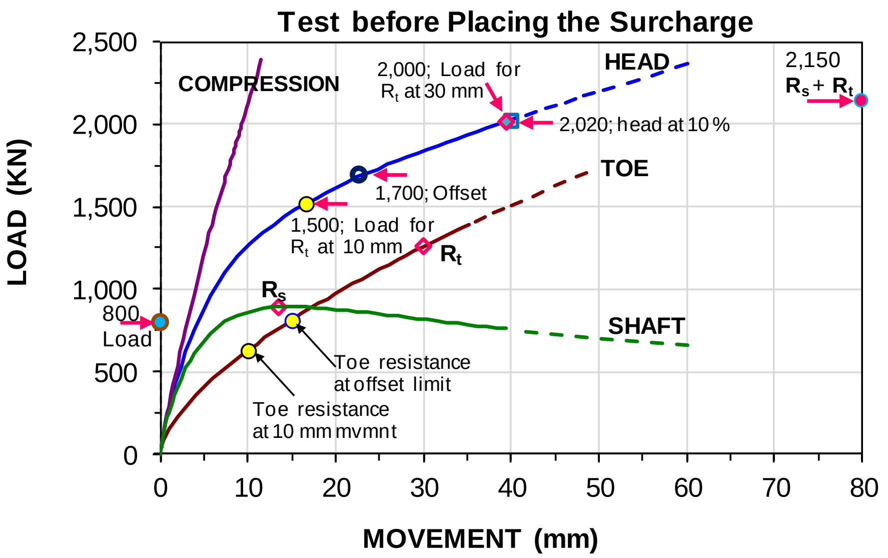

Fig. 3 Simulated results of a static loading test performed before placing the surcharge

The figure has been supplemented with values of load, as follows.

(1) The sum of the theoretically calculated shaft and toe resistances which amounts to 2,150 kN, using the assigned beta-coefficients and the toe resistance defined as the "ultimate".

(2) The Davisson Offset Limit (1,700 kN), which is defined by the intersection of the pile head load-movement curve with a line rising parallel the free-standing column load-movement line starting from a movement equal to 4 mm plus the pile toe diameter divided by 120. It is often considered to be a conservative estimate of capacity.

(3) The pile head loads (1,500 and 2,000 kN) that generated the pile toe resistances (890 and 1,260 kN) for 10 and 30 mm movements, respectively.

(4) The load (2,020 kN), that generated a pile head movement equal to 10 % of the pile-head diameter—coincidentally very close to the load that generated a 30-mm toe movement.

The Eurocode states that the load that generated a pile head movement of 10 % for the pile head diameter can be defined as the capacity of the tested pile. The value (2,020 kN) has been indicated in the figure. The "10-% definition" originates in a misquote of a statement by Terzaghi back in 1942. Terzaghi stated that assessing the capacity of a pile from a load-movement response should not be undertaken until the pile toe had moved at least 10 % of the pile toe diameter (Likins et al. 2012, Terzaghi 1942). Terzaghi did not state that the capacity would be the load that resulted in a 10-% pile-toe movement, let alone the load for a 10-% pile-head movement.

I have a preference for defining the capacity to be the pile head load that results in a pile toe movement of about "one inch"—about 30 mm. For the short-term test at a 30-mm toe movement, the toe resistance is 1,260 kN, the shaft resistance is 740 kN, and the load at the pile head (the capacity") is 2,000-kN. Due to the post-peak softening of the shaft resistance, the capacity—by any definition—is smaller than the capacity calculated from accumulatin of the theoretical ultimate resistances of each pile element.

Figure 4 shows the simulated load-movement curves for the pile head, pile toe, pile shaft, and pile compression for an assumed test performed at long-term condition. The thick line load-movement curves represent the test measurements for the case of all gages being set to zero immediately before the test started; much the way that a real-life test is recorded. Again, the curves beyond a pile head movement of 40 mm are dashed. The difference in shape of the load-movement curves as opposed to those shown in Figure 3 is because residual load is present in the pile at the start of the test. The development of residual load follows the same mechanism as the development of drag force. Thus, the long-term stationary condition existing at the start of the test will result in residual load in the pile distributed according to downward directed shear force above a neutral plane in equilibrium with upward directed shear force plus a small toe resistance.

The thin-dotted lines show the "true" load-movement curves as if residual load had been accounted for in the test or not been present. The specific "capacity" values determined by the same principles as in Figure 3 are indicated for the true curves. The increase of effective stress has resulted in a 20 % and 13 % increase of "capacity" for the Offset Limit, the 30-mm toe-movement value, and the theoretical ultimate resistance, respectively, as compared to the conditions before the surcharge.

As shown, the residual load, present and unaccounted for, has falsely increased the true shaft resistance and reduced the true toe resistance. It also implies a more plastic response of the pile-head load-movement than the true, "hardening" response. This means that if residual load would not be recognized to have affected the test results, it would be easy to believe that the test shows a well-accentuated capacity, whereas in reality, interpreting a capacity from the load-movement curve would be less than clear-cut. Again, due to the post-peak softening of the shaft resistance, the pile capacity—by any definition—is smaller than the capacity calculated from the theoretical ultimate resistance of each pile element.

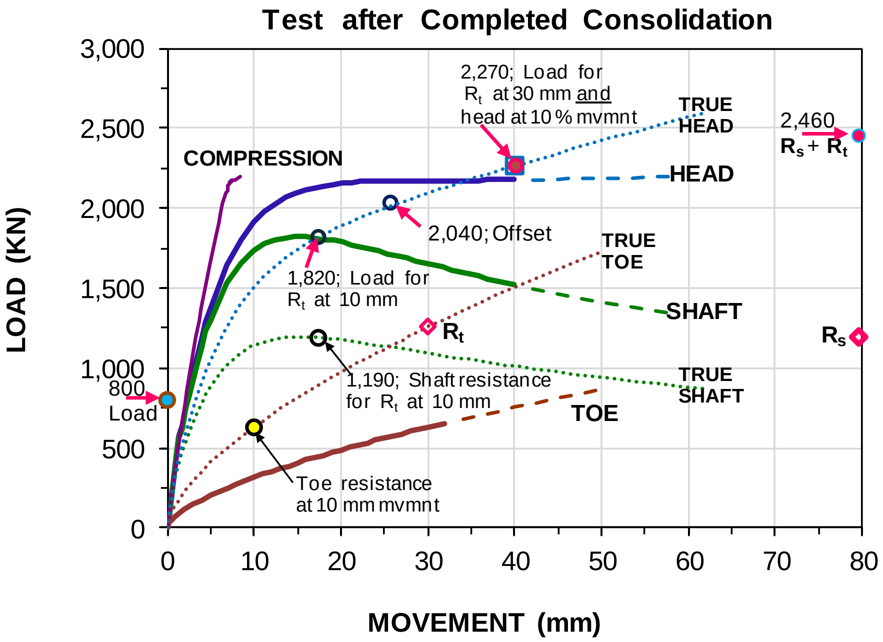

Fig. 4 Simulated results of a static loading test at long-term condition

If only the pile-head load-movement is recorded in a test, estimating the residual load is to a large extent a guesswork, albeit it could be somewhat informed. If the pile would be instrumented so that the load distribution and the shaft and toe resistance load-movement curves are measured, the true distributions can be evaluated. Note, when it would seem to make economic and technical sense to run a static loading test, the testing effort is mostly wasted unless the pile is instrumented to measure the load distribution.

Again, I prefer to define capacity from the pile toe movement. Applying the same 30-mm limit as for the no-surcharge condition results in a capacity of 2,270 kN, about 200 kN less than the theoretical value for accumulating the capacity of all pile elements. The shaft resistance mobilized for this toe movement is 1,010 kN, about 200 kN smaller than the theoretically ultimate shaft resistance.

Once the values of capacity have been determined in terms of ultimate total resistance and in shaft and toe resistances, the ULS calculation is simple, whether the design applies a global factor of safety or load and resistance factors.

How to pursue a SLS calculation is a bit less obvious. That is, the approach I consider correct, the settlement analysis based on the downdrag for single piles and small pile groups outlined in the foregoing, is straight-forward and quickly performed. So is the settlement analysis of larger pile groups. However, many apply a SLS analysis that employs loads and resistances with special factors applied and include the drag force in the calculations, which analysis is neither straight-forward nor quickly performed. For a start, what resistances should be used? I would suggest that the resistances developed for the stationary case are the ones to use. Employing the failure mode resistances would not be logical. And the values should be corrected for residual load.

Table 1 compiles the unfactored values for the ULS, without and with the effect of the surcharge, and the SLS long-term conditions for the modified example. The symbols used for denoting the various loads are those used in the Canadian Foundation Engineering manual; also defined in Figure 1). (That the long-term axial load at the neutral plane is equal to the long-term total shaft resistance is a coincidence).

TABLE 1 Resistances according to calculations ("Theor.") and simulated static "Test"

Item	Before Surcharge ULS Theor.	Before Surcharge ULS[1)] Test	After Surcharge ULS Theor.	After Surcharge ULS[1)] Test	After Surcharge SLS[3)] Test
Shaft Resistance R_s, (kN)	890	740[2)]	1,190	1,010[2)]	1,190
Toe Resistance, R_t, (kN)	1,260	1,260	1,260	1,260	630
Capacity, R^{ult}, (kN)	2,150	2,000	2,450	2,270	- - -
Depth to NP (m)	- - -		- - -	- -	13.7
Drag force, Q_n, (kN)	- - - -	- - -	- - -	- - -	590
Axial Load at NP (kN)	- - -	- - -	- - -	- - -	1,190
Pile Head Settlement (mm)	- - -	- - -	- - -	- - -	15

[1)] 30-mm toe movement [2)] after strain-softening [3)] 10-mm toe movement

4. THE MODIFIED EXAMPLE AS A BASE FOR DESIGN IN VARIOUS CODES

It is of interest to see how various codes around the world can be applied to the modified example, as I understand these codes are applied. The loads from the structure are 600 kN dead load (Q_d) and 200 kN live load (Q_l). For input to SLS calculations, I will use the values in the column headed "SLS".

4.1 The Eurocode

For the long-term condition (surcharge effect is included) and drag force according to the Eurocode, DA-1, Combination 2, the factored load is $1.00\,Q_d + 1.25\,Q_n$ and according to the Eurocode, DA-1, Combination 1, it is $1.35\,(Q_d + Q_n)$. This results in factored loads of 600 + 1.25x590 = 1,340 kN and 1.35(600+590) = 1,610 kN, respectively. Note, because the analysis according to the Eurocode includes the drag force, live load is not included.

According to the Eurocode, DA-1, Combination 2 and Combination 1, the factored resistance is $0.77R^{ult}$ = 0.77x(1,190 + 630) = 1,401 kN, and 1.00x(1,190 + 630) = 1,820 kN, respectively, which both are larger than the factored loads. The design for the long-term condition with surcharge is, therefore, acceptable. Had the calculations been performed for the no-surcharge condition before the drag force had developed, for when the maximum factored load is (1.35x800 = 1,080 kN), they would have resulted in factored resistances of 0.77x2,000 = 1,540 kN or 1.00x2,000 = 2,000 kN, respectively, larger than those for the long-term condition. This is a strange outcome, because the capacity has actually increased for the long-term. The apparent contradiction is entirely due to the fact that the drag force is lumped in with the load from the structure and only a portion of the shaft resistance is used. If in a real case, the calculated long-term condition would lead the designer to reduce the dead load (say, by increasing the number of piles), the location of the neutral plane be at a lower depth, the drag force would increase, and the resistance below the neutral plane would decrease. As a result, the design would show to be even less acceptable. Increasing the dead load would seem to be a better decision! Actually, in practice, the impasse is resolved, sort of, by not letting the nuisance of the real distribution of the forces bother the design effort, and instead declaring that the drag force is small or that the neutral plane really lies higher up, maybe at the boundary of the two soil layers. This might even work if no SLS (settlement) analysis is pursued.

4.2 The Canadian Code

The Canadian Highway Bridge Design Code (Canadian Standard Council 2006) applies both ULS and SLS, to geotechnical design. A load factor of 1.2 pertains to total axial pile load, and a resistance factor of 0.4 pertains to resistance determined from static analysis.

For the no-surcharge condition, theoretical analysis, the factored load is 1.2x800 = 960 kN and the factored resistance is 0.4x2,150 = 860 kN. Therefore, the short-term design is unacceptable. If assessed from the results of the static loading test, the factored resistance is 0.6x2,000 = 1,200 kN and the short-term design is now acceptable with a margin. SLS is not of concern for the short-term condition of the modified example.

For the surcharge condition, theoretical analysis, the factored resistance is 0.4x2,450 = 980 kN, showing the design to be right-on. If analyzed from the results of the static loading test, the factored resistance is 0.6x2,270 = 1,360 kN and the design is acceptable with good margin.

In the Canadian Code, the drag force is not included in the analysis against the geotechnical response. It only comes into play with regard to the axial strength of the pile. Thus, with the load factor on the 590-kN drag force of 1.25, the factored axial load is 1.2x600 + 1.25x590 = 1,460 kN (live load is not included) is well within the limits of a factored strength of a 400-mm diameter concrete pile.

For SLS conditions, the Canadian code applies the settlement analysis per the approach detailed above and illustrated in principle in Figure 2B. The long-term settlement would be limited to about 15 mm (10-mm toe movement plus about 5-mm pile compression above the neutral plane), which is an acceptable settlement for most foundations.

4.3 The AASHTO LRFD Specs

The AASHTO LRFD Specifications (AASHTO 2012) pertain to transportation projects, e.g., bridge foundations. It is the only Limits States geotechnical code in the USA although several guidelines such as the FHWA Manual (2006) addressing LRFD exist which are by many taken as equal to codes. The AASHTO Code is therefore often also applied to foundations for buildings. For the most common load combination, called Strength Limit I, the AASHTO code applies a load factor of 1.25 to dead load, and 1.75 to live load from vehicles. The load factor for drag force is 1.25. The AASHTO code specifies total stress analysis for piles in "clay", i.e., the α-method with, usually, a constant unit shaft resistance with depth, reserving effective stress analysis, the ß-method, for piles in "sand". The stated resistance factors for shaft and toe resistance in "clay" are 0.45 and 0.40, and in "sand" 0.55 and 0.50, respectively, as recommended by O'Neill and Reese (1999). The AASHTO code applies the same approach to the drag force as the Eurocode, i.e., the drag force is considered a load similar to the dead load on the pile and no shaft resistance contribution is allowed from the soil above the neutral plane.

The AASHTO code is usually interpreted to require live load and drag force to act simultaneously. That is, the drag force is added to the applied dead load and live load on the pile in assessing the pile for bearing capacity. This notwithstanding that Article 3.11.8 of AASHTO states that "*If transient loads act to reduce the magnitude of downdrag forces and this reduction is considered in the design of the pile or shaft, the reduction shall not exceed that portion of transient load equal to the downdrag force*". The commentary to this clause does not make the intent of the article more clear in stating that "*Transient loads can act to reduce the downdrag because they cause a downward movement of the pile resulting in a temporary reduction or elimination of the downdrag force. It is conservative to include the transient loads together with the downdrag*". The latter is not "conservative". Combining forces working in opposite directions is irrational and, therefore, including the drag force is simply "wrong".

For the same reason as for the Eurocode calculations, the AASHTO code calculations must employ resistances according to the SLS conditions. The soil for the example is clay, but as the resistance is calculated using effective stress, I have chosen the stated resistance factors of the ß-method in "sand".

Short-term Condition , theoretical analysis ("Theor.")

Factored load: $1.25\ Q_d + 1.75\ Q_l = 1.25 \times 600 + 1.75 \times 200 = 1{,}100$ kN
Factored resistance: $0.55\ R_s + 0.50\ R_t = 0.55 \times 890 + 0.50 \times 1{,}260 = 1{,}120$ kN

Long-term Condition

Factored load: $1.25\ Q_d + 1.75\ Q_l + 1.25\ Q_n = 1.25 \times 600 + 1.75 \times 200 + 1.25 \times 590 = 1{,}840$ kN
Factored resistance: $0.55(R_s^{\text{below NP}}) + 0.50\ R_t = 0.55 \times 560 + 0.50 \times 630 = 620$ kN

According to the AASHTO code, the design would be acceptable for the short-term (no surcharge) condition, but be vastly unacceptable for the long-term conditions despite the fact that the capacity had actually increased. Similarly to the Eurocode, and for the same reasons, reducing the dead load would actually indicate that the pile design had become less acceptable. Inasmuch the AASHTO code includes the live load with the drag force, it is actually more off the realistic modeling of the pile and soil interaction than the Eurocode. The conflicting results are due to the AASHTO Code confounding the SLS state with the ULS state.

4.4 The Australian Code

The ULS approach in the Australian Code ("Standard") usually applies a load factor of 1.20 on dead load, 1.50 on live load, and, for axial strength, 1.20 on drag force. (For special combinations of load, other factors may apply). The shaft and toe resistances have the same resistance factor and, if the resistance is determined from analysis, the factor is 0.40. A series of larger resistance factors are assigned to designs where the capacity is more reliably determined. Depending on the quality and number of static loading tests and other tests, those larger factors can become as high as 0.9. For the results of a single loading test similar in scope to the one presented here, a resistance factor of 0.6 appears to be suitable.

The Australian code expresses that, when assessing geotechnical strength of the pile in compression or in uplift, the design shall be assumed to be unaffected by drag force. See, for example, Poulos (2013). The stated reason is that under the "failure" condition of the pile for strength, negative skin friction will have been removed by the downward movement of the pile replacing the negative direction of shear with positive direction shear. Similar to the Canadian code, the Australian code combines drag force and dead load, but excludes the live load, when considering the adequacy of the axial strength of the pile at the neutral plane.

Applying the Australian code to the example results in the following.

Short-term Condition, theoretical analysis

Factored load $1.20\ Q_d + 1.50\ Q_l = 1.20 \times 600 + 1.50 \times 200 = 1{,}020$ kN
Factored resistance, Theor. $0.40\ R_s + 0.40\ R_t = 0.40 \times 890 + 0.40 \times 1{,}260 = 860$ kN
Factored resistance, Test $0.60\ R_s + 0.60\ R_t = 0.60 \times 740 + 0.60 \times 1{,}260 = 1{,}200$ kN

Long-term Condition

Factored load $1.20\ Q_d + 1.50\ Q_l = 1.20 \times 600 + 1.50 \times 200 = 1{,}020$ kN
Factored resistance, Theor. $0.40\ R_s + 0.40\ R_t = 0.40 \times 1{,}200 + 0.40 \times 1{,}260 = 985$ kN
Factored resistance, Test $0.60\ R_s + 0.60\ R_t = 0.60 \times 1{,}010 + 0.60 \times 1{,}260 = 1{,}360$ kN

The Australian code is very similar to the Canadian Code for the ULS approach. For SLS conditions, the Australian code applies a series of factors to modify the load and resistances and also includes the drag force as a load. It is an approach that I find complicated and difficult to grasp. However, the Australian code does allow the alternative design for SLS condition of using the more straight-forward settlement analysis and neutral plane consideration as outlined in relation to Figure 2 and recommended by the Canadian Code.

4.5 The Japanese Codes

Japan has four separate codes: the Road Association code, the Architecture Association code, the Port and Harbor Research Institute code, and the Railway code. All apply working stress design and make no distinction for the load from the structure being dead load or live load. The approach in Japan appears to be a partial factor of safety approach with different factors for shaft and toe resistances. The compilation in Table 2 shows the sum of the so-reduced resistance values for the modified example. Where the sum is larger than 800 kN, the design is acceptable by the particular code. These values are underscored.

TABLE 2 Compilation of the example applied to the Japanese codes

Code	F. O. S.		Sum of Factored Resistances				
			Short term		Long-term		
	Shaft	Toe	Theo.	Test	Theo.	Test	
Road Assoc.	3.0	4.0	610	560	710	640	none is acceptable
Architecture Assn	3.0	3.0	715	665	<u>815</u>	755	one is acceptable
Port and Harbor[1)]	>2.5	>2.5	<u>860</u>	<u>800</u>	<u>980</u>	<u>910</u>	all are acceptable
Railway	2.4	5.0	620	560	750	670	none is acceptable

1) I assume that for cases where uncertainty exists, the Port and Harbor increases the required FOS.

The Japanese codes do not specifically indicate that drag force and live load are combined and that only the resistance below the neutral plane should be considered as supporting the pile, but it is my information that the capacity above the neutral plane is disregarded. I have not included this in the above compilation table, however.

4.6 The Israeli Code

Israel has a Standard on Geotechnical Design (Israel 2008) that applies working stress design, assigning a global factor of safety of 3.0 to capacity based on theoretical analysis. The code does not mention drag force. However, when drag force is included in design, it is combined with dead and live loads. Drag force is generally considered only to develop in significantly settling soil layers. The contribution of the shaft resistance in those layers is not included in the capacity estimate.

The Israeli code results in the same factored resistances as the Japanese Architecture Association code.

4.7 The Singapore Code

The current code in Singapore is a working stress code (Singapore 2003). (Singapore intends to shift to a National Annex of the Eurocode in 2015). For design based on geotechnical strength, the current code applies a range of global factors of safety of 2.5 to 3.0. For capacity calculated using theoretical analysis, the practice has adopted to use a factor o f safety of 2.5. When capacity is separated on shaft and toe resistance, some practitioners apply a factor of 1.5 or 2.0 on the shaft resistance and 3.0 on toe resistance. The drag force is combined with the dead load and "sustained live load" (loads such as furniture and books, etc., that are removable but will be kept in place for a long time), but does not combine it with "transient" load, such as wind loads and traffic loads. However, the shaft resistance in layers generating drag force is not included in the capacity estimate, that is, only the "below NP" resistance is considered. Calculations based on the Singapore code would show the design of the example pile to be unacceptable.

4.8 The Indian Code

The Indian Code (2006) applies working stress design, assigning a global factor of safety of 2.5 to capacity based on theoretical analysis. No distinction is made between dead or live load. The same factor is applied to shaft and toe resistances. The code does not include provisions for drag force. However, practice is to subtract drag force from the capacity and then divide the balance with the factor of safety to obtain the allowable load. Again, calculations based on the Indian code would show the example pile to be inadequate.

DISCUSSION AND CONCLUSIONS

The design for geotechnical strength, the ULS condition, addresses a non-stationary failure process—the pile is moving down relative to the soil. By applying a factor of safety, or load and resistance factors to increase load and reduce the resistances, the designer ensures that the design has backed-off sufficiently from the possibility of the ULS condition. The premise is still that the pile would be failing! To include drag force in this scenario is a violation of principles because the pile approaches a failure condition, there is no longer any drag force present. To yet include it, perhaps defended by saying that "in a negative skin friction scenario, it is good to have some extra margin", is nothing other than design by ignorance. Why not instead boost the load factors and reduce the resistance factors? That would at least aim the ignorance toward the correct target.

The fact is that the phenomenon of negative skin friction, NSF, resulting in drag force plus dead load in balance with positive direction forces occurs for every pile—eventually. In ultimate limit states, ULS design, whether the settlement is small or large, the NSF issue is limited to checking the adequacy of the pile axial strength, which could be a deciding factor for sites where the depth to the neutral plane is more than about 80 to 100 times the pile diameter. Design of single piles and small pile groups must include assessing the expected settlement of the soil surrounding the pile and the downdrag of the pile, i.e., the settlement of the soil—and the pile—at the neutral plane in serviceability limit states, SLS. Indeed, for serviceability design, be the pile long or short, therefore, the issue is the downdrag, not the drag force. For pile groups, the settlement of the soil layers below the pile toe levels may show to be critical.

Addressing the ULS design for a NSF issue is not modeled by adding the drag force to the load from the structure. If a calculations model does not relate the depth to the neutral plane to a pertinent force equilibrium, the model would have little relevance to the actual conditions. Moreover, the tendency for many is to assume that the drag force only develops in soil that settles significantly in relation to the pile—a limit of 10 mm is often mentioned. Thus, the analysis returns a drag force conveniently small and of little bearing (pun intended) on the design calculations. In reality, long, mainly toe-bearing piles, even in soil exhibiting settlement much smaller than 10 mm, will be subjected to large drag force. When the correct drag force and location of the neutral plane are applied, adding the drag force to the loads from the structure will result in a mechanically impossible design.

The serviceability, SLS, design must be based on a settlement analysis incorporating the pile (or piles or pile group) response to unfactored loads and unfactored responses of primarily the pile toe and the settlement of the soils as affected by the stress changes at the pile location. For a margin to represent uncertainty, the design can apply a pessimistic approach to compressibility of the soil used in the settlement analysis and the estimate of the stiffness response of the pile toe.

My brief review of the various codes has demonstrated that there is a large lack of consistency in our practice for determining what really is the capacity of the pile. Yet, the practice seems to treat capacity as an assured number, proceeding to specify decimals for the various factors with no respect to how capacity was determined, the extent of the soils investigation, the number of static tests, the risks involved (i.e., the consequence of being wrong), the change with time, etc. The reviewed codes do either not address settlement of piled foundations or address them only very cursorily. The practice seems to assume that if the capacity has "plenty of FOS", or similar, the settlement issue is taken care of. This is far from the truth. I personally know of several projects where capacity was more than adequate with regard to geotechnical strength—the literature includes several additional cases—yet, the foundations suffered such severe distress that the structures had to be demolished.

A major weakness of most codes is that they refer to a "capacity" without properly defining what the capacity is, or not defining it by an acceptable method. Capacity related to the pile diameter is at best quasi. Just imagine two piles, one with a small diameter and one with a large, each subjected to a static loading test that shows exactly the same load-movement curve for the piles. Applying a definition based on the pile diameter would result in the curves being interpreted as the two piles having different capacities.

The movements measured in a static test are from 'elastic' compression of the pile (shortening), from build-up of shaft resistance that may exhibit an ultimate—plastic—response, but more often a response that is either post-peak-softening or a strain-hardening, and from pile toe movement increasing as a function of the pile toe stiffness similar to Eq. 1. There is no ultimate resistance for a pile toe! Indeed, the search for a pile capacity definition is charged with modeling the response to load by an elastic-plastic condition, when two of the three components definitely do not exhibit anything remotely like an elastic-plastic response and the third only rarely so.

As if the difficulty in choosing a suitable definition of capacity by itself would not cause enough uncertainty for applying the ULS code requirements, the practice employs a variety of definitions ranging from the Offset limit to the Chin-Kondner extrapolation (Fellenius 2013). Basing a design on geotechnical strength—the capacity—, be it by theoretical analysis or interpretation of results from a static loading test, is fraught with large uncertainty, hardly covered by the relatively small range of suggested factors of safety or resistance factors.

In answer to the requirement of the ULS condition, I prefer to recognize that what the structure supported on the piles is concerned with is the movement or settlement of the of the pile head, which is governed by the movement of the pile toe and settlement at the pile toe level, not by the shape of the load-movement curve or a value based on a pile diameter. The analyses leading up to assessing the SLS condition is the key to a successful design. Or more simply put: a large factor of safety does not ensure that the settlements will be small. However, an SLS analysis showing the settlements to be small does ensure that the capacity of the pile(s) is adequate. I am not suggesting we cease carrying out a ULS analysis, but we definitely need to improve how we do it and we need to pay more attention to the SLS.

Acknowledgements

I gratefully acknowledged comments received from my colleagues David Klingberg, Julian Seidel, and Stephen Buttling (Australia), Brian Simpson (UK), Dan Brown, Naresh Samtani, and Jon Bishop (USA), Gandhi, S.R. and Rama Krishna (India), Joram Amir (Israel), Tatsunori Matsumoto (Japan), and Harry Tan and Hartono Wu (Singapore). Inasmuch any of my brief accounts of the various codes is incorrect, this is certainly due to my interpretation of the codes and comments and not to incorrect statements received.

References

AASHTO 2012. LFRD bridge design specifications, Section 10 Foundations. American Association of State Highway Officials.

Canadian Standard Council, 2006. Canadian Highways Bridge Design Code, Section 6, Foundations. Canadian Standard Association, CSA-S6-06, Code and Commentary, 1,340 p.

Federal Highway Administration, FHWA, 2006. Design and construction of driven pile foundations. Volume 1 and II. US Dept. of Trp., Publ. No. FHWA NHI-05-042, 1,462 p.

Eurocode 1997. EN 1997-1. Geotechnical Design, Part 1: General Rules.

Fellenius, B.H., 1999. Bearing capacity — A delusion? Deep Foundation Institute, Hawthorne, NJ, Proceedings of Annual Meeting, Dearborn, Michigan, October 14 16, 1999, 17 p.

Fellenius, B.H., 2013. Capacity and load-movement of a CFA pile: A prediction event. ASCE GeoInstitute Geo Congress San Diego, March 3-6, 2013, Foundation Engineering in the Face of Uncertainty, ASCE, Reston, VA, James L. Withiam, Kwok-Kwang Phoon, and Mohamad H. Hussein, eds., Geotechnical Special Publication, GSP 229, pp. 707-719.

Fellenius, B.H., 2014. Basics of foundation design. Electronic Edition. www.Fellenius.net, 410 p.

Frank, R., Baudin, C., Driscoll, M., Kavvada, M., Krebs Ovesen, N., Orr, T., and Schuppener, B., 2004. Designers' Guide to EN 1997-1. Eurocode 7: Geotechnical Design, General Rules. Thomas Telford, 216 p.

Goudreault, P.A. and Fellenius, B.H., 2013. UniPile Version 5, Users and Examples Manual. UniSoft Geotechnical Solutions Ltd. [www.UniSoftLtd.com]. 108 p.

Indian Code, 2006. Indian Code of Practice for Design and Construction of Pile Foundations IS:2911 1980, revised 2006, 38 p.

Israel 2008. Geotechnical Design Standard IS940 Part 1. Personal communication with Joram M. Amir, Israel, 13-09-28.

Likins, G.E., Fellenius, B.H., and Holtz, R.D., 2012. Pile Driving Formulas—Past and Present.. Full-scale Testing in Foundation Design, M.H. Hussein, R.D. Holtz, K.R. Massarsch, and G.E. Likins, eds. ASCE GeoInstitute Geo Congress Oakland, March 25-29, 2012, State of the Art and Practice in Geotechnical Engineering, ASCE, Reston, VA., Geotechnical Special Publication, GSP 227, 737-753.

O'Neill, M.W. and Reese, L.C., 1999. Drilled shafts. Construction procedures and design methods, Federal Highway Administration, Transp. Research Board, Washington, FHWA-IF99-025.

Poulos, H.G., 2013. Pile design for ground movement. Proc. of the Int. Conf. on State-of-the-Art of Pile Foundations and Case Histories, Bandung, Indonesia, June 2-4, pp. A2-1 - A2-18.

Simpson, B. and Driscoll, R., 1998. Eurocode 7A Commentary. Construction Research Communications, Watford.

Singapore 2003. Singapore Standard Code of Practice for Foundations, CP4:2003. 253 p.

Terzaghi, K., 1942 Discussions on the Progress Report of the Committee on the Bearing Value of Pile Foundations. Proceedings of the American Society of Civil Engineers, February, pp. 311-323.