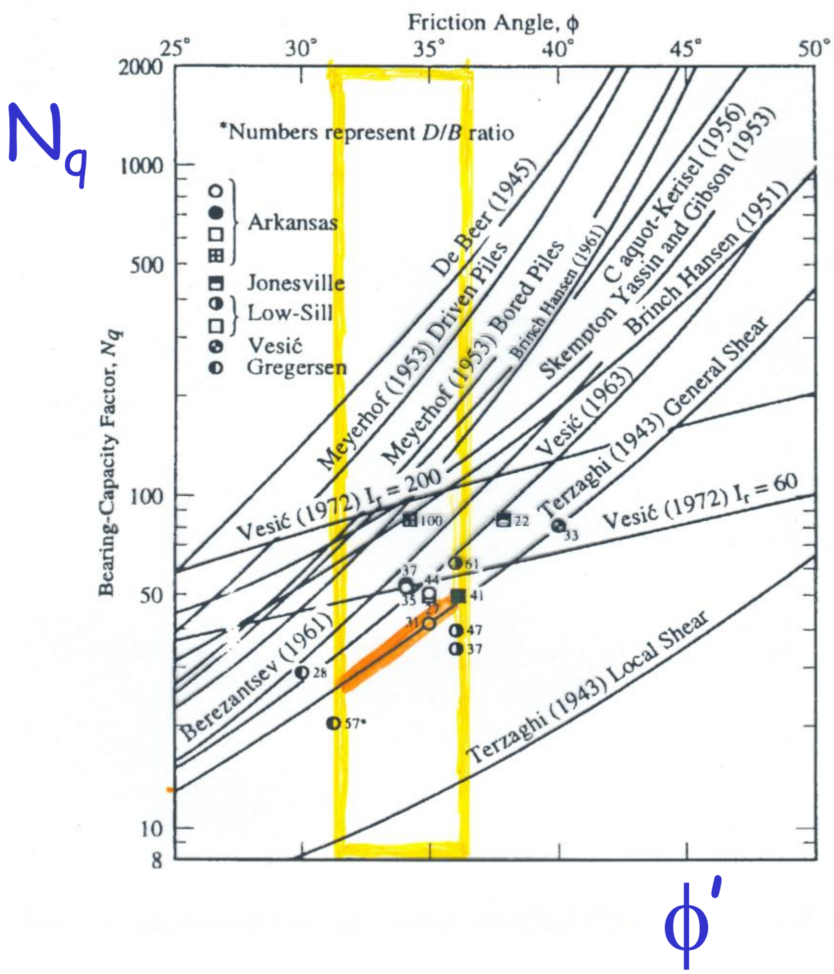

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Pitfalls and Fallacies in Foundation Design

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Abstract

Design of the load to be placed on footings and piled foundations conventionally includes calculated bearing capacity—a peak, or ultimate, resistance—usually defined as a plastic, or semi-plastic, response to increased load. Such bearing capacity can be found in loading tests on model footings, yet no full-scale tests on footings have show that an ultimate resistance mode has developed unless the load was placed off center—intentionally or not. For pile design, it is recognized that the pile toe is a footing with a long stem and none of the very large number of full-scale tests performed with the pile toe response measured separately from the shaft response, has shown a toe bearing capacity. Instead, the toe responses were similar to those found in tests on full-scale footings, i.e., a gradual, though less than linear, and increase of movement for increasing load.

Pile capacity is usually established (interpreted) from the pile-head load-movement of full-scale tests on single piles. However, the design analysis of the response of a pile to load is usually based on modeling a pile as a series of short elements, each with its ultimate resistance, or peak resistance. In most cases, the shaft resistance for each element beyond that peak is either strain-hardening or strain-softening. Therefore, as demonstrated by examples, unless the pile is very short or next to infinitely stiff, the accumulated value of the ultimate shaft resistances of the element is not equal to the pile capacity established from the measured load-movement curve of the pile.

Moreover, for pile groups comprising more than about five by five rows of piles, i.e., wide piled foundations, the maximum shaft resistance of the piles in a group is limited to the weight of the soil in-between the piles, and the response of the piles differ between interior and perimeter piles. Design of wide piled foundations and piled rafts must be based on settlement analysis and no contribution from contact stress can be assumed.

INTRODUCTION

In every science-oriented set of know-how, such as geotechnical engineering, there is a set of concepts held as true, never questioned, only amended and developed within the original framework. In a sense they are what Richard Dawkins (Dawkins 1976) called "memes", that is, self-replicating concepts, ideas, or styles that spread from person to person within a culture. The first such that comes to mind in foundation design is "capacity", the short-term for "ultimate resistance". Geotechnical text books addressing foundation design, devote much space to strength and ultimate resistance of samples and soil elements. No textbook omits presenting the Terzaghi triple-N bearing capacity formula (Equation 1 and Figure 1), which Terzaghi originally published in 1943, basing the theory on results of laboratory tests on small diameter plates.

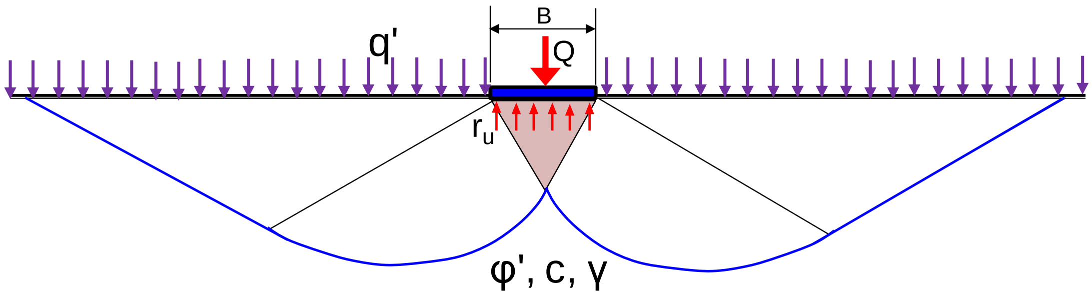

Figure 1 Background to the triple-N formula

(1) $$r_u = c' N_c + q' N_q + 0.5 B \gamma' N_\gamma$$

where	r_u	=	ultimate unit resistance of the footing
	c'	=	effective cohesion intercept
	B	=	footing width
	q'	=	overburden effective stress at the foundation level
	γ'	=	average effective unit weight of the soil below the foundation
	N_c, N_q, N_γ	=	bearing capacity factors

All building codes and standards addressing foundations indicate factors of safety to apply to the capacity of footings or piles to establish allowable load—these days, many also indicate resistance factors or partial factors of safety to apply to the capacity in order to establish factored resistance. The particular capacity is commonly determined based on simple soil mechanics principles or routine type of full-scale tests (then, mostly in the context of piles). Little space, if any, is given to the response to load in terms of movement and settlement. The paradox in our profession is the fact that a "safe" foundation design means ensuring that the soil forces calculated for the conditions of "capacity " will never occur, while little or no consideration is given to the soil forces, movements, deformations, and settlements that do occur for the actual conditions. As long as this bizarre situation prevails, we will never be able to advance our design beyond the level of an educated guess.

The first issue of a design should be "***will the short-term and long-term deformations and settlements be acceptable to the supported structure?***" and our practice should shift to this approach. I am sorry to say that establishing deformation as the primary design principle does not seem to be in the current collective geotechnical mind. For the purpose of convincing at least some of my colleagues of the absurdity of our current design approach, I will indicate a few of the foibles in the current "memes".

CAPACITY AS TAUGHT

Every text book defines "capacity" as a response to a movement along a shear plane culminating in the shear strength (ultimate friction or cohesion, tan φ' or c') being mobilized whereafter the soil enters a plastic state. That is, "capacity" is defined as an ultimate state, for which, once developed, adding load does not increase the resistance but simply results in additional foundation movement. This state is expressed in the Terzaghi triple-N formula (Equation 1) and Figure 1.

In practice, the formula is applied with an array of modified N-factors aiming to achieve a higher level of sophistication—for example, adjustments to friction-angle with reference to soil mineralogy, gradation, roundness, not to forget preconsolidation and cohesion relations. To verify whether or not the formula has relevance to actual response, requires comparing calculated capacity to actually observed responses, i.e., full-scale test records.

Figure 2a presents measured load-movement responses from static tests performed more than 30 years ago (Ismael 1985) on four square footings with sides of 0.25, 0.50, 0.75, and 1.00 m at a site. The footings were placed 1.0 m into a fine sand and the depth to the groundwater table was 2.8 m. The sand was compact, as indicated by an N-index equal to about 20 blows/0.3 m. Figure 2b shows the same data plotted as stress versus relative movement, i.e., the measured movement divided by the footing side. Notice that the curves are gently curving, having no break or other indication of failure—capacity—despite relative movements as large as 15 % of the length of the footing side. Thus, there is no capacity shown in the tests to compare to the calculated value!

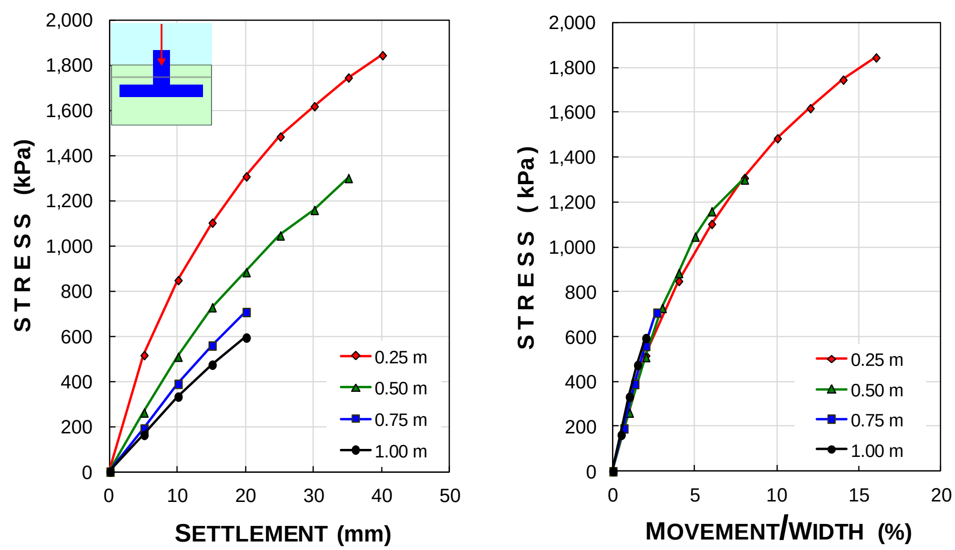

Figure 2a. Footing-test load-movement curves Figure 2b. Normalized curves (Fellenius 2006; 2017a. Data from Ismael 1985).

Briaud and Gibbens (1999) presented load-movement records from tests on five square footings in sand. Figure 3 shows the measured results as stress versus relative movement. Again, no change in response is noticeable that could be used to define a capacity. The figure shows a fit of the load-movement records to a q-z function according to the Gwizdala (ratio) relation (Fellenius 2017a). They could also have been fitted to a conventional settlement analysis with input of a suitably chosen preconsolidation stress and virgin and reloading moduli.

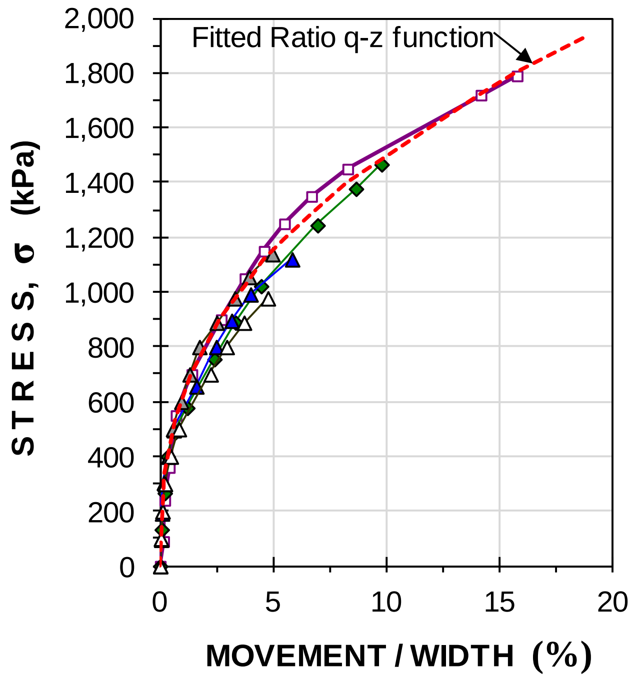

Figure 3. Normalized load-movement curves (data from Briaud and Gibbens 1994)

Results from many other similar footing tests have been published, e.g., Ismael (1985), Kusakabe et al. (1992), and Akbas and Kulhawy (2009). Not one has shown test curves that could reasonably and rationally be used to define a footing "capacity" as an ultimate resistance—the soil having entered a plastic state. The fact is, there is no such thing as a footing capacity. N.B., when the applied load is inclined and/or not applied at the center of the footing, tilting followed by collapse may occur, but it is then due to rotation ("overturning"). Thus, Figure 4 shows that when the resultant moves outside the middle third area of the footing (assumed rigid), the distribution of stress underneath the footing ceases to be linear and the point of rotation moves inward from the outside edge. That the safety against rotation would be determined for a rotation around the outside edge, is an additional fallacy found in many textbooks, codes, and standards.

In short-term loading of clays, the imposed loading is often rapid enough to increase the pore pressures, i.e., the state of effective stress changes and a collapse may follow. For long-term loading of a preconsolidated clay, the short-term loading generates negative pore pressures and collapse may occur when the pore pressures return to normal values.

A pile toe is in principle a buried footing with a long stem. Since the advent of the bidirectional pile test method (Elisio 1983, Osterberg 1989), numerous full-scale pile tests have been performed that include toe-response load-movement curves and none has demonstrated a definite toe capacity.

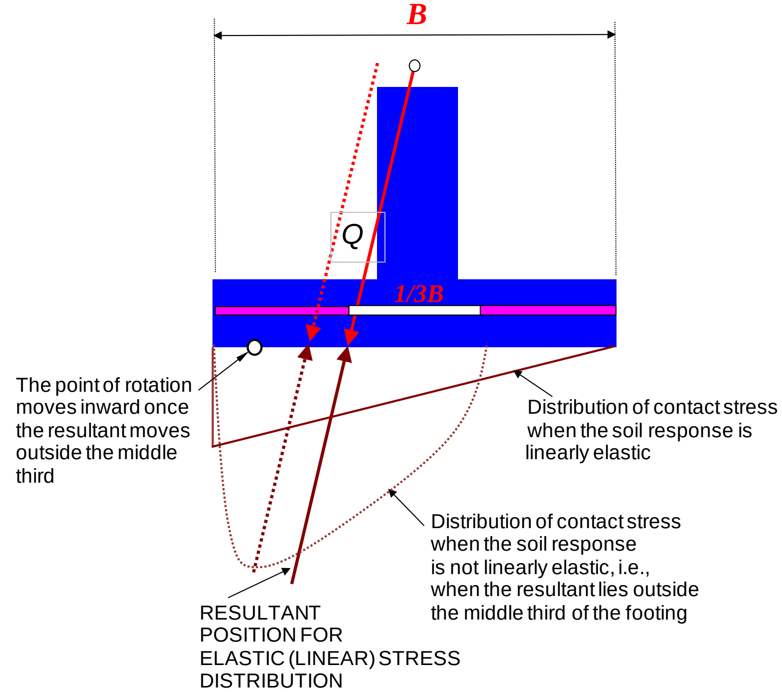

Figure 4. Forces against a footing with the resultant located inside and outside the middle-third (Fellenius 2006; 2017a).

In contrast to pile toe resistance, the shaft resistance along a pile element can actually develop a plastic response. However, most of the time, the shaft resistance response is by strain-hardening or strain softening. Fellenius (2017a) has indicated several mathematical relations of unit shear/stress-movement response of shear or stress to load. Figure 5 shows six different curves, so-called t-z functions, going through a common target point at 100-% stress or load at a specific target movement. The, as commonly assumed, ideally elastic-plastic response implies a sudden kink at the transition from elastic to plastic state is an unnecessarily simplified approach that can be replaced by the Van der Veen curve. This curve is elasto-plastic with an almost straight line shifting to a horizontal line, i.e., a plastic response, at the target, but the shift is continuous without a sudden kink.

The hyperbolic curve (the lower of the two "strain-hardening" curves) is often thought to be suitable for simulating plastic shaft resistance response. However, it only becomes approximately plastic after a long movement beyond the target point. (Note, the "movement" is not a slippage, i.e., definite sliding of the pile element against a stable body of soil, but occurs as a shear deformation within some zone or band of soil next to the pile element. Therefore, the movement along the side of the element is the relative movement between the pile element surface and a outer boundary or the shear zone, a somewhat undefined location, actually).

The toe resistance is rarely other than a "strain-hardening" curve and is best modeled by the Gwizdala (ratio) function. Of course, the target in Figure 5 is an arbitrary choice. It can range widely. In fact, were all actually possible curves plotted in the figure, there would be no color white between the curves.

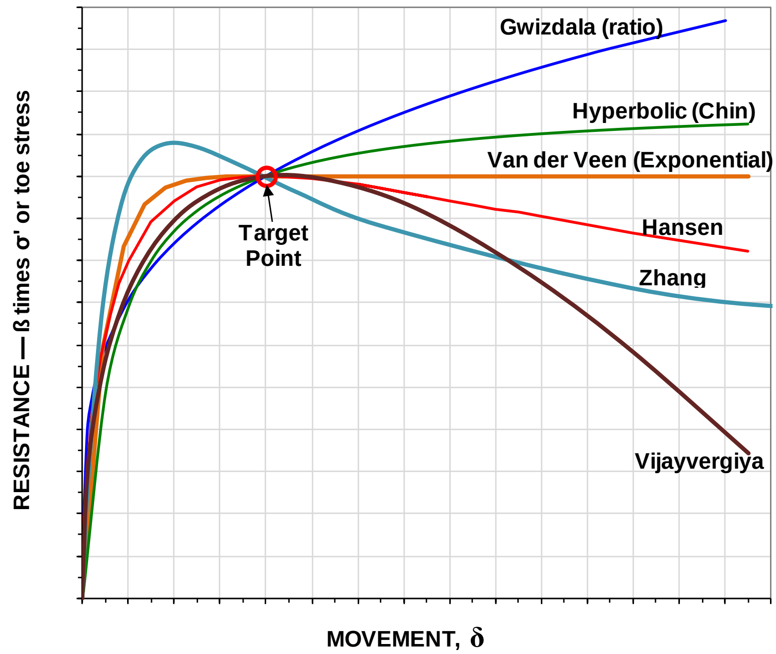

Figure 5. Typical t-z and q-z curves (Fellenius 2017a).

Each strain-softening t-z curve has a peak, which is an obvious target for "capacity" of the particular pile element. However, it should be recognized that the capacity of the pile is not the sum of all the capacities of the individual pile elements, but of the sum of the resistances having been mobilized at the particular pile head movement, a common fallacy. Figure 6a shows load-movement curves from a hypothetical static loading test on a typical pile in a uniform soil. The curves for the shaft resistance response and the toe response are also shown. (No load labels are included, only movements). As indicated, the pile shaft resistance is strain-softening (the pile toe response is strain-hardening, of course). Most engineers would interpret the test results to a pile capacity equal to the load at the peak load of the test.

The pile is assumed to be instrumented with gages measuring the axial load and movement at three depths and at the pile toe. Figure 6b shows the distribution of force and movement along the pile when the maximum load was reached in the test. However, only one or a few of the pile elements were at a peak-resistance state. The elements in the upper part of the pile were at a post-peak state and the elements closer to the pile toe were at a pre-peak state. The pile toe had hardly begun to move and the mobilized toe resistance was small. The figure shows that whatever the definition of pile capacity applied to the pile-head curve, it will not match the ultimate resistance defined or chosen for the individual pile elements. A routine back-calculation of the test results is likely to arrive at an incorrect understanding of the actual response of both the pile shaft and the pile toe. Applying this "understanding" to the design of smaller or larger, shorter of longer piles at the site is then not likely to be correct.

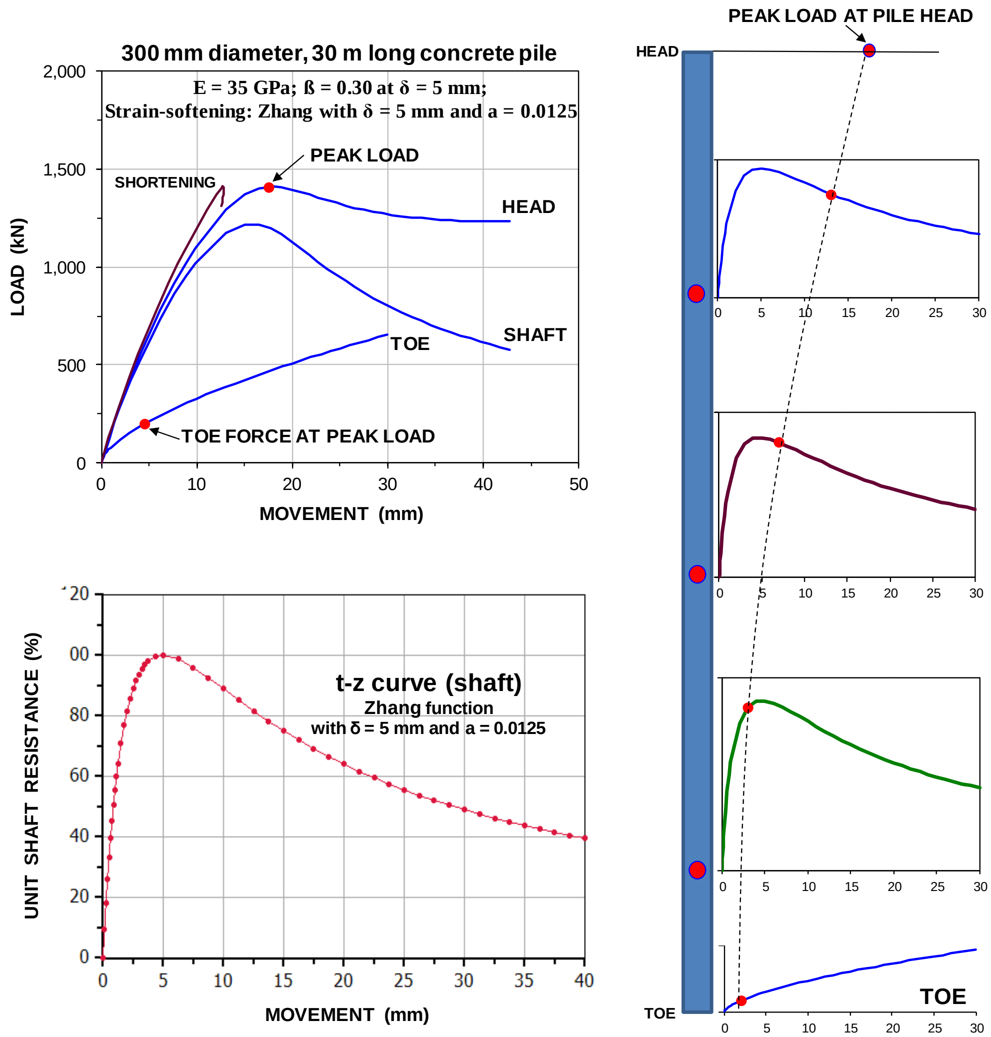

Figure 6 Comparison between load-movement curves and t-z curves.
6a. Load and unit shaft resistance vs. relative movement.
6b. Load-movement distributions with depth.

If the shaft resistance had been a mix of strain-hardening and strain-softening load-transfer curves at different depths, the conclusion would not have been any different and this is irrelevant to the test showing a peak resistance or not. Of course, without a clear cut peak resistance, the person evaluating the test records would have had to rely on a definition of "capacity" of which there are many in use. In North America, the Davisson offset limit is common, whereas, in Europe, the EuroCode applies the so-called Terzaghi 10-% of the pile diameter (originating in a misconception of Terzaghi's 1943 recommendation), while the hyperbolic t-z function and the infinite-movement load (Chin-Kondner extrapolation) as pile capacity is in widespread use in South-East Asia. Recognizing that the 10 % of a large diameter pile is a rather large movement, some cuts that in two and apply a 5-% limit, instead. For details of these and other definitions, see Fellenius (2017a). In seeing how imprecise a value of capacity one obtains from a static loading test (addressed next) or a theoretical calculation, it is strange to see that the profession does not worry more about the proper performance—i.e., serviceability of the designed foundation—and that so many can devote time and energy to discuss whether or not the safety factor on tested pile "capacity" should be 2.0 or 2.2, or the resistance factor be 0.65 or 0.70.

Additional factors, affecting the response of the pile to an applied load, are residual forces. Such forces will affect the stiffness of a pile response and, therefore, the interpretation of the test results (Fellenius 2006; 2017a).

CAPACITY AS ASSESSED IN PRACTICE

Every now and then, the organizers of a deep foundation conference will add a bit of spice to the event by arranging for a static loading test to be carried out in connection with the event, inviting participants and others to predict the pile capacity. N.B., the predictions will then be true, that is, be as the word implies, made before the test takes place. It is a bit of a roulette game, as a prediction will rarely be close to the actual results unless the predictor has access to prior results from previous piling work in the area or, at least, from experience in the particular geology. In contrast, nobody would commit to finalizing an actual design without that more intimate or reference information.

However, participating in a prediction carries no risk other than to one's pride. I have enjoyed participating in many predictions event and arranged a few. For example, in 2011, I solicited predictions of results from a static loading test on a 406-mm diameter, 18.5 m long CFA pile in stiff clay (Fellenius 2013). And did so again in 2013 for predictions of the results of a test on a 400-mm diameter, 17.5 m long bored pile in silty sand tested at the 1st International Bolivian Deep Foundation Conference (Fellenius and Terceros 2014). Both invitations requested the participants to submit a predicted pile-head load-movement curve and, then, on that curve to indicate the capacity they would consider their predicted test curve to show the pile to have. The predictions and the capacity assessments are compiled in Figures 7 and 8.

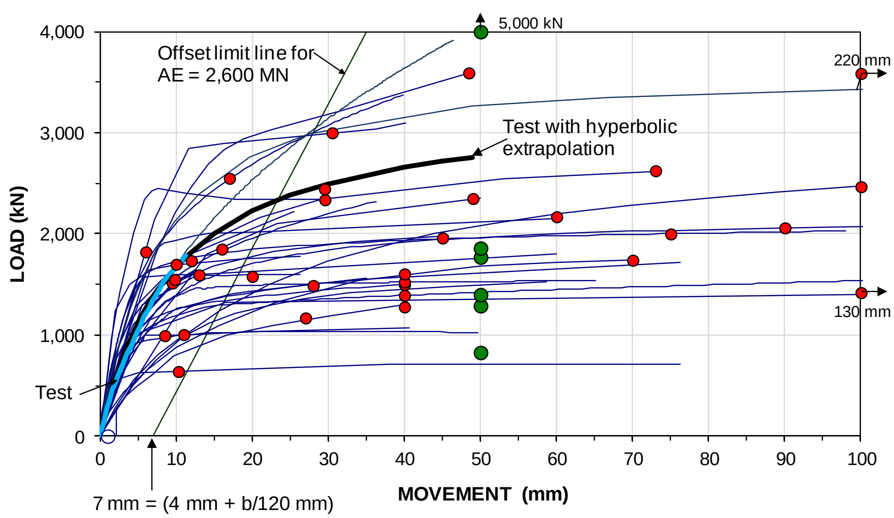

Figure 7. Predicted load-movements and assessed capacities for a pile in clay (Fellenius 2013).

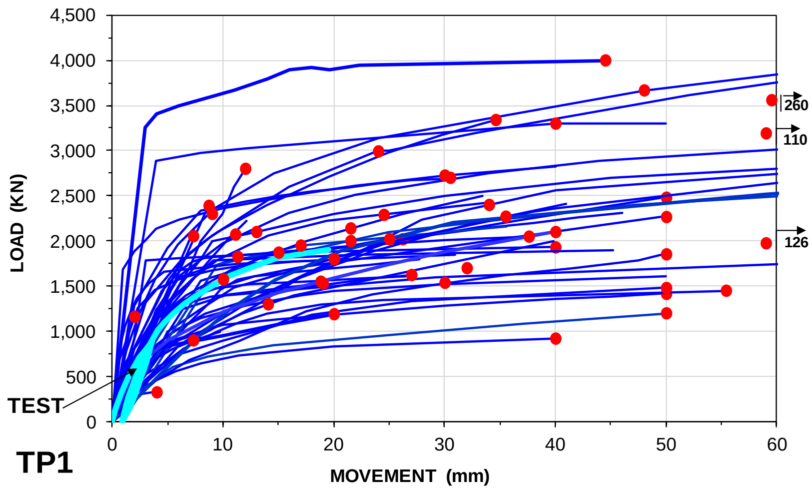

Figure 8. Predicted load-movements and assessed capacities for a pile in silty sand (Fellenius and Terceros 2014).

The two events attracted different groups of people and, but for one or two participants, the soil and geology were unfamiliar to all and nobody had prior experience of the response of other piles tested in either area. Although, the majority of the participants were well versed in pile design and analysis, it is no surprise that the predicted curves deviated considerably from each other. As happens in most random events, the actual response lies about in the middle of the predicted responses. However, the difference between the load-movement curves is not what's remarkable in the figures, it is the scatter of assessed capacities. Note, in contrast to the load-movement curves, the capacities were not predictions, but assessments, each based on the participant's preferred method of determining a capacity from a pile-head load-movement curve. Both events showed that the profession practices a wide range of methods to assess pile capacity from a pile-head movement curve.

A prediction event was organized by the Universidade Federal do Rio Grande do Sul in the Araquari Experimental Testing Site, Brazil in 2015 and comprised a 1,000-mm diameter, 24 m long bored pile in sand. The premise of the prediction was that the test be carried to a final movement of 100 mm, 10 % of the pile diameter (the the organizers defined pile capacity as load for this movement). The task was to predict the pile-head load-movement curve for the test pile up to that pre-determined capacity. To remove the prediction aspect from the results, after the prediction results had been published, I contacted all predictors and asked them to tell me, using their own definition, what capacity they would assess the test pile to have based on the actual test curve. Twenty-nine, about half of the total replied, Figure 9 compiles the capacities received. (The test included an unintentional unloading and reloading step). In contrast to the two previous results, the assessment was for an actual test curve common to all. The values diverge considerably. Seven accepted the organizers' assertion that the capacity was the load that gave a movement equal to 10 % of the pile diameter, whereas the others indicated values that were as low as two-thirds of the maximum with a 21-mm movement, as opposed to the 100 mm value stipulated by the organizers.

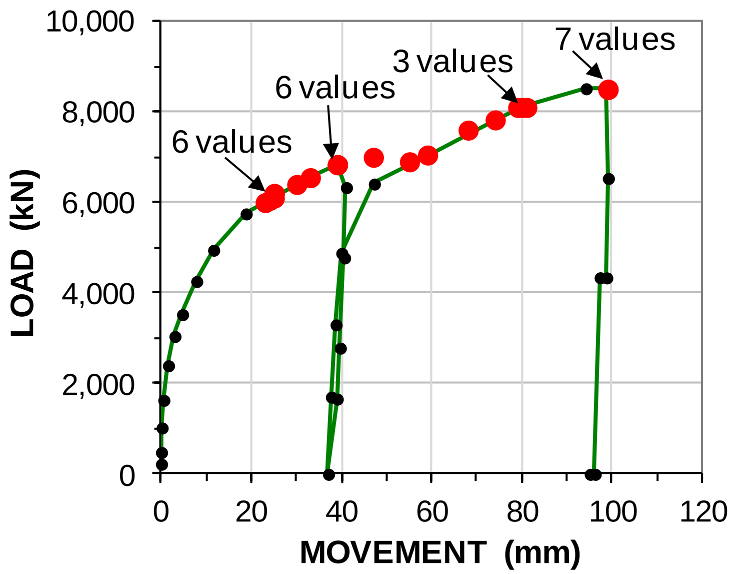

Figure 9. Test results and capacities assessed by 29 predictors for the Araquari prediction case.

Although only a few of the capacity assessments for the three prediction events are from people who participated in all three events, the number of people participating in the three prediction events is small. Earlier this year, I arranged a prediction event for four test piles that attracted a large number of participants and, like before, in disseminating the results of the actual tests, I asked the participants to assess the capacity of the test piles. As many as 94 provided assessments. Figure 10 compiles the values plotted one of the actual test curves, the results of a 450-mm diameter, 9.5 m long CFA pile installed in a silty sand. The results show that the profession indeed have very differing views on what a capacity is and applies different methods in determining is from full-scale test records. Such inconsistency within the profession is not safe. Moreover, it is also costly.

When the practice developed the approach of basing a design on a factor of safety on capacity, however assessed, much uncertainty existed with regard to how to analyze settlement. That situation—and excuse—is is no longer valid. The current state-of-the-art is quite explicit on how to calculate the settlement of foundations, be they single footing, single piles, or large group of footings or a raft, or narrow or wide piled foundations. That the industry often neither undertakes determining the site and soil information necessary for a settlement analysis, nor carries out the analysis is rather unsatisfactory.

Moreover, there are still people and standards considering that a drag force—calling it "drag load"—constitutes a load on a pile similar to the load from the structure, as opposed to it being an environmental force of concern for the axial structural strength of the pile, only. The same people disregard the downdrag often associated with the situation—the real problem. However, this is a case of ignorance and not a fallacy.

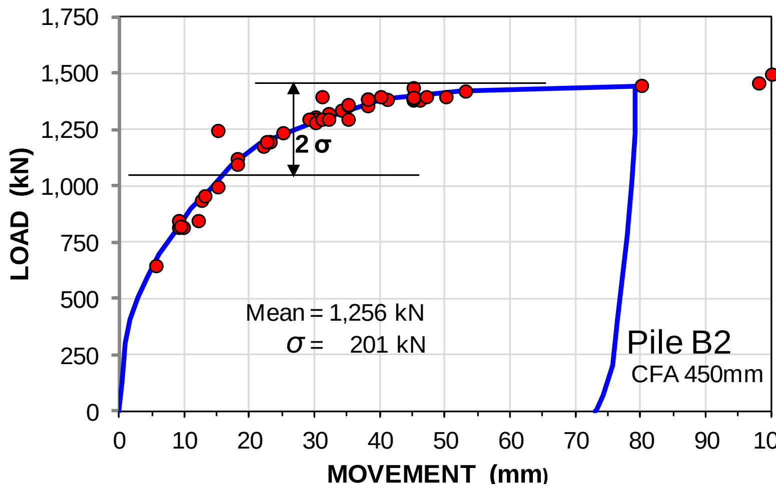

Figure 10. Test results and capacities assessed by 94 predictors for the 3rd CFPB prediction event (Fellenius 2017b).

ADDITIONAL ASPECTS

Capacity is just one of the many uncertainties involved in assessing the results of a static loading test. Since analysis of full-scale test results were published by Hunter and Davisson (1969) and Gregersen et al (1972), it has been known that the load distribution determined from measurements of strain-gage instrumented piles is frequently affected by axial force locked into the test pile before the start of the test, called "residual force". When not considered in the analysis of the test records, fallacies such as "critical depth" ensue, leading to incorrect interpretations that adversely affect the application of the test records to the particular case of the test and beyond. Today 50 years after the profession was first made aware of the fallacy, we still see test results presented that are clearly affected by residual force in the test pile, such as shown in Figure 11 from the "General Report on Design Methods Based on Static Pile Load Tests" in the Proceedings of ETC3, Symposium on Design of Piles in Europe, in Leuven, Belgium, April 2016. The load distributions show small or next to zero shaft resistance along the lower length of the pile despite the soil there being denser than above as indicated by the CPT-diagram.

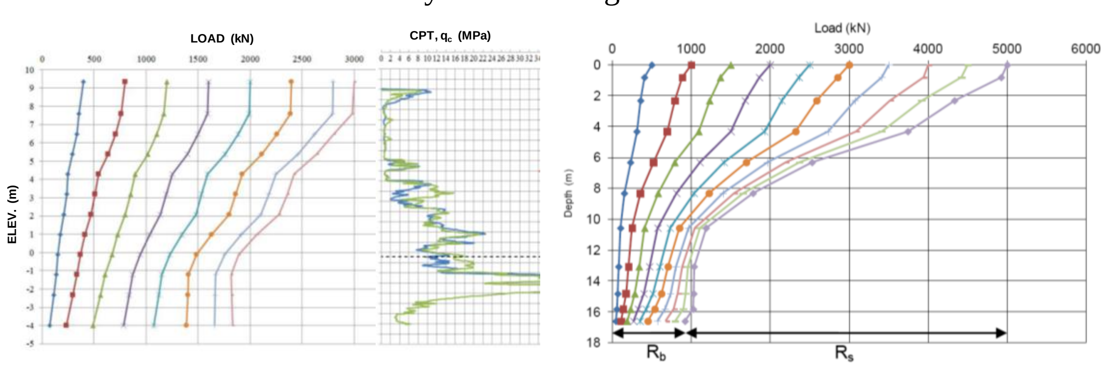

Figure 11. Load-distribution from a 508-mm diameter, 15 m long driven pile a 620 mm diameter, 16.6 m long screw pile.

Apart from the obvious misrepresentation of the soil response with depth, the presentence of residual load will have made pile-head load-movement curve appear stiffer than had there been no residual load, causing an overestimation of capacity by any method or definition.

A common approach in design of a foundation supported on a group of piles from results of a test on a single pile is to apply an "efficiency factor" smaller than unity. It is often assumed that the ability of the group to support a load is smaller than that of an equal number of single piles. It is usually expected that for dominantly toe-bearing piles, the efficiency factor could be close to unity, but for foundations supported on shaft-bearing piles, the factor would be a good deal smaller. I have seen several such opinions being referenced to work presented by O/Neill et al. (1982), who performed static loading tests on one single pile, a group of four piles, and a group of nine piles, all with the pile cap well above the ground surface. As illustrated in Figure 12, the test results showed that the pile head movement for the single pile was consistently smaller than that of the average pile in the four-pile group, for which, in turn, the pile head movement was smaller than that of the average pile in the nine-pile group. The inserted sketch suggests that the reason for the increase in movement for increased pile group width is due the fact that the stress-bulb below the piles became larger for each test and this resulted in compressions of the soil below the pile toe level as was manifested in larger movements. Note, the group piles are widely spaced; the center-to-center distance is ten pile diameters.

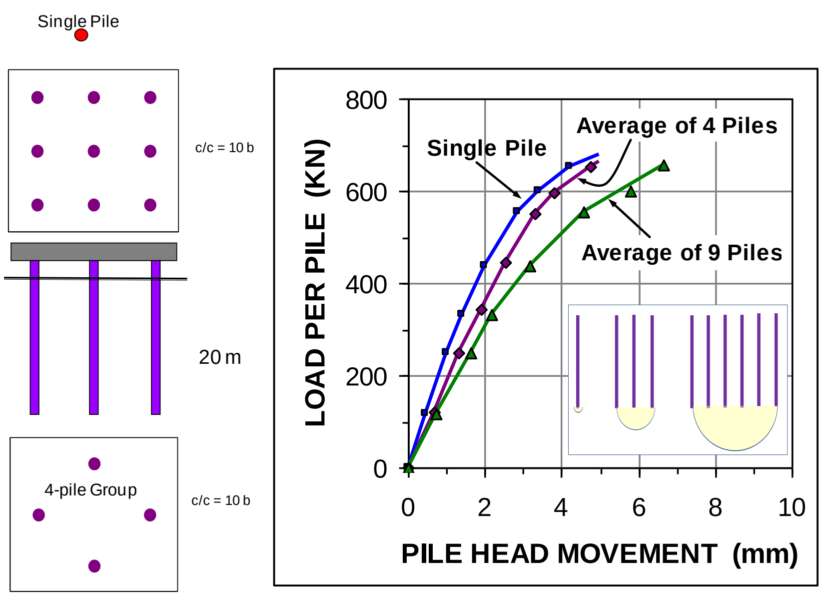

Figure 12. Pile-head load-movement from static loading tests on single pile, a four-pile group, and a nine-pile group (Data from O'Neil et al, 1982).

The response to load for a single pile is different to that of a group of piles, as will be addressed further down in this paper, but the "efficiency factor" approach is a fallacy.

THE ILLUSION OF CONTACT STRESS

It is often assumed that a pile cap in contact with the soil will add to the bearing by way of a significant contact stress. It originates in the concept of bearing capacity as the overriding issue for design of a foundation. However, when loading a piled foundation with a pile cap in contact with the ground, the cap will only experience contact stress below the perimeter overhang (area outside the perimeter piles) to an inward diminishing degree. In the short-term, this peripheral contact stress will reduce the imposed pile cap movement for a small (narrow) piled foundation, as illustrated in Figure 13. And, in case of a static test on the pile group to a perceived ultimate resistance, the cap will certainly increase the resistance along the edges in comparison to a cap with no soil contact. So, it would seem that when applying the same safety factor to the two inferred capacities, the cap in contact with the soil would take a larger allowable load. However, if the soil layers above the pile toe are even moderately compressible, in the long-term, the contact stress support would diminish and the final pile cap movement be determined solely by the pile response to the load (and that of the soil underneath the pile toe level).

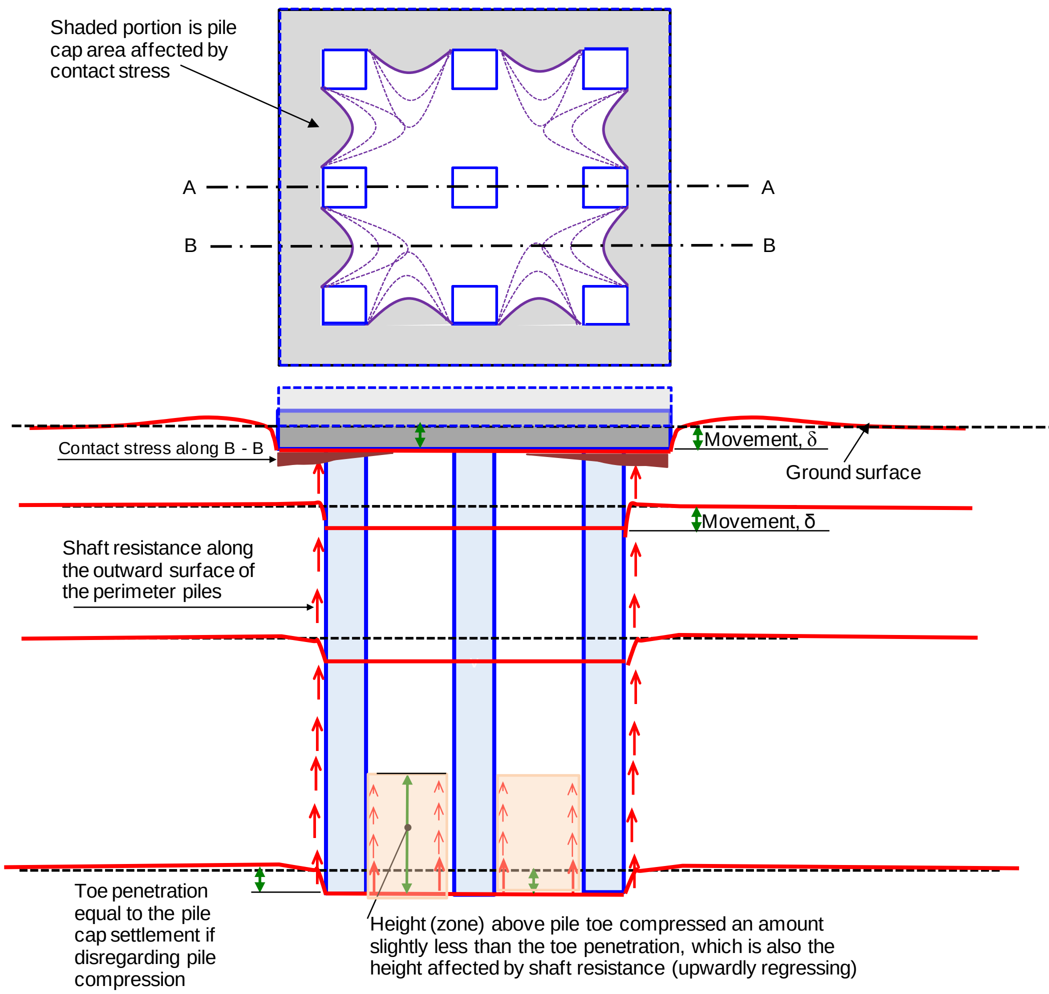

Figure 13. Distribution of contact stress and shaft resistance across a nine-pile cap. N.B., the pile cap perimeter overhang is exaggerated in the sketch.

When imposing a load onto a relatively rigid (from pile to pile) pile cap in contact with the soil, in the center of the pile group and immediately underneath the pile cap, the strain in the piles and the strain in the soil must be equal and so must be the movement of the piles and movement of the soil. The first realization is that, while the imposed load can develop a strain in the pile of about 100 microstrain as a broad number, that same strain in the soil—the strain cannot be different—represents a very small stress. This because the stiffness (modulus) of the pile is about three to four orders of magnitude larger than the stiffness of the soil. Thus, no significant contact stress will develop due to the load applied to the cap—all load on the pile cap will go to the piles.

Shaft resistance is the effect of a relative movement between the pile shaft and the soil and it is caused by the pile moving down in relation to the soil. For interior pile in a pile group, consider the relative positions of the pile and the soil at some depth down below the pile cap. What relative movement could develop there, when there was none at the pile head? Indeed, from the pile head and down to the vicinity of the pile toe, only a minimum of relative movement between the pile and the soil will develop and only minimal load or force can be transferred from the pile to the soil and vice versa.

Nature strives to transfer the load applied to the pile cap by means of a uniformly distributed strain across the footprint, and the pile and the soil having the equal strain means, that all the load is really carried by the piles. At the pile toe, however, there is a sudden change: no piles, only soil. Thus, immediately above the pile toe, the strain can no longer stay the same for the piles and for the soil in-between the piles. Up from the pile toe, the soil strain will therefore increase and the pile strain will decrease. Thus, the soil will appear to be pushed up along the pile (along a certain distance up from the pile toe) and, correspondingly, the pile toe will appear to be pushed into the soil. The relative movement between the pile shaft and the soil will result in build-up of shaft resistance. The length above the pile toe affected depends on the equilibrium developing between (1) the axial load in the pile, being reduced by the shaft resistance created by the upward movement of the soil according to the particular t-z relation of the case, with (2) the toe resistance determined by the pile toe q-z relation. If the pile toe is resting on a dense soil, already a small movement will build up a toe resistance almost equal to the load applied to the pile at the pile cap and shaft resistance along the interior piles will not develop until very close to the pile toe. If on the other hand, the soil below the pile toe level is loose or soft, the toe resistance might even become close to zero and all the load is carried by shaft resistance along the lower length above the pile toe to the height required. For either condition, an equilibrium is established between, on the one hand, the compression of the soil up from the pile toe and the so-developed shaft resistance and, on the other hand, the remaining load in the pile at the pile toe and the pile toe movement with the total soil compression (soil upward movement) equal to the pile toe movement.

N.B., for the special case, rather unrealistic, where the pile toe resistance is zero and the average pile load is larger than the ultimate shaft resistance of the single pile, the response of the piled foundation is that of a pier with minimal base resistance and a considerable shaft resistance along the perimeter of the pier.

Thus, a wide piled foundation (no toe resistance; "floating piles") will carry the applied load as shaft resistance along a lower length, maybe a distance up from the pile toe equal to one third of the pile length, depending on magnitude of the load and resistance. Of course, shaft resistance will develop along the full length of the perimeter piles. For the interior piles, all the load applied to the pile cap will reach the soil as a uniformly distributed stress that is best modeled as a flexible raft carrying all the applied load. In the short-tem the perimeter piles will carry larger load than the interior piles. As mentioned below, in the long-term, the perimeter pile will carry less load than the interior piles.

It is easy and not that time-consuming in a given case to determine the equilibrium condition of toe and shaft resistance and toe penetration in a trial-and-error analysis. The "given case" means that the soil parameters and the applicable t-z and q-z relations for the pile-soil condition near the pile toe along with the axial pile load are known. One can start by assuming a specific toe resistance (smaller than the axial pile load at the pile cap level) and determine the resulting pile toe movement. In a separate analysis, the response of the soil in-between the piles is modeled as a "soil-pile" with a cross section equal to the pile area divided by the pile footprint ratio and a compressibility equal to that of the soil, not of the pile. The circumferential shaft area of this "soil-pile" and its t-z relation needs to be input (this information would be the known data for the design case). The calculations of the response of the "soil-pile" will show the shaft resistance and the length of the "soil-pile" engaged due to a movement equal to the pile toe movement of the first calculation. After a couple of trials toward a conversion of movements, the calculated pile toe movement is equal to the upward soil movement. Then, the pile toe resistance is equal to the applied load minus the shaft resistance calculated for the "soil-pile" and the response of the pile group interior piles is determined. The key result is the toe movement, the "load-transfer" movement for the foundation. It will be the calculated toe movement for the final trial run and the equally large compression of the "soil-pile". Its actual value in a given case depends on the particular soil conditions and the interaction between the pile toe and the soil. A well-performed properly instrumented static loading test will establish the t-z and q-z relations for the pile. N.B., the pile toe movement cannot be taken directly from the results of the static test.

The load-transfer movement of interior piles usually develops as the structure is built and it occurs together with the 'elastic' shortening of the group, i.e., the "equivalent pier" effect.

The perimeter piles will have more shaft resistance, i.e., their response will be stiffer than that of the interior piles. Therefore, they will take on larger load than the interior piles. However, eventually, and probably rather soon, their shaft resistance will reduce, they will settle, and any differential movement ("settlement") will be reduced. For some geologies, the perimeter piles will be affected by downdrag and their load will be redistributed to the interior piles via the pile cap (which can be avoided by making the perimeter piles longer and, thus, providing a stiffer response.

For an actual design case, the most important settlement is that occurring in the soil underneath the pile toe level. This can be estimated by means of the "equivalent raft" approach.

The mentioned response of a piled foundation to load can be analyzed by a numerical approach. Dr. Hartono Wu and Dr. Harry Tan in Singapore (personal communication) have performed preliminary Plaxis analysis of a 6 x 6 pile group with 300-mm diameter concrete piles constructed to support a 7 m square, rigid pile cap in sand. The average c/c distance is 4.7 pile diameters. Some results for the center piles and the mid-side piles of the rigid pile raft (cap) are plotted in Figure 14. The graph appears to suggest that the pile toe resistances are about the same, but they are 42 kN for the center piles and 68 kN for the perimeter piles (the scale of the plot is deceiving). The toe penetrations are 4 mm and 13 mm, respectively. Because the cap is rigid, the movement differences are due to different axial shortening of the piles, in turn caused by the fact that the piles load response will be different. The rigid pile cap distributed the load over the cap, with more load being carried by the outer piles and less by the inner piles.

The perimeter piles show shaft resistance, while the interior piles show no shaft resistance but for a distance close to the pile toe level. The two piles at the mid-point of each pile cap side ("perimeter piles") have developed shaft resistance along most of the length. The four interior piles, although shielded by only two rows of piles, have practically no shaft resistance, except for along the lowest portion of the pile (starting about 2 m up from the pile toe). The results agree with foregoing qualitative theoretical discussion and show that the interior piles experience shaft resistance only a short distance up from the pile toe and only enough to establish an equilibrium of forces and movements at the pile toe. Moreover, even for this rigid pile cap, there is little contact stress developing even underneath the pile cap close to the perimeter.

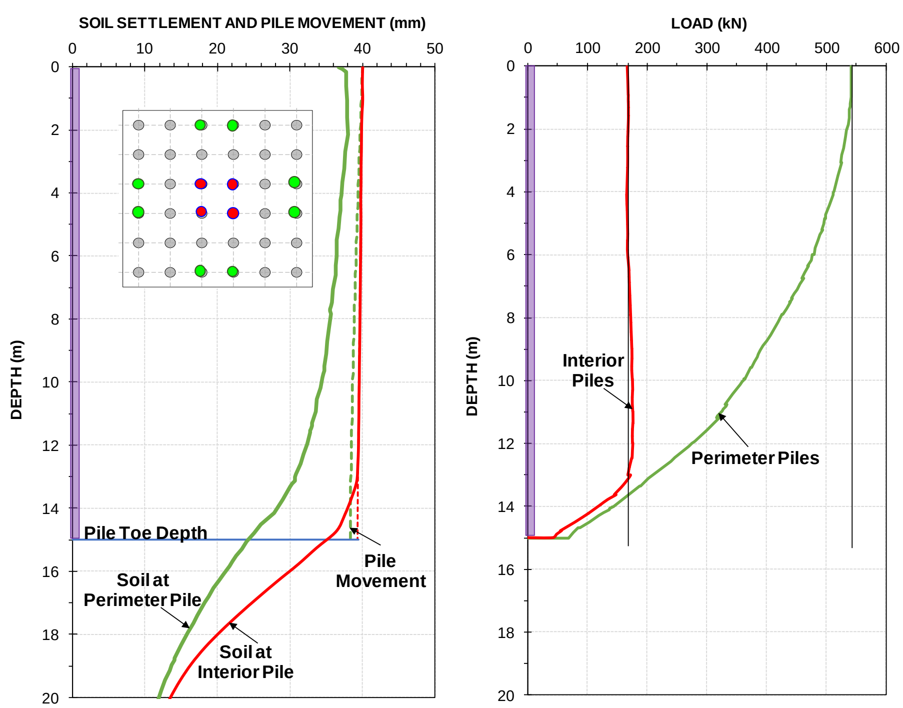

Figure 14. Distribution of soil settlement at perimeter and interior piles, pile and pile cap movements, and load for perimeter and interior piles.

Indeed, the Plaxis results support my assertion that assuming that contact stress would provide support to a piled raft is misleading. Moreover, the analysis shows that the response to load on the raft is deformation (shortening) of the piles, toe load-transfer movement, and settlement of the soil layers below the pile toe level.

CONCLUSIONS

In designing a piled foundations involving single piles or small groups of piles, the common approach is to assume that the pile or piles have definite ultimate toe and shaft resistances, i.e., a "capacity". However, ultimate pile toe resistance does not exist and a well-developed ultimate shaft resistance is a rare occurrence. Moreover, even when a static loading test shows a definite ultimate value, the individual pile elements making up the pile will have a range of mobilization of the ultimate resistance, and the sum of "ultimate" resistance of the various elements will not be equal to the ultimate value inferred from the test.

Moreover, the fact that the approach to defining the ultimate resistance, i.e., capacity, of a pile differs so widely in the profession adds considerable uncertainty to the capacity approach in conventional design.

The uncertainty of an interpretation of the results of a static loading test on a single pile is frequently further affected by omission of residual force in the test pile.

Transferring the results of a test on a single pile to the response of a pile group cannot be based on proportionality employing an efficiency factor, but must be made by applying t-z and q-z relations from the singe-pile test to the analysis of the pile group.

The notion that contact stress contributes to the capacity of a piled raft is highly questionable. The response of a wide piled foundations is governed by the deformation conditions below the pile toe level and by the how the perimeter piles are responding to the long-term development of the soil within the pile depth.

It follows, that the wisdom of basing foundation design on factors of safety or resistance factors is rather dubious. A foundation design commensurable with good engineering principles must primarily be based on deformation and settlement analysis. Such design is no more complex than a design based on a conventional capacity approach.

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