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Fallacies in Piled Foundation Design

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ABSTRACT. Full-scale tests on footings are not common, but none of the few reported—a couple of case history examples are quoted in the paper—show that a bearing capacity mode does not develop for loads acting at the center of the footing. For piled foundation design, it is recognized that the pile toe is a footing with a long stem. However, 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. 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. Unless the pile is very short or next to infinitely stiff, the accumulated value of the element ultimate resistances is not equal to the assumed shaft capacity. It follows that a foundation design, be it for a footing, a pile, or a pile group, must not be based on capacity assessment, whether it is a working stress or a load-and-resistance-factor design, but, instead, be based on deformation and settlement analysis.

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) named "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.

All building codes and standards addressing foundations indicate factors of safety to apply to the capacity of footings or piles, or, these days, indicates resistance factors or partial factors of safety to apply to establish factored resistances. The particular capacity is mostly determined based on simple soil mechanics principles or routine type of full-scale tests (mostly in the context of piles). Little space, if any, is given to the response to load in terms of movement and settlement. Yet, an acceptably "safe" foundation design means that the soil forces calculated using the "capacity-approach" will never occur. Instead, the first issue of a design should be "will the deformations and settlements be acceptable to the supported structure?". Although, the practice can amend and develop an existing approach—per a gradual evolution—but shifting to consider deformation first does not seem to be in the current collective geotechnical mind. In the following, I will indicate some of the foibles in the current "memes".

PRINCIPLES

"Capacity" is considered to be the response to a movement along a shear plane culminating in the ultimate friction, tan ϕ', or ultimate shear strength being mobilized and the soil entering a plastic state. That is, "capacity" is defined as an ultimate state, where, once developed, adding load does not increase the resistance but simply results in additional foundation movement. This state is expressed in Equation 1, the Terzaghi triple-N formula and Figure 1.

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

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

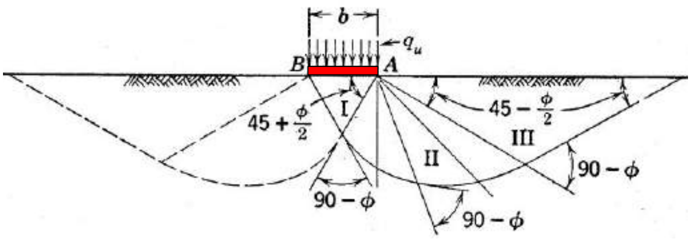

Fig. 1 Background to the triple-N formula

One can here go into a critique of the various N-factors and bring in an array of modifications aiming to achieve a higher level of sophistication—for example, adjustments according to the many approaches toward friction angle modifycations with reference to soil mineralogy, gradation, roundness, not to forget preconsolidation and cohesion relations—but I will not. The proof lies in eating the pudding, which means let's look at some full-scale test records. Figure 2a presents measured stress-movements from loading tests performed 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, where the soils consisted of fine sand 2.8 m above the groundwater table. The sand was compact, as indicated by an N-index equal to about 20 blows/0.3 m. The footings were placed at a depth of 1.0 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 despite relative movements as large as 15 % of the footing side.

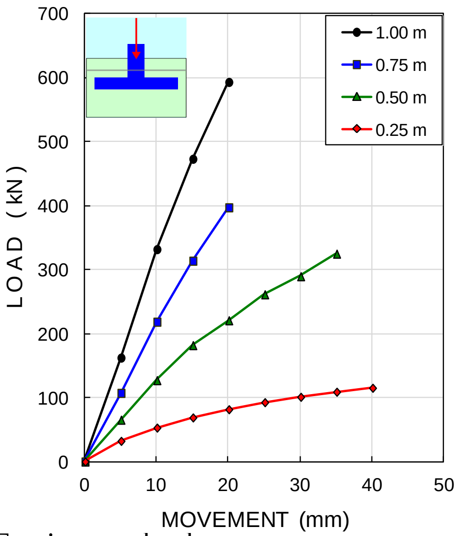

Fig. 2a. Footing-test load-movement curves (Ismael 1985).

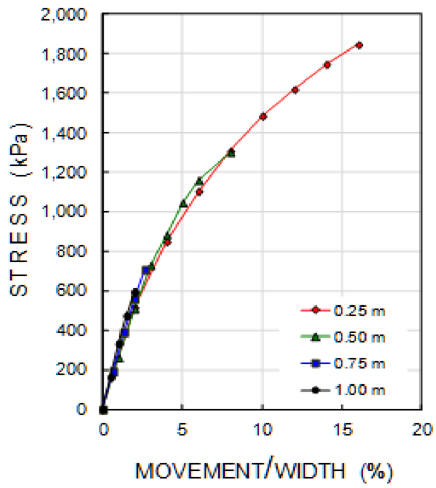

Fig. 2 b. Normalized curves (Fellenius 2016)

Briaud and Gibbens (1999) presented load-movement records from tests on five square footings in sand. Figure 3 shows the measured load-movement curves and, again, no change in response is noticeable that could be used to define a capacity. Results from many other similar footing tests have been published. None has shown test curves that could a reasonably and rationally be used to define a footing "capacity".

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 have established a toe-response load-movement curve and none has demonstrated a definite toe capacity. The fact is that there is no such thing as a footing capacity, nor is there any pile toe capacity. In contrast to a pile toe, a footing can fail. If so, it is by rotation ("overturning") due to eccentric and inclined loading, as shown in Figure 4. 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 a fallacy shared by many textbooks, codes, and standards.

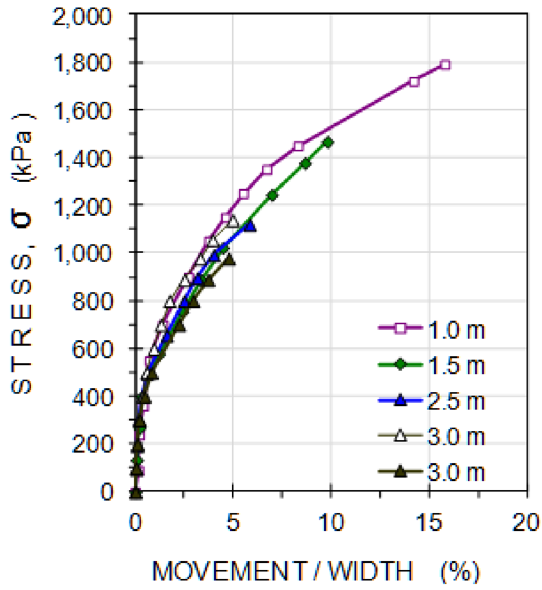

Fig. 3. Normalized load-movement curves (Briaud and Gibbens 1994)

In contrast to pile toe resistance, the shaft resistance along a pile element can actually develop a plastic response, i.e., true ultimate resistance. However, most of the time, the resistance response is by strain-hardening or strain softening. Fellenius (2016) has indicated several relations of unit shear/stress-movement response of shear or stress to load. Figure 5 shows four different curves, so-called t-z/q-z functions, going through a common 4-mm point, indicating a point common for the curves: a common target at 100-% stress, or load, and a specific movement, here chosen to 4 mm. The t-z and q-z terms in the ordinate title refer to shaft resistance and toe resistance, respectively. The t-z/q-z curves are based on very simple mathematical relations. One shows an almost straight line shifting to a horizontal line, i.e., a plastic response, at the 4 mm target. The usually assumed ideally elastic-plastic response implies a sudden kink, which is an unnecessarily primitive approach.

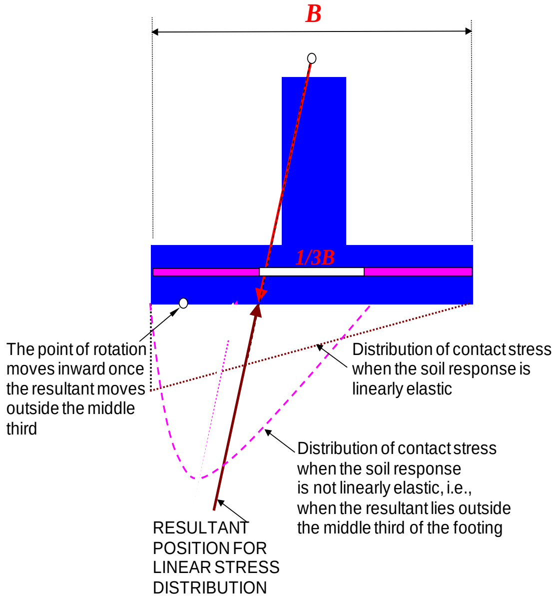

Fig. 4. Forces against a footing with the resultant located inside and outside the middle-third.

The lower of the two "strain-hardening" curves has a hyperbolic shape, which shape is often thought to be suitable for simulating plastic shaft resistance response. However, it only becomes approximately plastic after a long relative movement. The toe resistance is rarely other than the upper "strain-hardening" curve. Of course, the "target" movement can also be other than 4 mm. In fact, were all actually possible curves plotted in the figure, there would be no color white between the curves.

The 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. 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 value of the test.

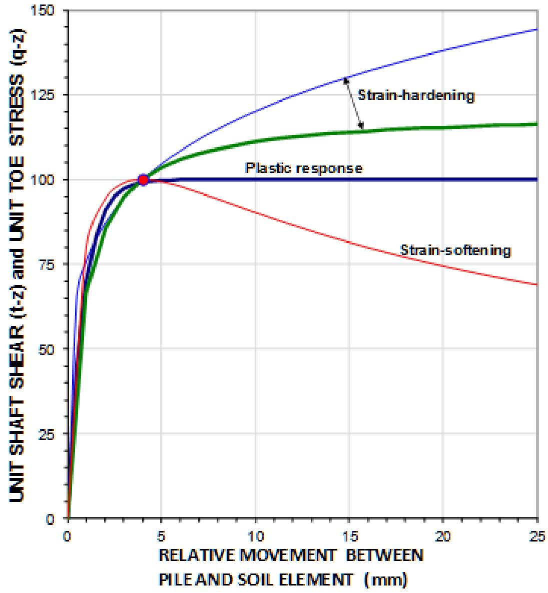

Fig. 5. Typical t-z and q-z curves (Fellenius 2016).

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. The figure shows that when the maximum load was reached in the test. However, only one or a few of the pile elements were at a stage representing their peak resistance. 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 has hardly begun to move and the mobilized toe resistance is small. The figure shows that whatever the definition of pile capacity applied to the pile-head curve, it will not harmonize with or correlate to 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 analysis of smaller or larger, shorter of longer piles at the site is then not likely to be correct.

If the shaft resistance had been a mix of strain-hardening and strain-softening 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

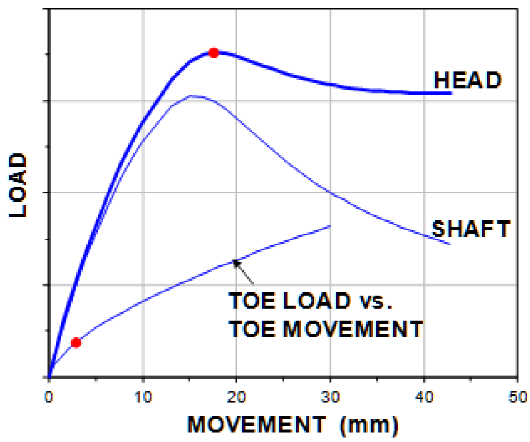

Fig. 6a. Load-movement records from a static loading test.

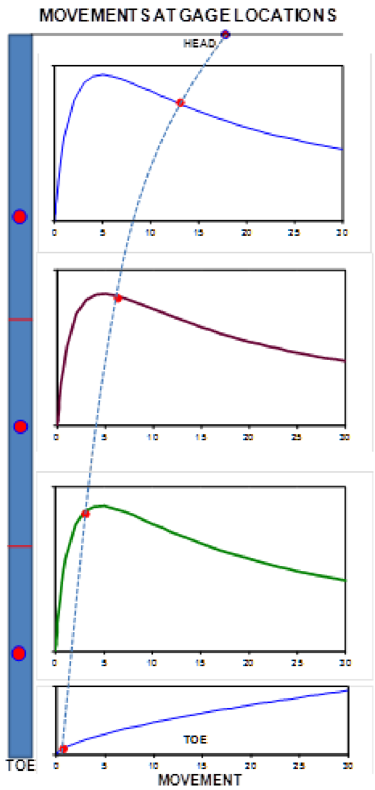

Fig. 6b. Development of resistance at gage locations.

(originating in a misconception of Terzaghi's 1943 recommendation), and the Chin-Kondner extra-polation is in widespread use in South-East Asia. For details of these and other definitions, see Fellenius (2016).In seeing how approximate a value of capacity one may obtain from a static loading test 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 a safety factor on tested pile "capacity" should be 2.0 or 2.2, or a resistance factor be 0.65 or 0.70.

CAPACITY AS EMPLOYED 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 other 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 other than by chance your prediction will not be close to the actual results unless you have access to prior results from previous piling work in the area or, at least, from experience in the particular geology. You would not commit yourself to a design without that more intimate or "insider" information, would you?

Participating in a prediction carries no risk other than to one's pride, however. 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 on for a 400-mm diameter, 17.5 m long bored pile in silty sand tested at the 1st 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 are compiled in Figures 7a and 7b.

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, therefore, 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 approach to determine the capacities. Note, in contrast to the curves, the capacities were not predictions, but assessments based on methods of determining a capacity from a pile-head load-movement curve. The surprise lies in the enormous range of pile-head movement that categorized the capacities.

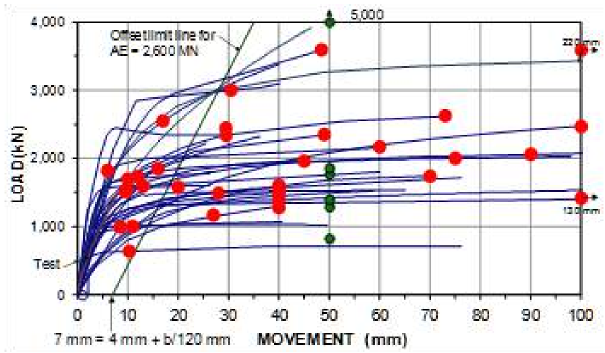

Fig. 7a. Predicted load-movements and assessed capacities for the pile in clay (Fellenius 2011).

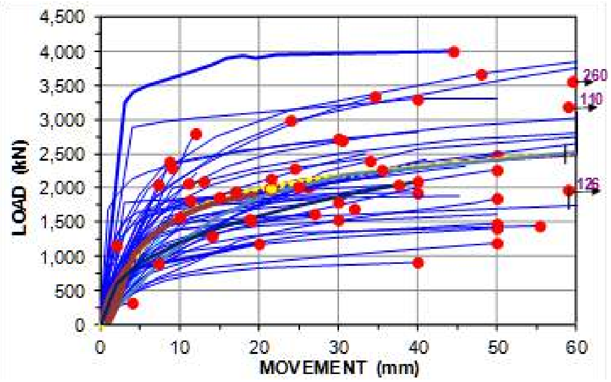

Fig. 7b. Predicted load-movements and assessed capacities for the pile in silty sand (Fellenius and Terceros 2013).

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 mm long bored pile in sand. The test included an unintentional unloading and reloading step. The premise of the prediction was that the test be carried to a final movement of 100 mm, 10% of the pile diameter. The task was to predict the pile-head load-movement curve for the test pile. After the prediction results had been published, I contacted all predictors and asked them to tell me, using their own definition, what capacity the actual test curve demonstrated. Twenty-nine, about half of the total replied, and Figure 8 compiles the capacities received. In contrast to the two previous results, this time the assessment is 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.

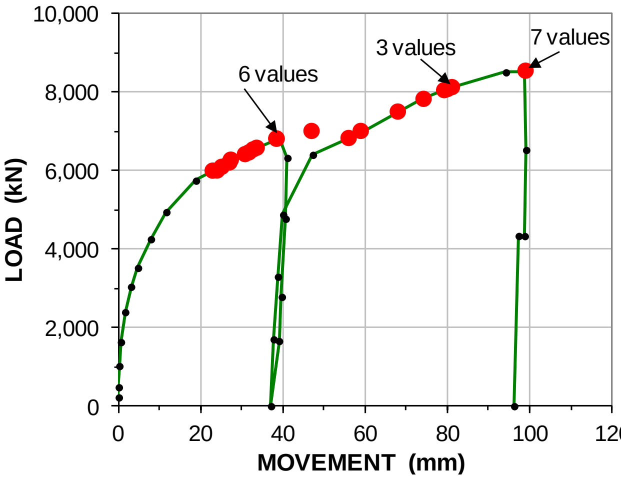

Figure 8. Test results and capacities assessed by 29 predictors for the Araquari prediction case.

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. However, ultimate pile toe resistance does not exist and ultimate shaft resistance is a rare occurrence. Moreover, even when a static loading test, shows a definite ultimate value—also a rare occurrence—the individual pile elements making up the pile will have an range of mobilization of the ultimate resistance and the sum of the various element resistances will not be equal to the ultimate value inferred from the test. Thus, theoretical calculations of shaft resistance is likely to overestimate the total resistance of the shaft. 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. Additional factors, not mentioned above, 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. 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 not any more complex than a capacity approach. However, discussing it is outside the scope of this paper.

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