Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 42 No.2 June 2011

GEOTECHNICAL

ENGINEERING

Journal of the

SOUTHEAST ASIAN GEOTECHNICAL SOCIETY

&

ASSOCIATION OF GEOTECHNICAL SOCIETIES IN SOUTHEAST ASIA

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Guest Editor

Prof. Tatsunori Matsumoto

Fellenius, B.H., 2011. Capacity versus deformation analysis for design of footings and piled foundations. Geotechnical Engineering. Journal of the South-East Asian Geotehcnical Society, 42(2) 70-77.

CAPACITY VERSUS DEFORMATION ANALYSIS FOR DESIGN OF FOOTINGS AND PILED FOUNDATIONS

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ABSTRACT Measurements of settlement of a piled raft supporting forty grain elevators and of settlement of piled foundations supporting five furnaces showed that the settlements were unrelated to deformation due to load transfer from individual piles to the soils and were instead governed by compression of the soils lying below the pile toe level. Full-scale static loading tests on footings and pile toes in sand showed that the soil response is not that of building up to an ultimate value — capacity —, but is rather due to gradually increasing compression of the soil below the footing or the pile toes. Pile toe movements from 26 piles founded in sand, although showing a scatter of soil deformation response, confirm the results of the tests. The findings are used to reaffirm that basing design on a capacity approach is flawed and that, instead, design of a piled foundation requires analysis of interaction between load transfer, soil settlement, and pile toe load-movement response.

INTRODUCTION

Footing and piled foundations are conventionally designed on capacity with a factor of safety or resistance factor applied to an ultimate resistance value — a capacity — defined by or perceived in a multitude of ways. The conventional approach is both costly and, at times, apt to provide unsafe foundations. The reference and knowledge on how to do better have been around a long time, but are frequently overlooked. It is necessary to turn the attention of foundation design toward the settlement aspect, being the more important component of the design. To illustrate what is lacking in common designs, the following presents results of full-scale observations and indicates what aspects that the designs could need to emphasize.

TWO CASE HISTORIES ON PILED FOUNDATION SETTLEMENT

Ghent Silo, Belgium

Goossens and VanImpe (1991) presented results of ten years of monitoring settlement along the side of a tightly-spaced group of 40 grain elevators, 52 m tall, founded on a 1.2 m thick concrete raft over a 84 m by 34 m footprint, and supported on 697 piles. The soil profile consisted of sand alternating with clay, as indicated in the cone stress diagram shown in Figure 1. The groundwater table lies at 3.0 m depth. For fully loaded silos, the total load distributed evenly across the footprint corresponded to a stress of 300 kPa.

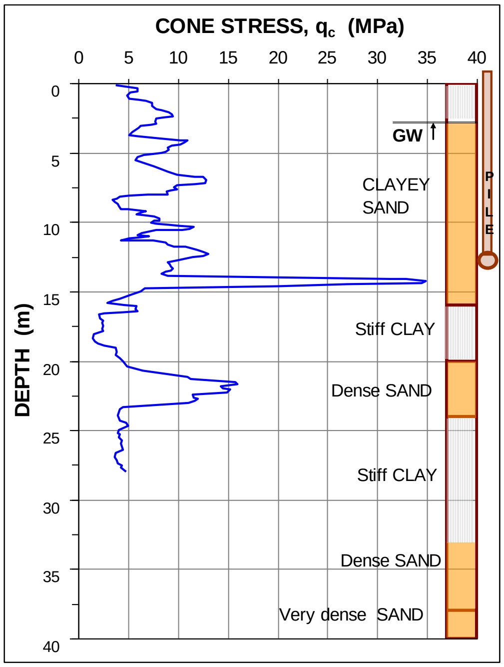

Fig. 1 Cone stress diagram and soil profile

The piles consisted of 520 mm diameter, 13.4 m long, driven, cast-in-place concrete piles with expanded base (Franki piles). As part of the construction, static loading tests were performed on two construction piles, Piles #085 and #585, to twice the assigned working load of 1,250 kN. The results of the tests are shown in Figure 2. Obviously, the pile capacity is more than adequate. At a load equal to the working load, the pile head movements were a mere 3 mm. Based on this, the settlement of the piled foundation was expected to be small. To investigate the long-term development, a programme of settlement monitoring at five bench marks affixed to the raft along one side was implemented. Figure 3 shows the results of the monitoring. Fitting by trial-and-error settlement calculated at the mid-point benchmark at the side of the raft to measured values calibrated a settlement analysis (the soil parameter input to use). The calibrated values were then used to calculate the settlement at the center of the raft, which indicated that it would have settled about 300 mm. Thus, the differential settlement to the corner would have been about 200 mm over 40 m, or about 1:200. However, Goossens and VanImpe (1991) report no sign of distress for the silo.

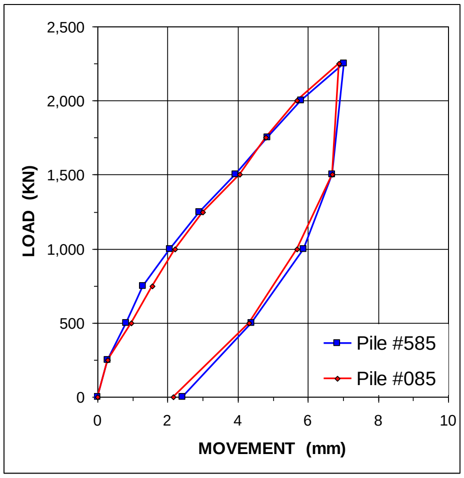

Fig. 2 Load-movement curve of the two static loading tests

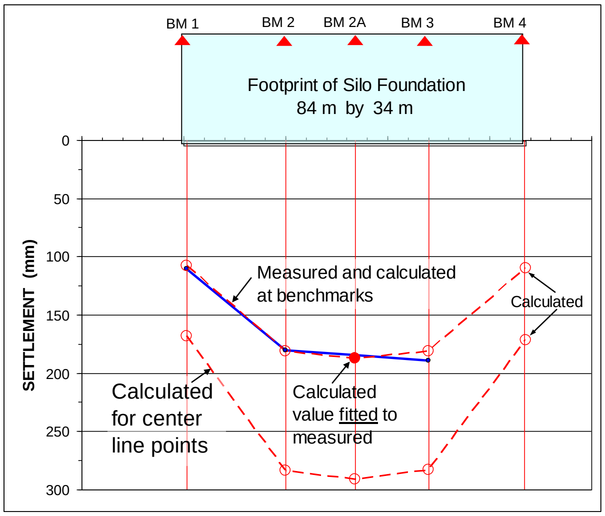

Fig. 3 Settlement measured along the silo side after ten years

In the author's hindsight opinion, the foundation would have responded just as well had about half as many piles been used.

QIT Plant, Quebec

Golder and Osler (1968) presented a case history of twelve years of settlement measurements of a bank of five furnaces. The furnaces were placed with long sides in parallel next to each other at a depth of 1.5 m about 6 m apart over a total footprint of about 16 m by 54 m (Figure 4). Each furnace has an 16 m by 10 m footprint and is supported on a group of thirty-two, about 6.0 m long, 600 mm diameter expanded-base piles (Franki piles) installed to a depth of 8.5 m and at c/c spacings ranging from 2.1 m through 3.2 m. The total furnace load is 21 MN, which corresponds to a load of 670 kN/pile and an average stress of 130 kPa over each furnace footprint.

As indicated in Figure 5, the soil profile consists of an upper 24 m thick sand deposit on a more than 50 m thick layer of Champlain Sea clay (formerly called Leda clay). The sand is composed of compact alluvial brown sand to a depth of 10.5 m, a 1.5 m thick interbedded layer of fine sand and soft clay, compact to dense grey sand to 19 m depth, and 5 m of sandy clay. The groundwater table lies at 4 m depth and the pore pressures are assumed to be hydrostatically distributed. Golder and Osler (1968) report results on laboratory tests available from four Champlain Sea clay samples obtained in a borehole located approximately 2,000 m away from the QIT site at depths ranging from 14 m through 38 m. Figure 6 shows the results of one of these tests. The four tests indicate virgin Janbu modulus numbers ranging from about 5 through 9, a re-loading modulus number of about 90, and a preconsolidation stress margin of 30 kPa through 80 kPa. The values are in reasonable agreement with typical values for Champlain Sea clay, usually exhibiting Janbu virgin and re-loading modulus numbers of 7 and 60, respectively, and a preconsolidation margin of at least 30 kPa.

Figure 7 shows the load-movement curve from a static loading test performed on one of the piles to a maximum load of 1,800 kN, twice the working load. The load-movement curve of the test was essentially a straight line indicating that the pile capacity is much larger than the maximum load applied in the test. The measured movement of the piles for a load equal to the working load was about 1 mm. The load-movement of the pile toe for the applied maximum test load is considered small, a few millimetre only. The test results were used to predict that the settlement of the furnaces under full load would amount to 10 mm.

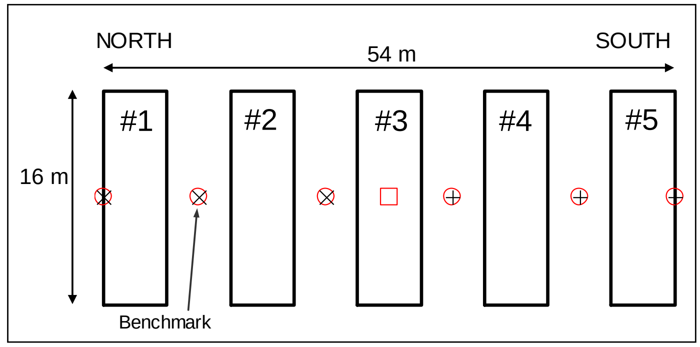

Fig. 4 Layout of furnaces

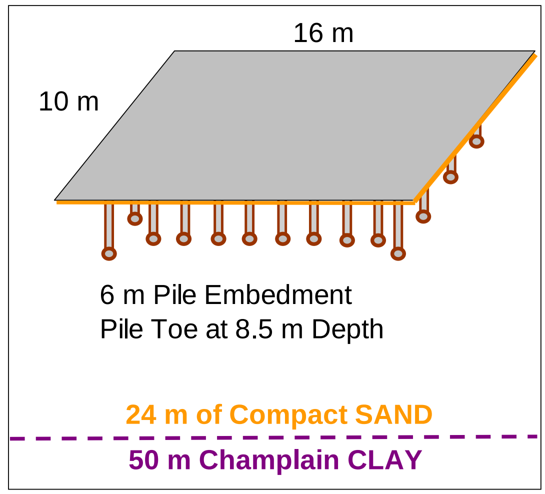

Fig. 5 Sketch of a furnace pile group and soil profile

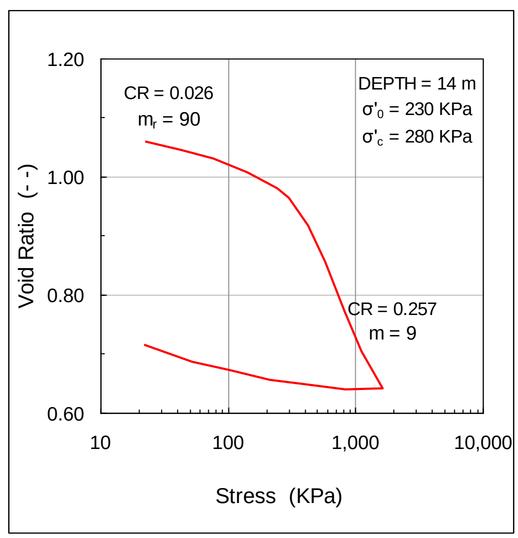

Fig. 6 Void ratio versus stress from consolidation test

The furnaces were built in early 1951. Settlement of the furnaces was monitored until November 1965 at six benchmarks placed between the furnaces. Figure 7 presents settlements measured across a section through the furnaces from April 1951 (when all five furnaces were completed) through January 1962. The figure also shows the settlement calculated, using conventional methods, along the sides of Furnace #3 as fitted to the January 1962 values. The parameters obtained by the fitting were then used to calculate the settlements along the other benchmarks.

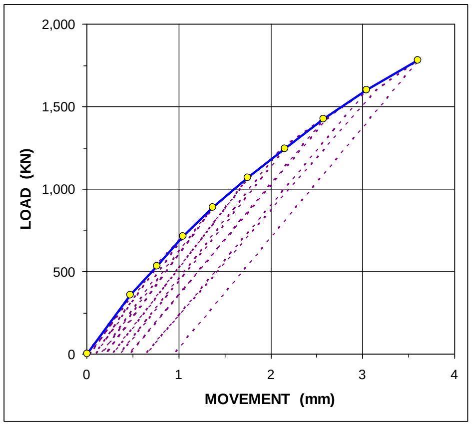

Fig. 7 Load-movement from the static loading test

Figure 8 shows the measured settlements versus time with eye-balled trend lines superimposed. The dashed trend lines represent measurements at the center of Furnaces #1 and #5 ("Side Furnaces") and measurements taken between Furnaces #2 and #3 and Furnaces #3 and #4, (the "Center Furnaces").

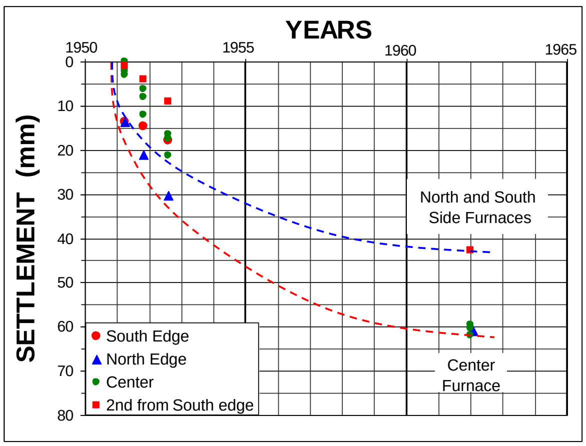

Fig. 8 Settlements from early 1951 through January 1962

It is obvious that the soil stresses due to the furnace loads will be the largest under the center furnace, Furnace #3, and be the smallest for the two end furnaces, Furnaces #1 and #5. The stress distributions can be determined numerically. Figure 9 shows the Boussinesq vertical stress distributions underneath each furnace as caused by the load on one furnace, Furnace #1, only, as calculated with the UniSettle program (Goudreault and Fellenius 1996). The stress distributions indicate that the load on one furnace has little effect beyond the depth of one furnace width, about 10 m at 20 m depth. However, the stresses from all five furnaces accumulate and overlap.

Figure 10 shows the settlement distribution calculated for benchmarks below the outside edge of Furnaces #1 and #5 and below the center of Furnace #3. The calculation has been fitted to the settlements observed for Furnace #3. The so-"calibrated" soil parameters were then used to calculate the settlements below centers of the other furnaces. The plotted values show good agreement between calculated and measured values. Note that for a preconsolidation margin of, say, 30 kPa, at a depth interval of 30 m through 40 m, the settlement caused by the stress below the center of Furnace #3 will result in significantly larger settlement as opposed to those from the stress below the outside edge of Furnaces #1 and #5, where the imposed stress is smaller than the value of the preconsolidation margin.

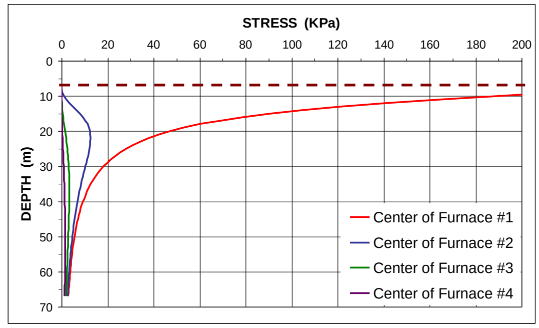

Fig. 9 Boussinesq stress distribution below furnace centers

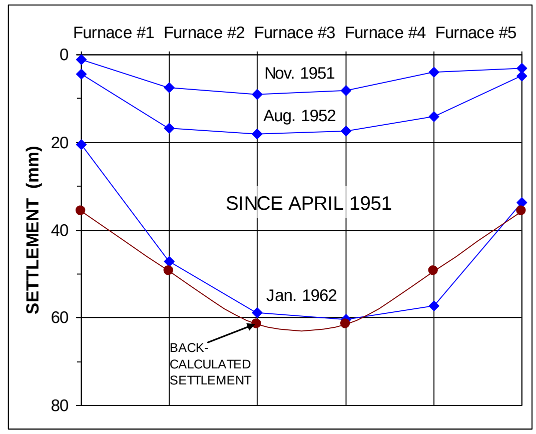

Fig. 10 Settlements measured below furnace centers

Figure 11 shows the settlement distribution versus depth at the edge and center of the furnace bank, calculated using the parameters fitted to the observed settlements at the Furnace #3 benchmark.

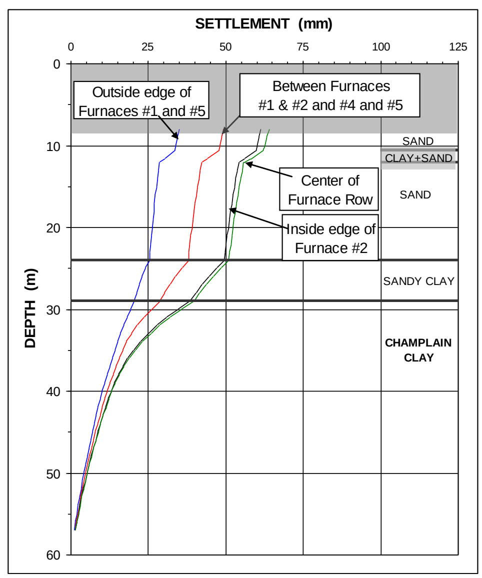

Fig. 11 Vertical distribution of settlement calculated at the edge and center of the furnace bank

The settlement observations indicate moderate settlements of about 60 mm. However, this value is much larger than what originally expected for the piled foundations. Yet, as shown, the actual settlement could have been predicted using methods generally available already in the 1950s.

THE BEARING CAPACITY OF FOOTINGS

Footings in Kuwait

Ismael (1985) presented results of loading tests on square footings buried at a depth of 1.0 m in compact sand. Footing diameters were 0.25 m, 0.50 m, 0.75 m, and 1.00 m The groundwater table was located at 2.8 m below grade. The data are replotted and normalized in Figure 12. After normalization, the results of all footings show the same tendency and no specific or characteristic value is indicated that could by any flight of imagination be claimed to be a "capacity", despite the relative movement being as large as 15 % of the footing diameter.

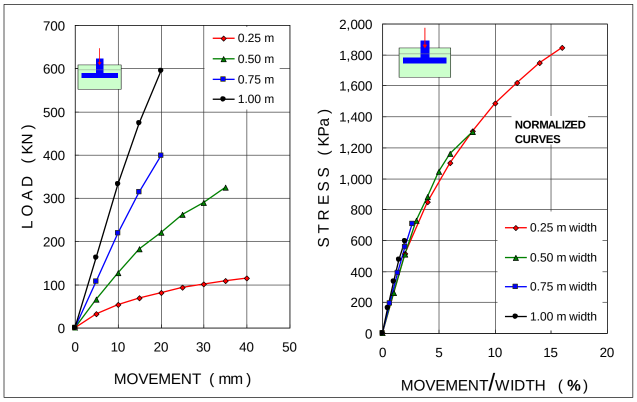

Fig. 12 Measured and normalized footing load-movements for footing tests in Kuwait (data from Ismael 1985)

Footings in Texas

Briaud and Gibbens (1994) presented results of five square footings placed on an excavated surface of compact silty fine sand. Figure 13 shows a replot and normalization of the data. Again, no characteristic values is shown that can be defined as "capacity". A "q-z"-relation of the form shown in Eq. 1 has been fitted to the normalized curves.

$$\frac{R_1}{R_2} = \left(\frac{\delta_1}{\delta_2}\right)^e \qquad (1)$$

where R_1 = Load 1
R_2 = Load 2
δ_1 and δ_2 = movement mobilized at R_1 and R_2, respectively
e = an exponent usually ranging from a small value through unity

The exponent "e" fitted to the data is equal to 0.4. However, the measured values can equally well be fitted to a conventional settlement calculation, employing values of preconsolidation stress and compressibility for reloading and virgin loading. On manipulating a set of three parameters, almost every load-movement response can be made to fit data and "prove" a theory.

Footings in Japan

Kusakabe et al. (1992) carried out plate-loading tests in a 16 m wide and 14 m deep excavation to a depth about 12 m below the groundwater table. The tests were made in a caisson using compressed air to keep the water at bay. The soils consisted of a volcanic, highly overconsolidated, sandy gravel. The normalized load-movement results are shown in Figure 14. The initial steeper portion of the curves and maximum curvature at about 5 % movement show the effect of the preconsolidation conditions, which to the uninformed might be misinterpreted as a "capacity" value.

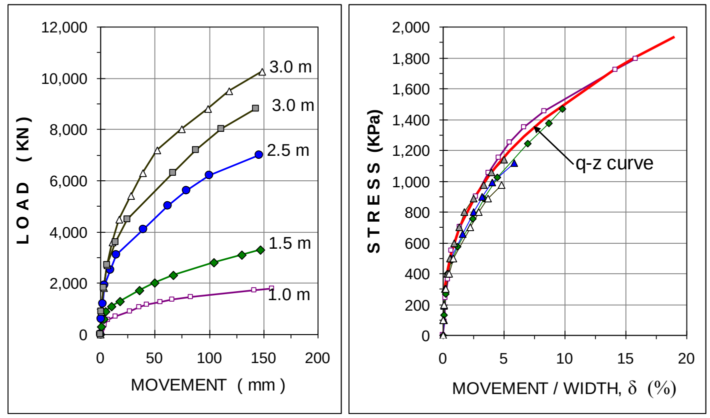

Fig. 13 Measured and normalized footing load-movements for footings in Texas (data from Briaud and Gibbens 1999)

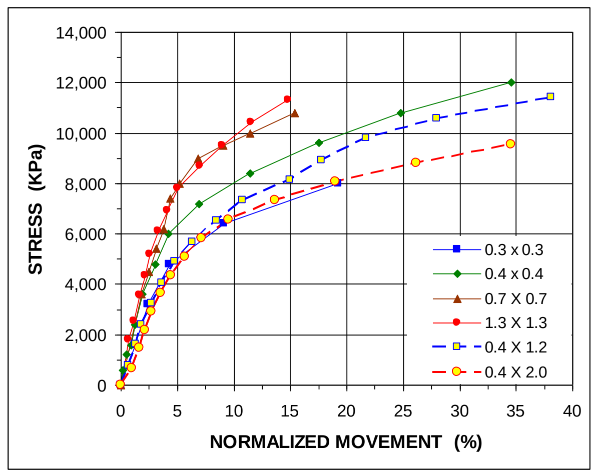

Fig. 14 Normalized footing load-movements for footings in Japan (data from Kusakabe et al. 1992)

Footings in Australia

Lehane (2008) organized a prediction event in Perth, Australia, on the response to load applied to three square footings, two of 1.0 m and one of 1.5 m diameter placed in sand 1.0 m below ground. Figure 15 shows the measured and normalized load-movement curves.

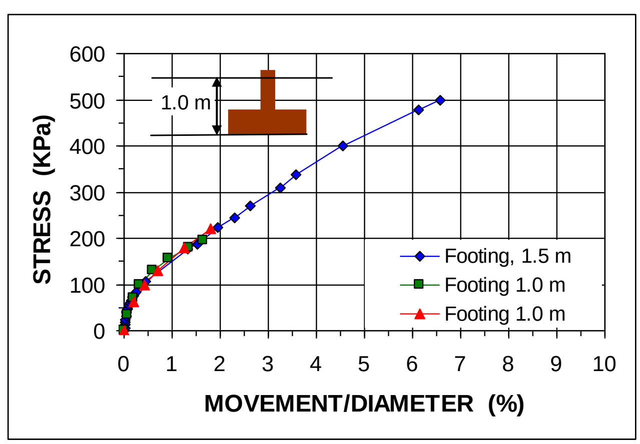

Fig. 15 Normalized footing load-movements for footings in Australia (data from Lehane 2008)

The results shown in Figure 15 indicate a presence of preconsolidation or cementation, but, similarly to all the previously mentioned tests, no indication is evident of a "capacity" value.

Footings in Sweden

Bergdahl et al. (1985) reported tests in compact silty sand on footings of size 0.55 m x 0.65 m and 1.1 m x 1.3 m, as presented in Figure 16 (normalization is made for average side length). Again, the eye-balled trend of the normalized curves is a gently rising curve with no suggestion of any distinct "capacity" value.

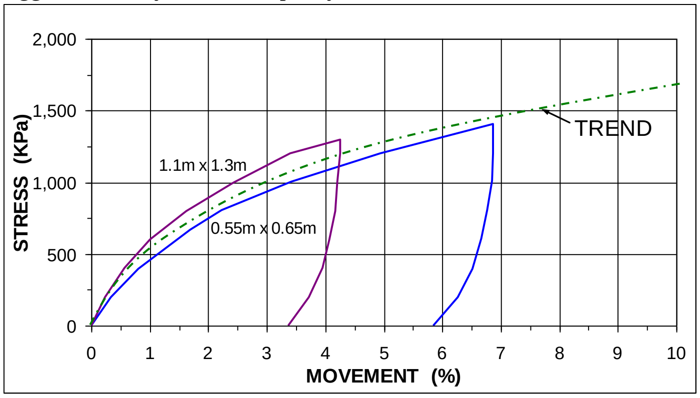

Fig. 16 Normalized footing load-movements for footings in Sweden (data from Bergdahl et al. 1985)

Many additional examples of footing response to load can be referenced, showing similar absence of a distinct "capacity" value. Of course, the literature does include tests that do show a plunging response. However, those cases are invariably from tests on small diameter model footings on the surface of relatively loose soil. Actual footings do not behave the way small model footings do.

Moreover, the examples shown are from tests in sand. For tests in clay, the load-movement curve often does show a failure value. However, this is not because clay is fundamentally different from sand. It is, due to the fact that in clay, unlike in sand, after a load is applied, the induced pore pressures will linger on. Then, unless the time before each next increment is long enough to allow for the induced pore pressure to dissipate, the gradual increase of pore pressure as the load is increased reduces the effective stress. Because effective stress governs strength, eventually, the applied load cannot be sustained, and a failure condition develops. A test with increments applied with adequate time in-between each increment would show a response very similar, qualitatively, to that of a test in sand.

THE RESPONSE OF A PILE TOE TO LOAD

A pile toe is in principle a footing with a long stem. This is recognized by the fact that since long, when determining pile toe bearing capacity, people usually model the pile toe as a footing. With the advent of Dr. Jorj Osterberg's invention in the late 1980s, the bi-directional cell test, and the thousands of O-cell tests made since then, it has become very clear that, like footings, a pile toe does not demonstrate a capacity condition even at large movement into the soil. Figure 17 shows results of an O-cell test in the form of stress applied to the pile toe versus the pile toe movement divided by the pile diameter in percent. The pile is from a test in Puerto Rico on a 914 mm diameter, 16 m long bored pile constructed with the pile toe in a fractured saprolite. For conformity with the previous load-movement graphs, the data are plotted in the first quadrant. The load-movement data have been fitted to a q-z function and extrapolated.

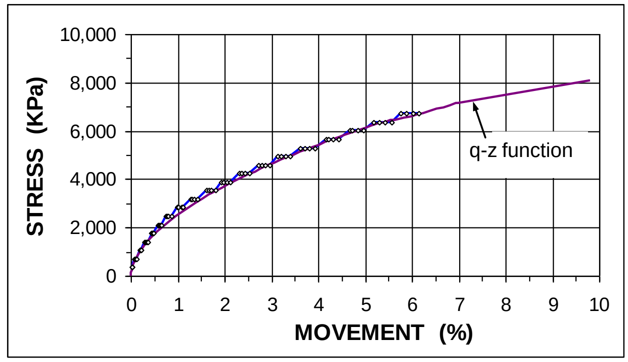

Fig. 17 Normalized load-movements measured for a pile toe in saprolite, Puerto Rico (data from Loadtest 2007)

The results of the O-cell test together with the results of the footings tests from different geologies and regions of the world show clearly that the response of actual footings and pile toes to load, even at large relative movements, is governed by the deformation characteristics of the soil and by the fact that the volume of soil involved or affected changes all through the loading. The conventional bearing capacity concept is simply deeply flawed and should be discarded, because when the basic concept for a response is wrong, plainly, all interpretations of the results based on that concept are also wrong!

It is not that the profession lacks an alternative approach. As the author aims to show, the alternative exists and is simple.

The stress-movement response shown in Figure 17 can easily be fitted to a theoretical calculation using conventional parameters of elastic modulus (or Janbu modulus number). To fit a calculation to the measured curve shown in Figure 17, which indicates presence of preconsolidation or cementation, only three parameters are needed**:** Elastic modulus in the preconsolidation range, E_r, virgin elastic modulus, E, and a preconsolidation margin, dσ' (the use of the term "consolidation" is not meant to suggest that the process necessarily is one of consolidation, i.e., dissipation of pore pressures). For the match shown in Figure 17, Boussinesq stress distribution and the following parameters were used: E_r = 200 MPa, E = 25 MPa, and dσ' =1,000 kPa. The E-moduli correspond the a Janbu stress exponent, j, of unity and modulus numbers of 2,000 and 250, respectively. The back-calculations are made by means of the UniSettle software, (Fellenius and Goudreault 1994). [The author intentionally imposed non-precise parameters and employed purely elastic analysis, which is the simplest method possible for fitting the calculations to the data. Indeed, for fitting theory to data, every method is as good as every other, and a good fit does not prove a method to be correct].

The foregoing is intended to make the following two matters clear.

1. Bearing capacity theory for the response of a footing or a pile toe to load is flawed and has no longer any place in serious geotechnical design.

2. Response to load applied to a footing or a pile toe can and should be pursued by deformation analysis.

Proper design of a piled foundation requires calculation of the pile toe response to the imposed loads. As shown above, the response can be back-calculated using conventional theory for deformation, simple or more sophisticated — an easy task, once the response is known. In contrast, design involves predicting the response, which requires that the input parameters are reliable and representative for the site and geology. If the parameters are uncertain, a design will either have to be overly conservative, and therefore costly, or the parameters and actual site conditions be determined by means of a well-planned test. The test must consider the particulars of the analysis so that the results can be properly analyzed to yield the pertinent parameters. If the analysis is limited to establishing a "capacity" value, much of the test effort is wasted and, at times, the design may fail to provide a serviceable foundation.

THE COLISEUM EXPERIENCE

The Northridge earthquake in Los Angeles, California, in January 1994 was a "strong" moment magnitude of 6.7 with one of the highest ground acceleration ever recorded in an urban area in North America. The earthquake caused an estimated $20 billion in property damage. Amongst the severely damaged buildings was the Los Angeles Memorial Coliseum, which repairs and reconstruction cost about $93 million. The remediation work included construction of twenty-eight, 1,300 mm (52 inch) diameter, about 30 m long, bored piles, each with a working load of almost 9,000 kN (2,000 kips), founded in a sand and gravel deposit.

The piles had been designed using the usual design for capacity approach with adequate factors of safety to guard against the unknowns, which considered that the acceptable maximum movement was more stringent than usual.

It was imperative that all construction work was finished in six months (September 1994, the start of the football season). However, after constructing the first two piles, which took six weeks, it became obvious that constructing the remaining twenty-six piles would take much longer than six months. Drilling deeper than 20 m was particularly time-consuming. The design was therefore changed to about 18 m pile length, combined with equipping every pile with an O-cell at the pile toe. — Note, the O-cell was now used as a construction tool, an innovative and very successful approach. On completion of each pile, the O-cell was activated twice. First to precompress the soil below the pile toe and, second to verify that desired precompression was obtained. The O-cell load-movement records from second loading (re-loading) of each pile toe were reviewed for verification that the pile toe response had been made adequate. For a couple of piles, a second reloading was needed to ensure a satisfactory result. A background to the project is provided by Meyer and Schade (1995). Schmertmann and Schmertmann (2009; 2012) describe the results, showing the benefits for the project in terms of both time (the construction was finished on time) and the considerable savings of costs achieved.

The results of the initial loading, which is the "virgin" test of the piles as constructed "conventionally", are of interest in the context of this paper. It is indeed unusual — an understatement — to have records of tests from all piles at a site constructed under the same conditions. Figure 18 shows load-movement O-cell results of one of the piles (Pile 10B; this pile that had a largest test-induced pile shaft upward movement). Figure 19 compiles the O-cell results of all twenty-six piles, demonstrating a variation of pile toe response — scatter of results. The virgin tests were either brought to a maximum O-cell load about equal to twice the working load, or the O-cell was unloaded when the cell opening approached 100 mm. For some of the tests, the virgin loading indicated a response that one might consider acceptable for the piles. However, for most, the virgin pile response was inadequate. That is, had a conventional design and construction resulted in the actual piles and, had that design used a single proof test for its verification, chances are that the proof test would not have revealed that some piles were not going to provide a satisfactory response. Or, worse, the proof test result might have been thought as proving adequacy of the design.

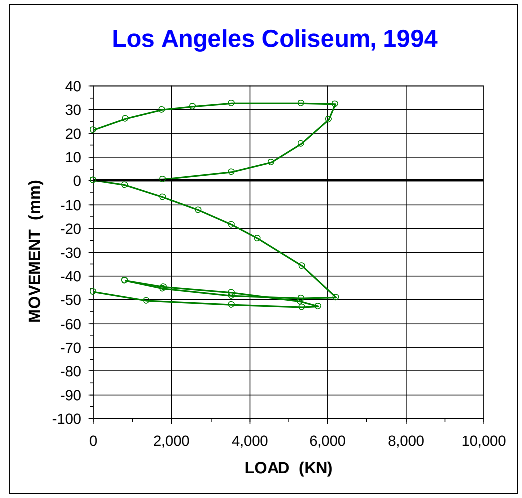

Fig. 18 O-cell results of the Coliseum tests for Pile 10B

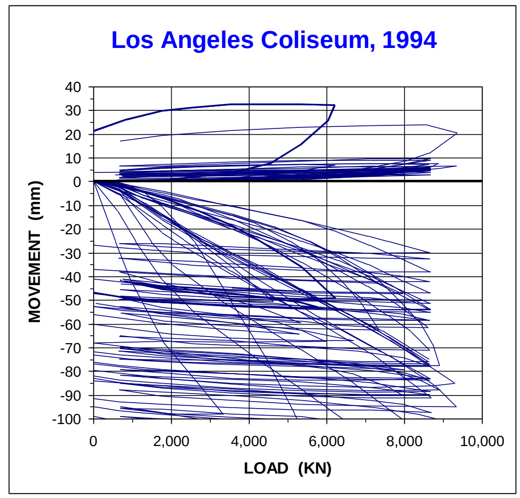

Fig. 19 Results of all twenty-six tests compiled

Figure 20 shows two graphs marked "A" and "B". Figure 20A shows the downward load-movement curves of the virgin tests and Figure 20B shows those of the first re-loading. (For consistency with the manner of showing the footing response, the data are plotted in the first quadrant).

For the virgin tests, the stiffness is quite variable. On average, the load that imposed a 25-mm movement was about 4,000 kN. Figure 21 shows relative frequency of the load for 25 mm movement. The "σ" stands for standard deviation value, and the 2σ and 4σ intervals encompass 67 % and 95 % of all results, respectively. The coefficient of variation, COV, is 0.45.

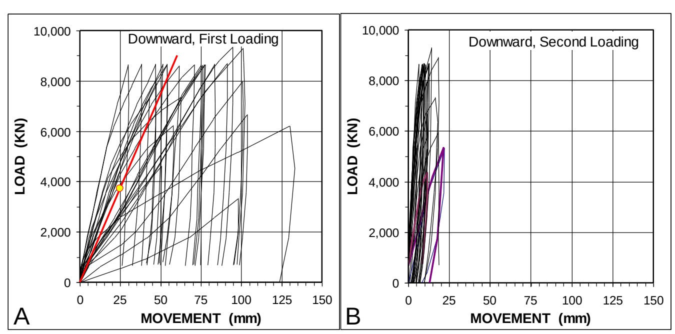

Fig. 20 Downward O-cell load-movements
A. Virgin tests. B. Reloading tests

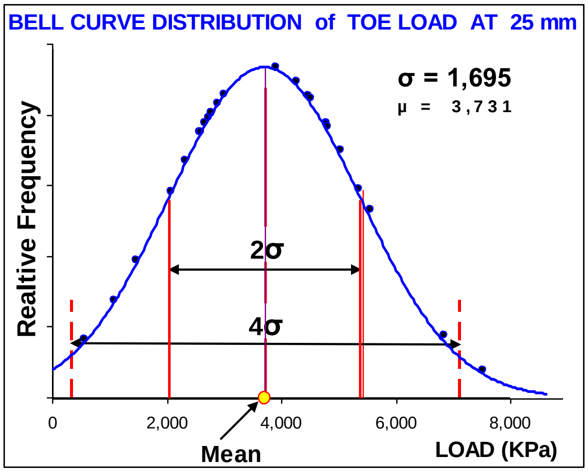

Fig. 21 Relative frequency of load having imposed a 25 mm movement

Figure 22 shows the load-movement curves between 20 and 60 mm imposed movements. The slope of the lines appear to be very similar for the tests. However, the slopes taken as load change between 25 mm and 50 mm movement divided by 25 mm (after adjustment to pile toe area) actually range from 20 MPa through 120 MPa. If this range is taken as a range of stiffness —elastic modulus — it corresponds to a Janbu modulus number, m, ranging from 200 through 1,200 (stress exponent is unity).

Figure 23 shows the slope values in a frequency chart indicating that the average is 71 MPa and two-thirds of the values lie between 45 MPa and 97 MPa. (Values from three tests are not included because the maximum O-cell load did not impose a 50 mm movement in those tests).

Even under the ideal conditions of the Coliseum case, i.e., piles of equal length and diameter, all pile toes in the same soil deposit, piles being tested at same time after construction, and absence of a cementation or preconsolidation effect, the variation of load-movement response in the virgin tests is substantial. However, a good deal less so than the numerous theoretical methods of determining the pile toe capacity would have shown, and, unlike such approaches, the pile toe load-movement response is accessible to analysis and modelling. Note, unlike the capacity approach, a load-movement approach is accessible to analysis and modelling. This can be by simple modelling, as employed in this paper, or by more sophisticated models using numerical methods based on theoretically acceptable soil stress-strain behavior.

Results of an O-cell test are usually thought to be carried out "just" for establishing pile capacity. However, its value goes far beyond that, because knowing the pile toe response to load is essential for a pile settlement analysis. As an example (Fellenius and Ochoa 2009) of the use of that knowledge, Figure 24 shows the interaction of soil settlement below the force and settlement equilibrium (neutral plane) and two pile toe movement curves from O-cell tests (marked G1 and B1). The pile toe load is a function of the enforced pile toe movement, which is a function of the soil settlement at the neutral plane, which location is a function of the pile toe load — a loop for which an iterative calculation establishes the equilibrium between forces and movements.

Piled foundation design requires input of expected load-movement behavior of the pile toe. Pile design is far more than cutting back some perceived capacity value by slipping in a more or less out-of-the-air-or-code-selected factor-of-safety or resistance factor and obliging the so-reduced value to be no smaller than the desired working load. The process is far more complex than this. Design of a piled foundation requires establishing the load and resistance distribution with depth and settlement of the soil profile below the depth of equilibrium of forces — the neutral plane — and, in particular, pile toe load-movement response. The design process is a simple iterative activity (Fellenius 1984; 1988; 2004, and Fellenius and Ochoa 2009).

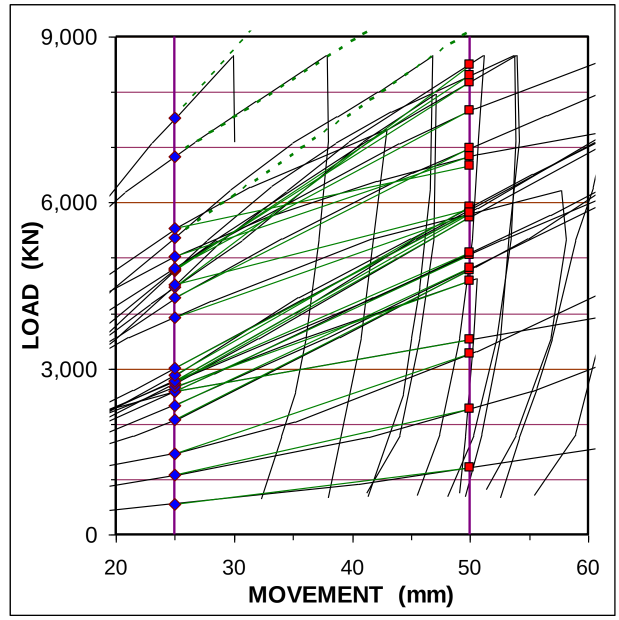

Fig. 22 Detail showing loads at 25 mm movement

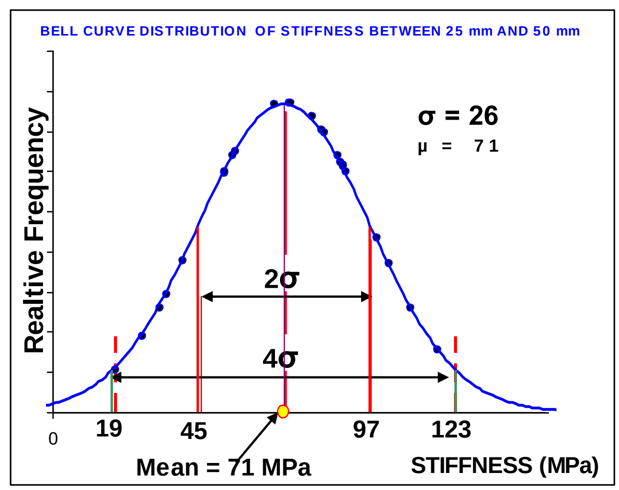

Fig. 23 Relative frequency of stiffness response for loads having caused movements of 25 mm and 50 mm

CONCLUSIONS

Where compressible soil layers exist below the pile toe level, considerable settlement of a piled foundation may occur in these layers.

The response of a footing or a pile toe to an applied load does not display a characteristic point, such as a "capacity" value, but is a function of the deformation characteristics of the soil and the stress conditions below the footing or the pile toe.

While the load-movement response can be analyzed, actual response can show considerable variation. When no reliable reference is available, tests are necessary or a design could be both costly and uncertain.

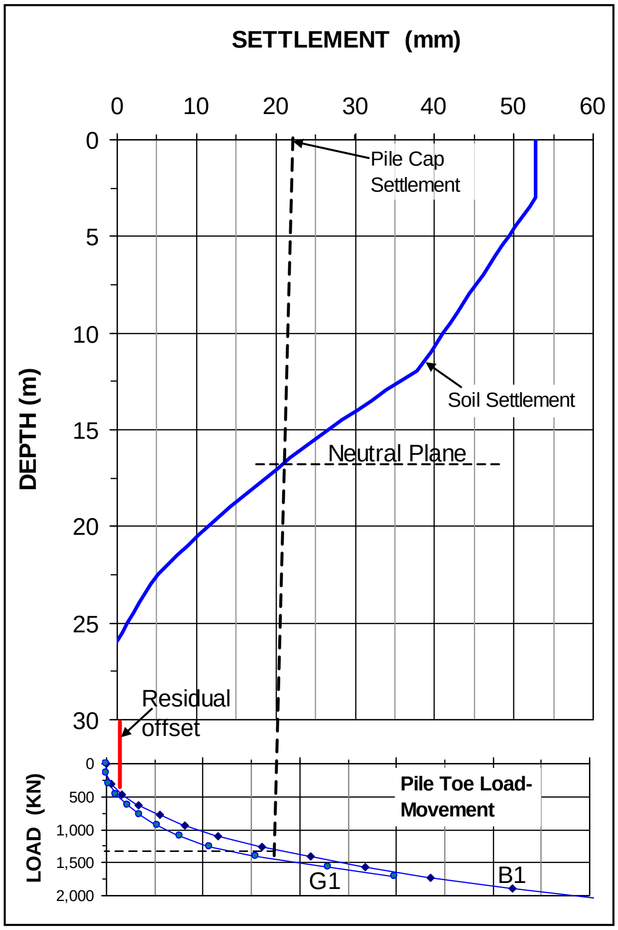

Fig. 24 Interaction between soil settlement, pile toe movement and pile toe load (Fellenius and Ochoa 2009)

The Coliseum study shows how settlement of piles can be controlled by means of imposing a prestress effect using the O-cell, i.e., employing the O-cell as a construction tool. The principle of prestressing the pile toe is not new. Pressure grouting in sand has been used for many years as a routine method for stiffening up a pile toe as deemed necessary in design or as a remediation measure. However, a novelty and advantage lies in that the O-cell method includes the means for checking that the results are satisfactory.

The design of a piled foundation requires analysis of load transfer, determining distribution of settlement of the soil around and below the piles over time, and establishing the load-movement response of the pile toe.

The settlement of a piled foundation due to load transfer from the pile to the soil is less critical in design as opposed to the settlement due to downdrag, i.e., the settlement of the pile as imposed by the soil settlement at the neutral plane.

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