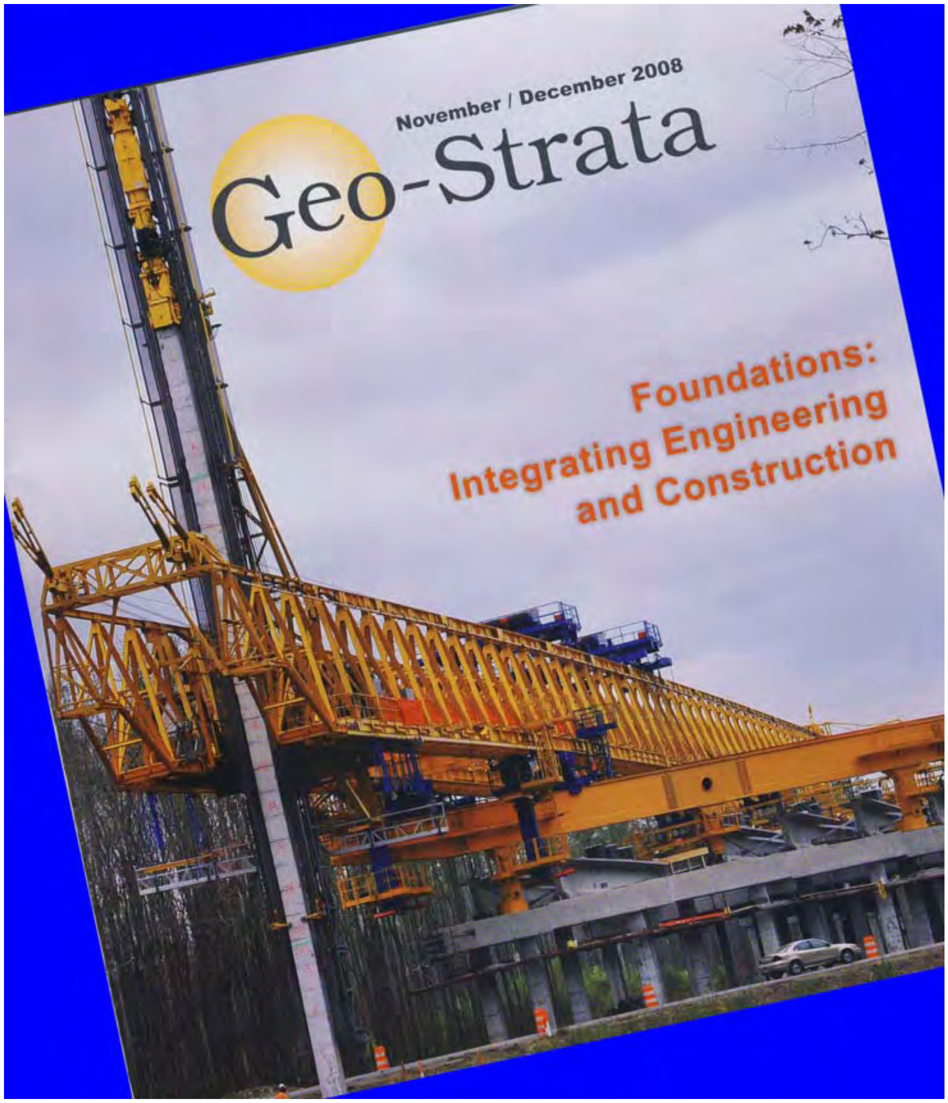

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Foundation Design Approach of Past, Present, and Future

By Bengt H. Fellenius, Dr. Tech., P.Eng., M.ASCE

The Past and Present

In the Past, practioners designed from observed settlement response to load. Then, a number of papers published from the 1920s through the 1940s established geotechnique as a field that used analysis and calculations to arrive at an economical and safe foundation design. The Factor of Safety on Capacity became the magic concept. During the almost century long time since the 20s, the profession refined the analysis methods and these days—the Present—computer programs make everyone a "wizard" in analysis of response of a foundation to applied load. Amazingly, there has been very little advancement in what goes into these programs. The Standard Penetration Test (SPT) is still the dominant field exploration tool. Total stress—undrained shear strength—is still the most common soil parameter used as input for calculating capacity and linear elastic modulus is assumed when calculating movement.

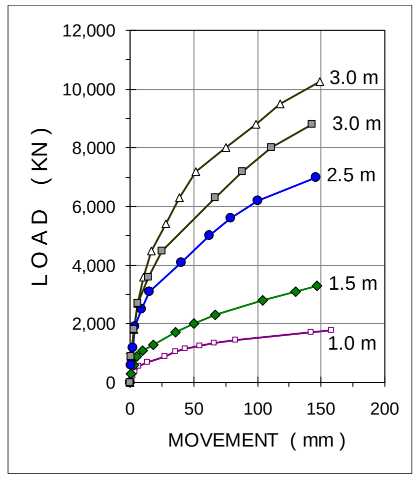

Figure 1
Load-movement of five 1.0 m to 3.0 m square footings on sand.

The Historical Perspective

What analysis case can be simpler than that of loading a footing with a diameter of a metre or two placed a short distance into a sand? Terzaghi presented in 1943 the "triple N formula" for calculating the capacity of such a footing. Many others refined the original N coefficients using ultimate resistance values from model footing tests. The range of published values for the Nq coefficient varies by more than an order of magnitude. This wide range of the key parameter should have alerted the profession to that perhaps the pertinence of the formula could be questionable. When critical state soil mechanics came about (advancing the concept proposed by Casagrande 1935), the reason for the model tests reaching an ultimate value became clear: model tests affect only the soil to shallow depth, where even the loosest soil behaves as an overconsolidated soil. That is, on loading the model, after some initial volume change, the soil dilates and finally contracts resulting in a stress-deformation curve that implies an ultimate resistance.

Actual footings do not behave the way model footings do. See, for example, the Texas A&M tests on square footings in sand presented in Figs. 1 and 2. Note that even at the extreme movement of 15 % of the footing width, no indication of failure is shown. Real footings do not reach an ultimate failure mode (unless the soil is clay and the loading is rapid causing pore pressures to increase). So for the future, that should be now, let's abandon the "triple N formula" and rely on design using deformation characteristics. But what deformation characteristics should we use?

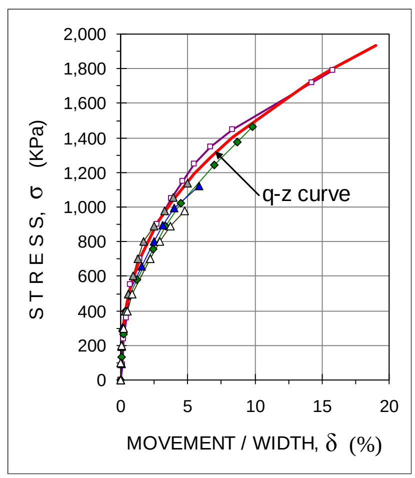

Figure 2
Stress versus relative movement and fitted q-z curve for footing in Figure 1.

Where to Go From Here; Footings

It is common to calculate the settlement of a footing by applying an elastic modulus. The modulus value is often taken from test results, choosing an average or a perceived representative value. However, the example tests indicate as many E modulus values as there are applied loads. This variation of the E moduli should not be surprising, the observed movements are affected by immediate deformation, creep during load-holding, increased volume of soil affected from one applied load to the next, and, primarily, by a significant cementation or preconsolidation condition.

The easiest footing to design is the one that is identical to a tested footing. But, what to do when the footings are of different size and loaded to larger stress? Well, as indicated in Fig. 2, the curves can be approximated to a shape called q-z curve ("q" is stress and "z" is movement) and extrapolated with confidence. A q-z curve can be expressed several ways. One of the most useful is:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{\delta_1}{\delta_2}\right)^e$$

where	σ_1	=	stress No. 1
	σ_2	=	stress No. 2
	δ_1	=	movement at stress No. 1
	δ_2	=	movement at stress No. 2
	e	=	exponent

Any two data pairs of series of load movement data (or stress versus relative movement) that satisfy Eq. 1 can be used to determine the exponent "e". Fig. 2 shows the q z curve determined from the example test, where σ1 and δ1 were selected from the mid-range pair of values, and the σ2 values were applied trying different values of e until the calculated δ2 agreed with the measured. The procedure established an exponent of about 0.40 for the A&M footing tests. This means that we can replace the present, quasi design approach of applying a certain factor of safety to the non existing bearing capacity of footings, and base the design for deformation on a q-z correlation from full scale footing tests.

Where to Go From Here; Deep Foundations

It is generally recognized that there is a similarity between the response to load of a footing and that of a pile toe. This similarity has led the profession to apply the bearing capacity formula also to a pile toe. Usually, the recommended values for the pile toe bearing capacity coefficient, Nt, ranges from two to three times the Nq value of the soil, but values smaller and larger are frequent.

However, there is no more an ultimate resistance for the pile toe than there is an ultimate resistance for a footing. (Pile toe capacity can of course be defined as a toe load for a certain penetration or relative penetration, but as such it has little meaning). This has very clearly been shown in numerous full-scale pile tests using the bi directional pile test, the O cell test, developed by Jorj Osterberg and co-workers. This test measures load-movements of the pile shaft and of the pile toe separately. Fig. 3 shows the results of a test performed on a 900 mm diameter, 15 m long drilled shaft in clayey silt saprolite and socketed a short distance into weathered bedrock. Similarly to the footing tests, the O cell test pile toe load movement follows a slightly curved line and no ultimate resistance is discernable despite the maximum toe movement of 6 % of the pile diameter.

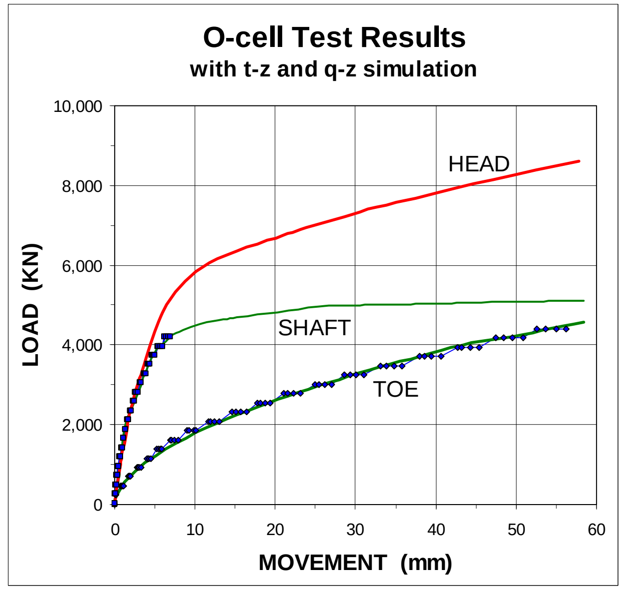

Figure 3 Pile shaft and pile toe O-cell results with t-z and q-z curves fitted to the results, and head-down load-movement curve calculated from the fitted values.

The load-movement of the pile toe can be approximated by a q z curve, and so can the load movement of the shaft, which is then called "t z curve". The fits for the O-cell test are shown in Fig. 3. They are achieved using an exponent of 0.55 for the pile toe data and 0.20 for the pile shaft data. The shaft resistance is determined assuming, conservatively, that the shaft was about to start developing ultimate resistance along its full length. The q z and t z curves are combined to establish the also shown equivalent head-down load-movement curve, incorporating the stiffness of the pile. Although it is an interesting exercise, the pile head load movement curve adds little insight to the assessment to the pile foundation assessment. Apply a larger load and the pile moves down some more. Obviously, the conventional capacity thinking is here of limited relevance.

The more important result of the analysis is the distribution of load along the pile for long term conditions. Figure 4A shows the load distribution, determined from the test data (the test pile was strain gage instrumented) for an assumed sustained load of 4,000 KN. Assume that the soils at the site for some reason will either experience a "large" settlement in the long-term or, alternatively, a "small" settlement, as shown by the "I" and "II" settlement distributions in Fig. 4B. Negative skin friction will develop, of course, and the load will increase down the pile to a maximum at the neutral plane, the location of force equilibrium as well as of settlement equilibrium.

For Case I, the neutral plane will develop at a depth of about 10.2 m. Below the neutral plane, the shaft shear against the pile acts in the positive direction, and, as shown in Fig. 4A, the force at the pile toe is equal to the maximum

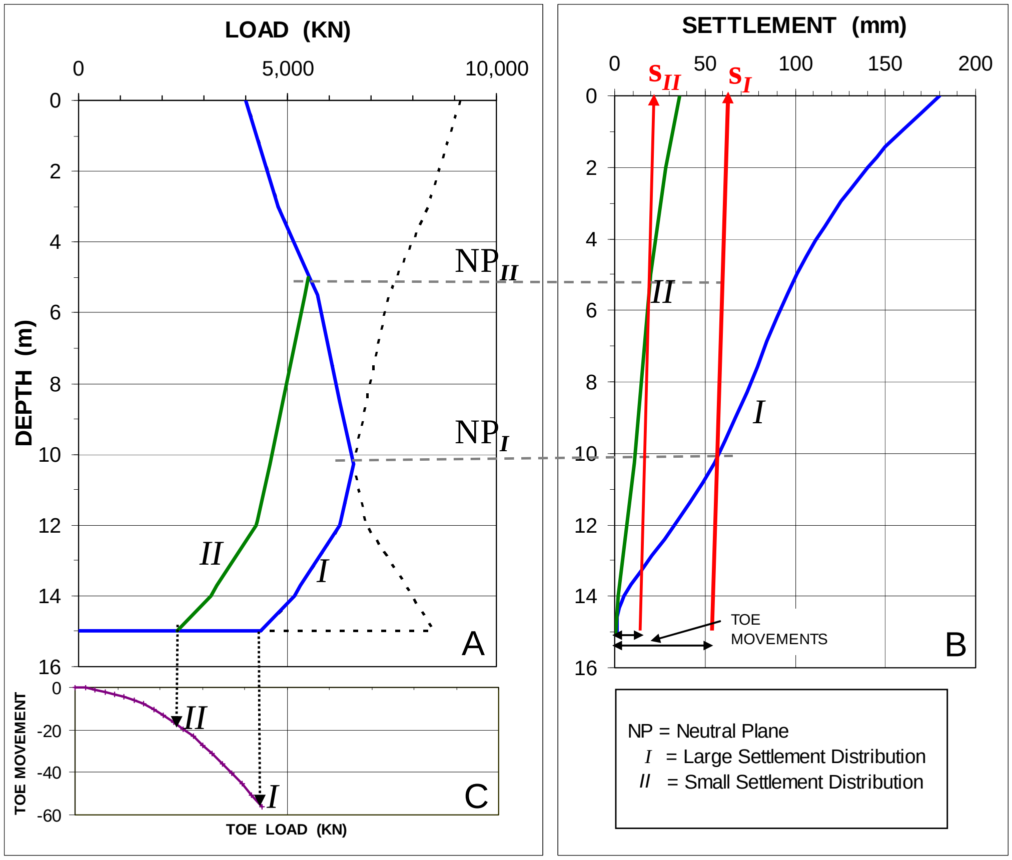

Figure 4 **A**. Distribution of load in the pile for the long-term condition of large (I) and small (II) settlement of the soil around the pile.
B. Pile toe movement measured in the O-cell test.
C. Distribution of two cases of settlement: *I* = "large" and *II* = "small".

O-cell test load. As the measured O-cell load movement diagram (Fig. 4C) shows, the movement of the pile toe is then 55 mm. Figure 4B illustrates that for this toe movement, and considering the shortening of the pile and the shown interaction between forces and movement, the pile head will settle slightly more than 60 mm. If on the other hand the soil settlement is "small" (Case II), then, the neutral plane is located higher up and the pile toe force is reduced to about 2,300 KN, which only requires a toe movement of 16 mm. By the construction shown in Fig. 4B, the pile head will then settle only about 20 mm.

The case history example demonstrates conclusively that what governs the long-term safe function of the piled foundation is the soil settlement at the site. It goes to show that in designing a piled foundation, settlement and soil compressibility at the site can be of outmost importance for the complete design. This is unfortunately not generally recognized in current practice. It certainly will have to be recognized in the future. Note that the analysis requires tests that can separate the shaft response from the toe response. Loading tests that only determine the pile head movement are of limited value for analysis.

Conclusion

The relatively recent shift to load-and-resistance-factor-design, LRFD, has caused consternation and uncertainty about the assuredness of a design in some cases. A check of the design in an analysis for deformation and settlement which is performed with unfactored values—serviceability limit states design—then offers the designer a reassurance needed in our litigious society. Indeed, in the future, capacity will lose its singular importance, and settlement and deformation analysis will be a required feature of foundation design.

Bengt H. Fellenius, Dr. Tech., P.Eng., M.ASCE, is an internationally recognized authority in the field of soil mechanics and foundation engineering, and, in particular, in deep foundations. He can be contacted at: <Bengt@Fellenius.net>.