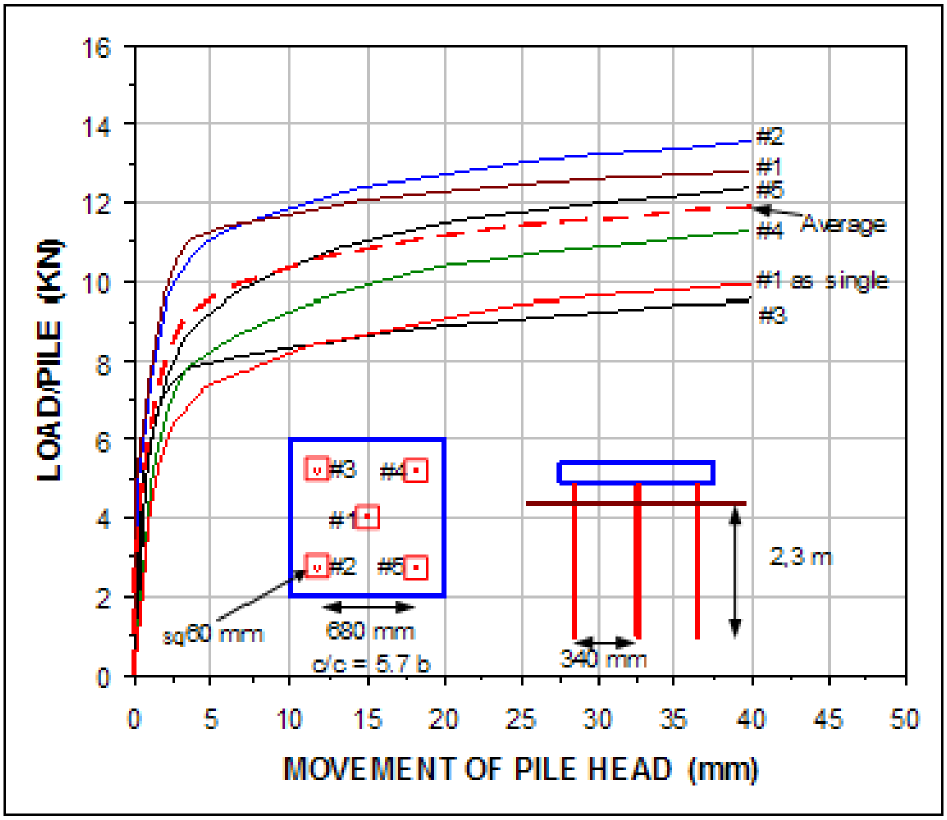

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DISCUSSION

Discussion of "Load tests on full-scale bored pile groups"[1]

Bengt H. Fellenius

The authors are correct in stating for static loading tests that "there are few full-scale … pile group load tests reported in the literature." I agree. However, a few more references are available than those listed by the authors, e.g., O'Neill et al. (1982*a*, 1982*b*), Phung (1993), and O'Neill and Reese (1999). Figure D1 shows the load–movement measured by Phung (1993) in comparing the response of a single pile to a group of five piles driven at a center-to-center spacing (c/c) of 5.7 pile diameters in a fine sand. The center pile (pile #1) was installed and tested as a single pile before the other piles were installed and connected by a rigid cap. The load–movement response of the five piles was different, but the difference was limited to the development during the initial loading. Beyond the first about 4 mm of movement, the load–movement curves were essentially parallel. Measurements of load distribution showed that the difference was mostly due to the difference in shaft resistance — the toe resistances were essentially equal for the five piles — and no correlation to location within the group could be discerned. The differences are considered caused by unsystematic compaction of the sand with no apparent effect of the driving sequence or other driving effect. A main observation was that the response in the loading of the center pile as a part of the group in effect was a reloading of the pile. The response at first loading of the pile as a single pile was considerably less stiff.

Similar to the authors' work, the references mentioned above involve pile groups that at most consist of nine piles. A group of just a few piles — and nine piles is a very small number, where pile groups are concerned — will show minimal interaction and variation between the piles in supporting a structure. For a small pile group, the difference in load response between the piles in a piled foundation subjected to *working load* from the supported structure will be more affected by load variations, such as load center, load inclinations, and lateral loads, as opposed to when subjected to a *static loading test* on the group.

Very few well-documented case histories are available in the literature with regard to full-scale studies of the performance of large pile groups under working load. However, a few are; for example, Golder and Osler (1968), Badellas et al. (1988), Goossens and Van Impe (1991), and Savvaidis (2003). The case histories show beyond doubt that the capacity and the load distribution of an individual pile in a large group of piles is of little relevance to the response of the piled foundation. Instead, the response of a piled foundation made up of a good-size pile group constitutes a settlement problem, and the capacity and load distribution of either an individual pile or the group is not the governing issue for a design.

Despite that the authors (as do so many others) imply that the static loading test measures pile settlement, what is measured in a loading test is a *movement response* to a series of applied loads, *not settlement*. The authors' paper presents the movement response to applied load for a single pile and a few very small pile groups, not the settlement. Of course, settlement assessment relies very much on the results of a static loading test; in particular, on the response of the pile toe. However, the actual settlement of a piled foundation due to a working load, whether composed of a single pile, a few piles, or a large group of piles, is a very different issue.

The authors present the load–movement response of two single piles and state the capacity criterion that the capacity of the piles is based on the "traditional 10% relative settlement criterion". Although the criterion is used less often these days, it does keep appearing in the literature. A couple of years ago, I searched an assortment of successively older papers, textbooks, and standards that essentially stated the same criterion — sometimes with a slight modification away from the 10% value. Many did not give reference to the source, but some did, and I found the original source. The criterion has its origin in a mistaken quotation of a now 70 year old statement by Terzaghi (1942). Terzaghi wrote: "the failure load is not reached unless the penetration of the pile is at least equal to 10% of the diameter at the tip (toe) of the pile." (For full quotation and context, see Likins et al. 2011). Note, Terzaghi did not define the capacity as the load generating a movement equal to 10% of the pile diameter, he emphatically stated that whatever definition of capacity or ultimate resistance used, it must not be applied until the *pile toe* has moved at least a distance corresponding to 10% of the pile toe diameter. (The pile head will then have moved an additional distance equal to the pile shortening.) Most certainly, Terzaghi did not suggest that a fixed movement value, however determined, could serve as a definition of capacity.

Figure D2 shows the load–movement plot of the authors' static loading test on pile DZ1L. The usually very conservative definition called "offset limit", or Davisson limit, indicates a lower-bound value of 1400 kN. It is here offered for reference. I do not suggest that the offset limit would be the pile capacity, but it does show the load for which the ultimate shaft resistance would have been reached. The Hansen 80-percent method results in an interpreted capacity of 1700 kN, coincidentally the maximum load applied in the test — the pile seems to be plunging. The Chin–Kondner and Decourt extrapolation methods indicate 1850 kN. Thus, coincidentally, the 1540 kN capacity per the authors' "traditional" definition happens to be a reasonable value to choose from the load–movement curve. For full definitions and description of the methods for determining the capacity from the pile-head load–movement response, see Fellenius (1975, 2012).

Figure D3 shows the authors' load distributions as evaluated from the strain-gage measurements in pile DZ1L up to a load of 1440 kN. The authors did not include the distribution for the maximum applied load (1700 kN). I have supplemented the figure with the authors' q_c diagram from the sounding pushed nearest the test pile, soil descriptions, and layer boundaries. I have also added a distribution determined by means of both total stress and

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Fig. D1. Load–movement response of a five-pile group (data from Phung 1993). *b*, pile diameter.

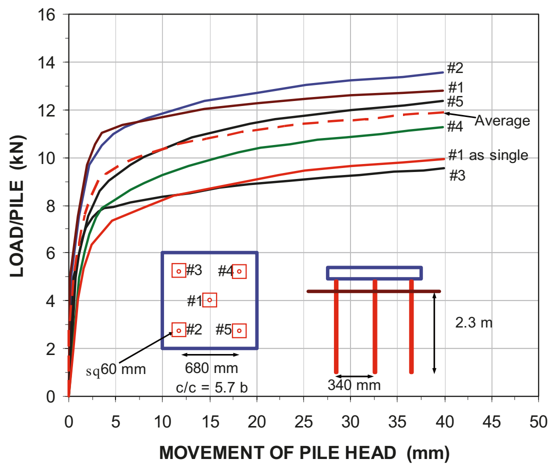


Fig. D2. Load–movement curve of test on pile DZ1L. R_u, ultimate resistance.

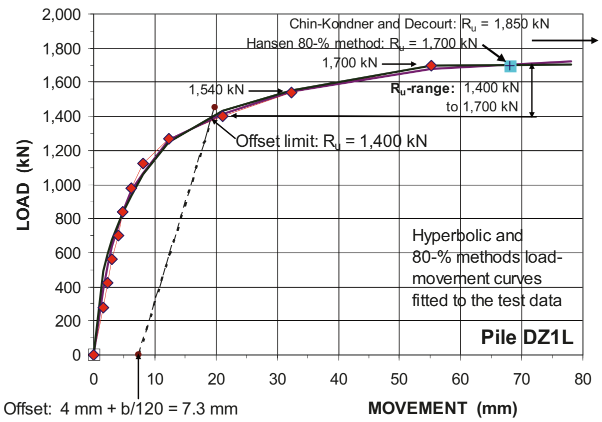
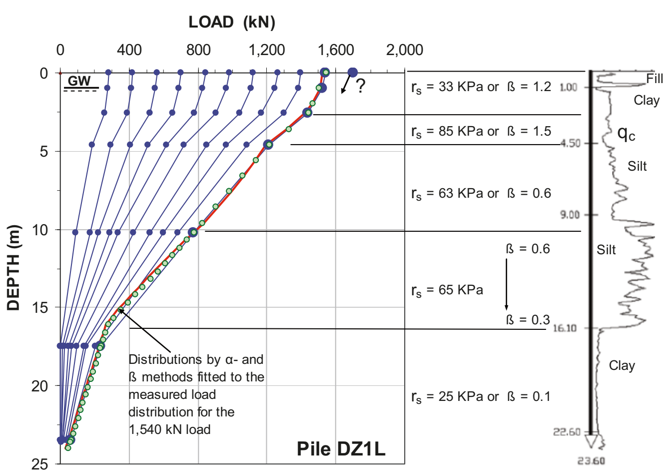
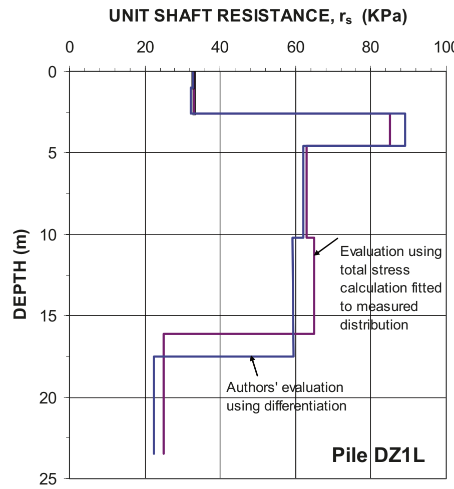


Fig. D3. Pile DZ1L load distribution. GW, groundwater level; r_s, unit shaft resistance; q_c, cone stress.



effective stress calculations back-calculated to fit the distribution at the 1540 kN applied load. The total stress values of the average unit shaft resistance, r_s, and the β-coefficients I used to achieve the fit are shown to the left of the q_c diagram. For the calculations, I used the UniPile program (Goudreault and Fellenius 1998).

Fig. D4. Evaluated distributions of unit shaft resistance for pile DZ1L.



The authors differentiated the loads determined at the strain-gage levels and determined the average unit shaft resistances between the gage levels. This method requires that the strain measurements be accurate, which seems to have been the case for this test. When the accuracy is less good, the inaccuracies will be enlarged by the differentiation. The alternative of evaluating the shaft resistance by fitting calculations to a load distribution makes for results less dependent on inaccuracies. Moreover, when the gage levels are not at the layer boundaries, as is the case at the 16.1 m boundary level and 17.5 m gage level, and the shaft resistances in the layers are different, the differentiation method will be somewhat distorted. By evaluating the shaft resistance between the layer boundaries as opposed to between the gage levels, the potential distortion is avoided.

Figure D4 compiles the authors' unit shaft resistance values obtained by differentiation and those I have obtained from the total stress calculation fitted to the measured distribution. The values of unit shaft resistance by the two methods agree quite well where layer boundaries and gage levels are at the same depth, but they deviate where the gage levels and boundaries are not.

The authors discuss the interaction between piles in a pile group by comparing the load–movement results of a single pile to that of the piles in the group (where the pile head loads were measured individually). The authors state that the pile caps were cast and "rested on the ground." If indeed the pile caps were in contact with the ground during the tests on the pile groups, this would have added some resistance and stiffness to the group tests. I wonder if the contact stress was measured and, if so, how large it was.

Moreover, there does not seem to have been any measurement of the compression of the clay below the pile toe level. The two 9-pile groups have a footprint area of about 10 m² and the stress produced by the maximum applied load distributed over that area was therefore about 1500 kPa. The applied test load produces shaft resistance that is transmitted downward through the soil, and although it would be somewhat dispersed laterally, a good portion of it will reach the pile toe level together with the toe stress (which was small). Although the authors do not provide details of the clay, I would expect that the measured pile head movement for the pile group will have experienced some additional movement due compression of the clay below the pile toe level. This would have appeared as a reduced stiffness for the group piles as opposed to the single pile even for the case of shaft and toe resistances and toe movement being equal for a group pile and a single pile. And it would, therefore, explain part of the authors' observed stiffness differences between single piles and group of piles.

The load–movement response of a shaft bearing pile group is not just governed by the soil shear strength. (The test piles at the subject site were essentially shaft bearing and the test on pile DZ1L showed a mobilized shaft resistance of about 1400 kN.) The buoyant weight of the soil in between the piles has a moderating influence on the pile stiffness response, depending on the spacing between the piles. Once the buoyant weight of the soil between the piles placed in a group is smaller than the shaft resistance for a single pile, the amount of shaft resistance available to a pile inside the group becomes correspondingly limited. The center pile of the 24 m 9-pile group has a share of the soil weight equal to the square of the spacing minus the cross section of the pile times the effective stress at the pile toe. The effective stress at the 24 m pile toe level was about 240 kPa. Thus, the share of soil weight for a 400 mm diameter pile inside the group of piles spaced c/c 3.0 diameters is about 300 kN. That is, at such spacings, when the shaft resistance demand becomes larger than 300 kN, there will be interference between the piles, resulting in a softer shaft response for the interior piles in the group as opposed to that of a single pile. Had the spacing been about twice larger, as for the case shown in Fig. D1, this "buoyant weight" influence would have been minimal.

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