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Case Study

# Revisiting Okabe (1977), Cooke *et al.* (1981), Hansbo (1984), and Russo and Viggiani (1995) - The response of a wide pile group to load

**Bengt H. Fellenius[1]***

**Abstract:** Okabe (1977) reported response of instrumented piles in a wide pile group in subsiding soils and showed that the response of the perimeter piles was similar to that of single piles and that interior piles were unaffected by either drag force or shaft resistance. As the pile toes were in soft soil, the toe resistance was small for both perimeter and interior piles. Ordinarily, perimeter piles in a wide group will carry more load than interior piles. With time, downdrag on the perimeter piles will add force and soften the pile re-sponse, which means that some load is transferred to the interior piles. In the long-term, the interior piles may even have to carry larger load than the perimeter piles. Cooke *et al*. (1981) reported a case history of a wide group in overconsolidated soil and showed that the perimeter piles received larger load than the interior piles. The pile toe resistance was considerable and the observations indicated that the interior piles engaged the shaft resistance upward from the pile toe level. Hhigher up along the pile, the response was similar to that of the piles in the Okabe (1977) case, i.e., minimal shaft shear. Hansbo (1984) compared the response of two adjacent piled foundations subjected to very similar stress, 60 kPa and 66 kPa, respective-ly. One was conventionally designed with large safety factor and the other was designed considering the response to be that of raft and piles combined. The first had twice as many piles as the other. Observations over 13 years showed that the settlements of the two buildings were equal indicating that the ratio of load to ultimate bearing on a single pile is irrelevant to the response to load on a wide pile foundation. Russo and Viggiani (1995) confirmed the observation of the perimeter piles having a stiffer response as opposed to the interior piles, somewhat lessened by the presence of general subsidence.

***Keywords: pile group, force distribution, interior piles, perimeter piles, drag force, contact stress***

## Introduction

The geotechnical literature includes a large number of case histories reporting results from static loading tests and long term monitoring of single piles. However, case histories on monitoring the response of pile groups to load are rare, in particular those addressing wide pile groups, i.e., groups at least 4 pile rows wide. Indeed, the literature contains no more papers on wide pile groups than can be counted using one's fingers. Depending on the requisite level of quality and completeness of the data, one hand might even suffice. This is understandable. Most pile case histories are expected to address bearing "capacity" in one form or other and full-scale pile groups would be costly or impractical to test to a bearing "capacity" level of load. However, it is not generally realized that the response of a pile group to the actual load from the supported structure is much more worthy of attention than a perceived "capacity".

A few researchers have realized this and presented milestone quality papers. Four of them are addressed herein in the sequence of when they first appeared. The case histories complement each other and point to what model to use in analyzing the response of a wide pile group to load from the supported structure.

**Okabe (1977)** monitored force distribution in two instrumented, 600 mm diameter, single closed-toe pipe piles installed to 47 and 42 m depth, respectively. The piles, Piles 1 and 2, served as reference to several interior and perimeter test piles in a wide octagonal pile group supporting a bridge pier and comprising thirty-eight 700 mm diameter piles, driven to 41 m depth. The intended sustained load for the foundation piles was 300 kN/pile.

The force distribution in the two single piles was monitored over 1,663 and 807 days, respectively. After 550 days, a sustained load of 500 kN was applied to the head of Pile 2, and after an additional 63 days, the sustained load was increased to 1,300 kN. Figure 1 shows the measured force distributions for the two piles. Because the pile toes were in soft soil, the toe resistances were small. The primary purpose of the single-pile monitoring was to study the development of

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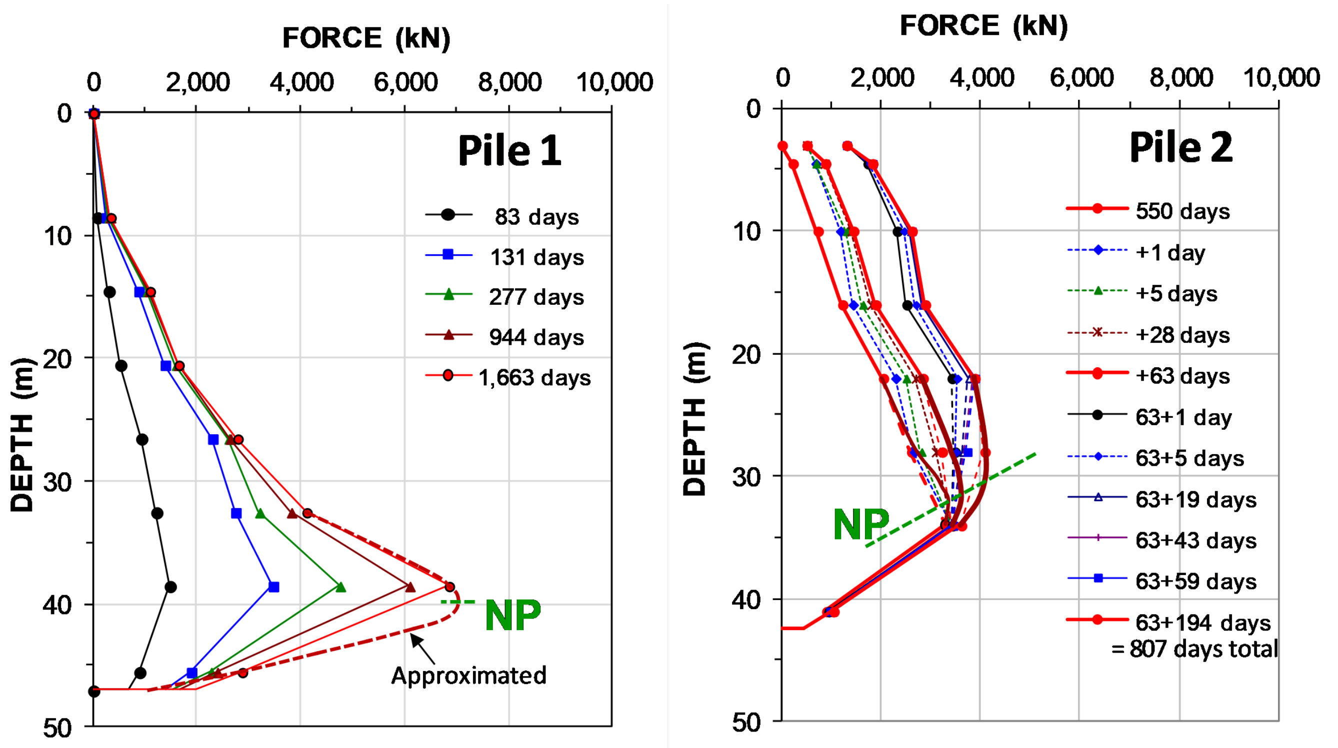


Figure 1. Force distribution in the single piles, Piles 1 and 2 (data from Okabe 1977)

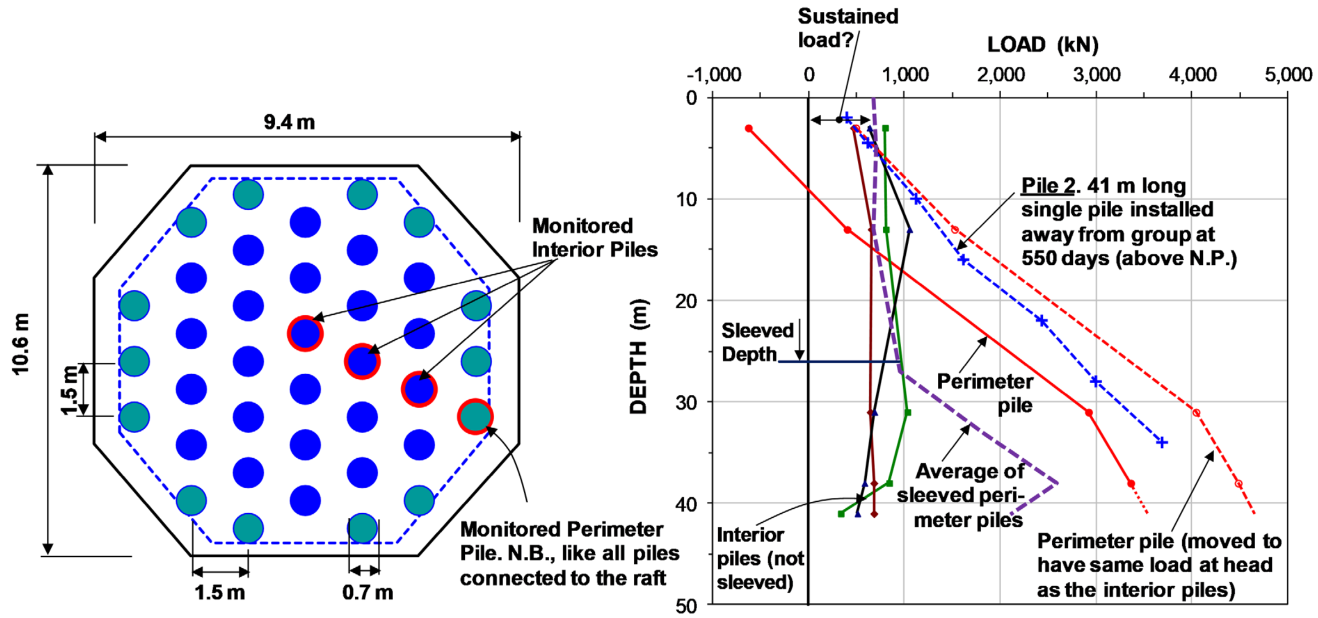


Figure 2. Pile group layout and measured force distributions (data from Okabe 1977)

drag force and effect of sustained load. The records showed that the level of the neutral plane rose when the sustained load was increased for Pile 2.

The piles in the pile group were installed at an equilateral 1.5 m spacing. The Footprint Ratio, FR, the ratio between total pile area and total raft area, was 14 %. The sustained load was 300 kN/pile. The raft (the thickness was not reported) was placed at about 2 m depth. The soil profile was a more than 40 m thick deposit of compressible silt and clay underlain by sand. The silt and clay had a water content close to the Liquid Limit ranging from 40 to 60 %. A fill, placed on the ground (no information of fill height) over a vast area of the site and water mining in sand created general subsidence amounting to about 100 mm/year.



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Four piles in the pile group were instrumented for monitoring axial force: three interior piles and a perimeter pile immediately outside the three. Figure 2 shows the layout of the pile group. An outer casing down to 26 m depth was placed over the six other perimeter piles. The purpose of study was to measure and investigate means to reduce or eliminate the drag force expected to be caused by the subsiding soils.

The distributions of force were monitored in the instrumented group piles over 1,040 days. The distribution measured after 550 days in Pile 2 ( the 600-mm diameter single pile away from the group) has been added with the distribution increased by ratio of the pile diameter to produce a direct comparison to the distributions for the 700-mm group piles. Raft movements and settlements were not stated to have been monitored.

The measured force at the pile head shows that the load on the interior piles was larger than that on the perimeter piles. Indeed, the load on the latter was negative due to the downdrag from the subsiding soil. The about 500-kN/pile apparent sustained load on the interior piles is likely the combined result of the about 600-kN/pile pull force due to the downdrag and drag force on the eight not-sleeved perimeter piles, resulting in an about 200-kN/pile increase of the load from the bridge on the 24 interior piles.

Comparing the response of the not-sleeved perimeter pile to the Pile 2 single pile shows that the perimeter pile was affected by the general subsidence in much the same way as the single pile. Both the perimeter piles and the single pile indicated minimal toe force and minimal shaft resistance immediately above the pile toe. Comparing the force distribution of the not sleeved perimeter pile to that of the sleeved perimeter piles shows that the sleeve effectively prevented shaft shear from developing along the pile. However, below the sleeved depth, the sleeved piles developed an equilibrium between the negative and positive direction shaft shear. No difference was observed in regard to negative skin friction and positive shaft resistance between the not-sleeved and sleeved interior piles.

The Okabe (1977) paper added to the then growing library of measurements of drag force on single piles, which in the 1960s and 1970s was the main bogeyman for pile design. Settlement, although recognized as the cause of drag force, did not attract much attention. This is probably the reason for why the Okabe (1977) neither included settlement observations for the single piles nor for the pile group. At the time of the study, the effect of subsiding soil on piles was not understood. It was thought that the drag force was the issue, while the real issue, the downdrag, i.e., settlement, was overlooked. Okabe (1977) drove the conclusion that the study had shown two means of eliminating drag force: sleeving piles or adding "sacrificial" perimeter piles. (If the latter, in contrast to the test piles, sacrificial piles must not be connected to the pile raft). That neither solution is practical or economical does not diminish the value of the factual observations of the case history.

The important main contribution of Okabe (1977) was first showing that perimeter piles in a pile group responded much the same way as single piles and, second, that the interior piles in a wide group are neither affected by negative skin friction nor positive shaft resistance. The explanation to the latter had to wait until Franke (1991) suggested that interior piles engage the soil from the pile toe upward in contrast to perimeter and single piles. Fellenius (2019; 2025) proposed an analytical method for the interaction between the interior and perimeter piles with the raft, the piles, and the soil (modeled on the upward and downward response to a bidirectional test, with the cell placed at the pile toe).

**Cooke *et al.* (1981)** reported observations from 6 years of monitoring load and settlement of a wide pile group comprising bored piles supporting a 16-storey apartment building in London clay at Stoney Park. Over the years, the paper has been cited by many in regard to contribution of contact stress to the bearing of a piled foundation. However, it also contains many noteworthy and enlightening additional observations.

The soil profile consisted of overconsolidated London Clay with an undrained shear strength of about 200 kPa and total density 2,100 kg/m$^3$. Prior to the pile installation, the building footprint area was excavated to a 2.5 m depth, the groundwater table, for a full raft area basement.

A static loading test was carried out at the onset of the pile installation on one of the piles (its location in the group was not stated). The test pile was not instrumented. The test schedule included an initial "incremental test" comprising four 135-kN increments to 540 kN with an about 30-minute load-holding and unloading after about 5.5 hours followed by a constant-rate-of-penetration (CRP) test to 1,600 kN. Figure 3 shows the schedule of the incremental test and pile-head load-movement records.

The load-movement recorded for both tests is shown in Figure 4. The CRP rate was not stated, but was presumably 0.5 mm/minute, a commonly employed rate at the time. Therefore, the CRP test to the 6.3-mm total movement would have lasted about 13 minutes. The measured load-movement is supplemented with a simulation of the test based on input of the same beta-coefficient (ß = 0.3) as fitted to the corner pile at 16-month monitoring event (addressed below). The target pile-head load and movement for the analysis and simulation are 1,600 kN and 6 mm, respectively. The pile-head load-movement target was combined with t-z and q-z functions and a target toe force fitted to the measured load-movement curve. The simulation fit required assuming strain-softening shaft resistance after the target movement and a large toe resistance with a strong early rise—an indication of presence of residual force. The lack of measured toe response, or of axial force near the pile toe, makes the particular combination of beta-coefficients and toe resistance at the target movement, and associated t-z and q-z functions, only one of many possible.

The building raft, 43 m long, 20 m wide (860 m$^2$), and 900 mm thick, was placed at the 2.5 m excavation level and supported on 450 mm diameter bored piles, rectangularly spaced at c/c 1.60 m in 13 columns and at c/c 1.63 m in 27 rows, to a total number of 351 piles, all with 13 m embedment length below the raft. The footprint ratio, FR, was 6.2 %. The piles were uncased, the concrete had a cube strength of 255 kPa. The upper 3 m length of the piles was reinforced by four 16 mm bars (to connect the piles to the raft). The piles were

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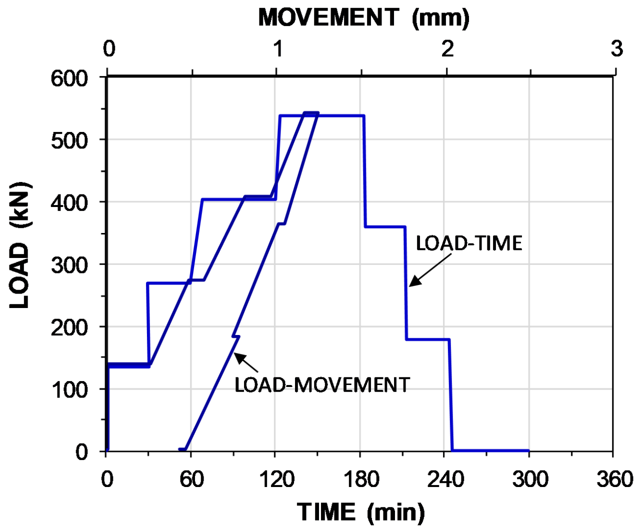


**Figure 3. Incremental test: Load-movement and Load-time**

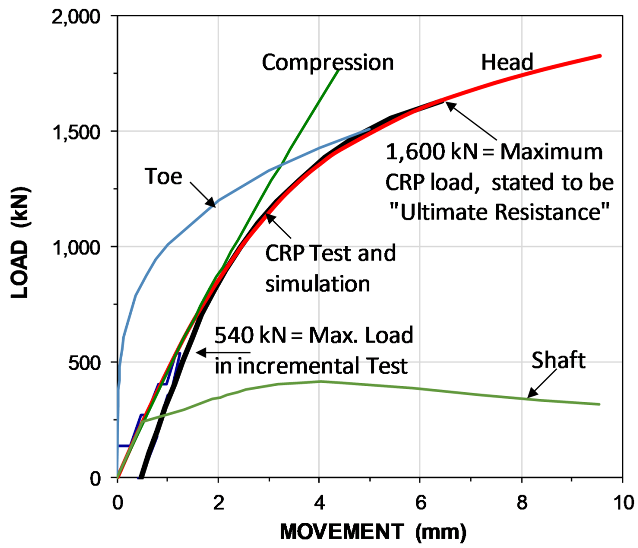


**Figure 4. Load-movement curves of incremental and CRP tests—measured and simulated to fit**

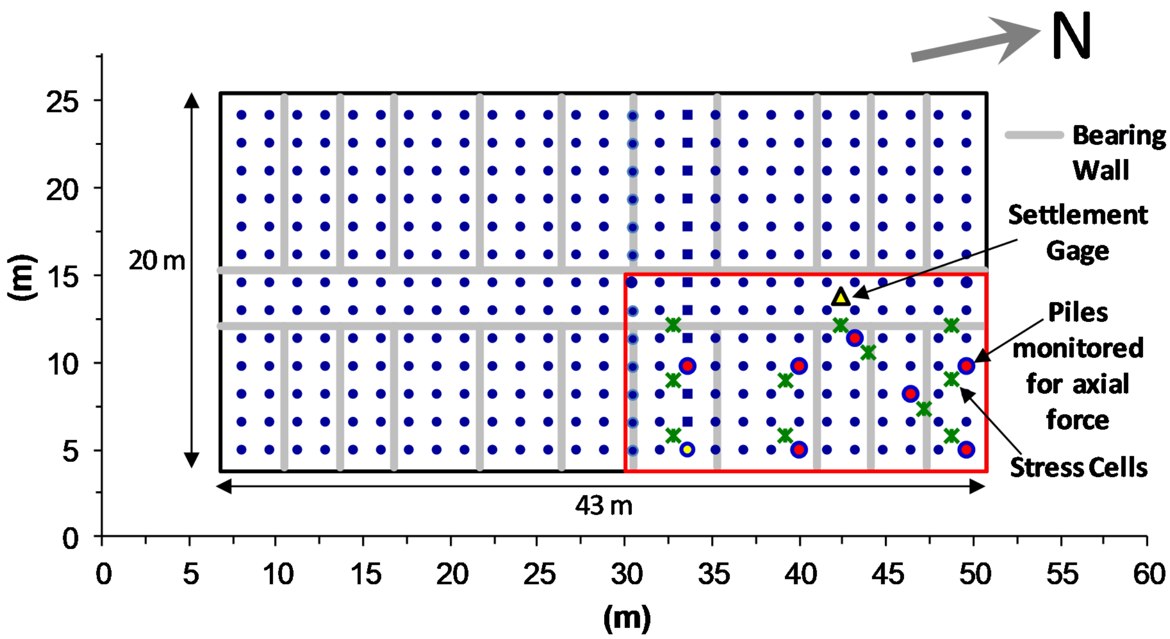


**Figure 5. Raft plan with bearing walls and piles (After Cooke *et al.*, 1981)**

installed during January and February 1973. The building was completed March 1975 and fully occupied in the Fall of 1975. Full load was 155.6 MN, including a 18.6-MN raft weight, correlating to stresses of 180 and 20 kPa over the full raft footprint, respectively, and to an average of 440 kN/pile and about 50 kN/pile, respectively. As the perimeter piles were expected to receive a load larger than the average load, the structural pile design was based on an unfactored load of 565 kN/pile.

The building was designed with a perimeter and interior walls as bearing walls. Figure 5 shows the layout of the walls and piles. Eight piles—one corner pile, three side piles, and three interior piles—were instrumented with pneumatic load cells for monitoring axial force at pile head and pile toe during and after the construction over two years (until full occupancy). Seven of the instrumented piles gave good records, one side pile (open circle in the graph) did not. The location is also shown of eleven earth stress cells for measuring contact stress and one deep settlement station for multiple points down to 35 m depth.

Figure 6 shows the pile-head loads monitored during and after the construction for the seven instrumented piles with average of corner piles, side piles, and interior piles. The load at both the head and toe was measured with a 360 mm wide,


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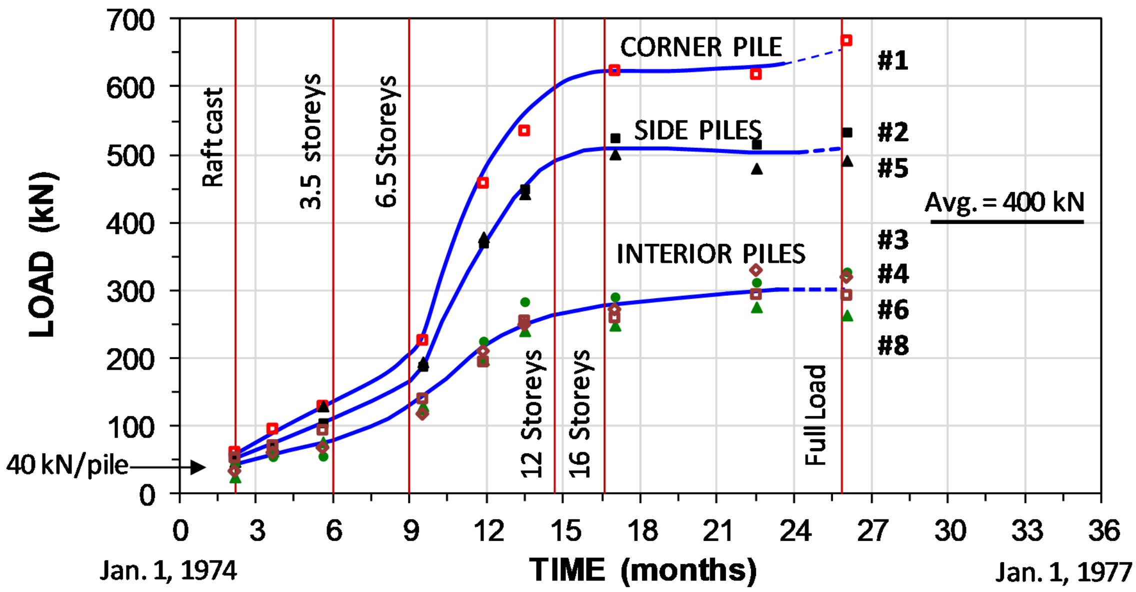


**Figure 6. Load at head of monitored piles over two years (Fig. 8)**

100 mm thick cell cast in the pile that measured force using eight cylinders between two steel plates with vibrating-wire gages for determining imposed strain that was calibrated to force. Considering that the nominal total load corresponded to an average of 440 kN/pile, the difference to the 400 kN/pile monitored total average load implied a total contact load of 14 MN, correlating to 16 kPa contact stress (40×351÷860). Assuming that the piles have a 30-MPa E-modulus (axial stiffness $E_{pile}A_{pile}$, becomes 5 GN), the axial pile strain was 80 με. The soil underneath the raft, engineered fill or natural London clay, likely experienced the same strain. The 80-με strain and the 16-kPa stress correlate to $E_{soil}$ = 250 MPa, which is a realistic value for both an engineered fill and the overconsolidated London clay.

The contact stress was measured at eleven gage locations. The cells converted strain measured in steel rods between two 350-mm circular plates. Five stress cells were located amongst the interior piles and the monitored contact stress ranged from about 40 to 100 kPa at full load (17 months). The Cooke *et al.* (1981) discussion about what the measured values represented is a bit confusing and the paper does not present the individual measurements of the eleven points of contact stress measurements, only the derived contour lines. Toward the side of the raft, the stresses increased and, at the cell nearest the corner, the stress was about 200 kPa. Thus, the measured contact stresses were about ten times the stress determined from the difference between the nominal total load and the sum of the pile loads. I am not surprised. Measuring earth stress with a pressure cell that possesses a different stiffness to the material they are embedded in does not work well. An induced stress change will result in a strain change in the soil and only if the cell and the soil has equal stiffness will the cell register the stress change correctly—if the cell's stiffness if different to that of the soil, the cell reading will be off. That is, if the cell, and the material placed immediately

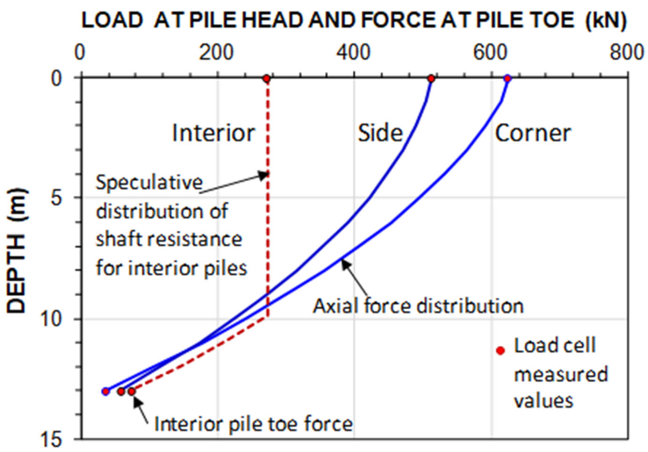


**Figure 7. Load measured at head and toe of monitored piles at 17 months (full 16 storeys)**

around when it was installed, is stiffer than the soil, the cell will report a larger than actual stress and, *vice versa*, report a smaller if the cell is softer. Earth stress cells should be designed to measure the shortening between two adjacent levels without requiring a spring force. The shortening is then used to calculate the strain and obtaining the stress by correlation of the strain to the soil E-modulus determined by other means or estimated. It is obvious that the stress values "measured" by either method will be rather imprecise. Note, also, that the strain is only that of the soil wherein it is measured, that is, the strain of the engineering fill (usually) immediately under the raft. The contact load (and stress) taken as the difference between the total load and the sum of the loads measured on the piles would appear to be the more reliable estimate.

The round dots in Figure 7 show average values of measured pile-head loads and pile-toe forces for the seven

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instrumented piles monitored at 17 months (full 16 storeys). The dots representing the pile-head load and pile toe force of the corner and side piles are connected with curves, implying axial force distribution (calculated using effective stress with a constant beta-coefficient for the side and corner piles). When the pile toe of the interior piles penetrated the soil, the shaft resistance mobilized upward from the pile toe will have been similar to those of the perimeter piles (side and corner). The so produced axial force cannot have been larger than the applied load. Thus, the dashed distribution line indicates the distance above the zone affected by the pile-toe and shaft resistance response to the applied load. In this regard, the Cooke *et al*. (1981) and Okabe (1977) both show agreement with the Fellenius-Franke principles of response to load by interior piles in a wide pile group. The distribution can be simulated as a bidirectional (BD) test with the BD-cell placed at the pile toe. The downward and upward BD loads at equal movement combine to be the applied pile head load. Unfortunately, the pile toe movement was not monitored along with the pile toe force.

Moreover, in contrast to the observations of Okabe (1977), the measured load on the perimeter piles was larger than that on the interior piles. Cooke (1986) stated that "*for the most common spacings, the corner piles can be expected to carry at least twice the load of interior parts and side piles at least 1.5 times the load on interior piles*". The statement is a conclusion from the typical conditions of pile groups in the UK: relatively small ratio of pile length to raft width, overconsolidated soil, no general subsidence, and rigid pile rafts. The Okabe (1977) case history showed that for different conditions, such as general subsidence, soft compressible soil, and/or piles longer than the foundation width, a very different distribution of load between interior and perimeter piles can result

Figure 8 shows the average total load and load/pile of all seven monitored piles as the structure was being built. Initially, the "raft only" load (added to the diagram in the paper) was likely all contact load.

Figure 9 shows six years of soil settlements measured at different depths at the single settlement station. The settlements originated in the clay below the pile toe level consolidating under the loads from the building.

Figure 10 shows the distribution of soil settlement with depth plotted from combining the data of previous two figures. The diagram is supplemented with the estimated pile compression and average measured toe penetration. The 10-mm settlement indicated below the pile toe level at 13 m depth corresponds to a Janbu modulus number of 800 for the clay. The paper included no mention of the reason for the settlement in the upper 13.5 m layer, that is, of the "pier" comprising soil and soil and piles, and said nothing about the data showing smaller ratio of settlement (steeper slope) in the soil within the first about 2 m thick zone under the pile-toe than deeper down. The implied toe penetration is much too large (as denoted for the 72-month records). I believe the large values of settlement within the pile group and the implied large toe penetration are measurement errors. The dashed lines show the more plausible distribution below the pile toe level. They demonstrate a more realistic toe penetration and development of settlement below the pile toe level. The change lowers the back-calculated Janbu modulus number to 500 for the clay.

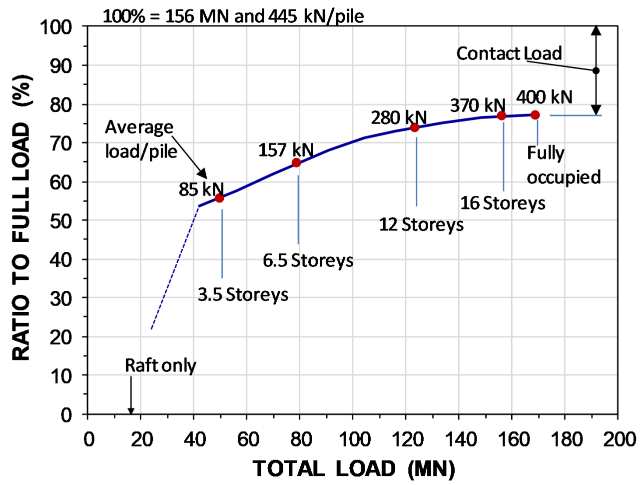


**Figure 8. Ratio of average monitored load to total load as the structure was being built**

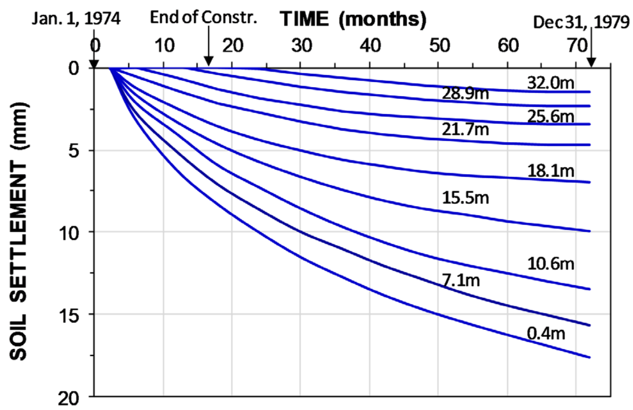


**Figure 9. Soil settlement at nine depths monitored over six years**

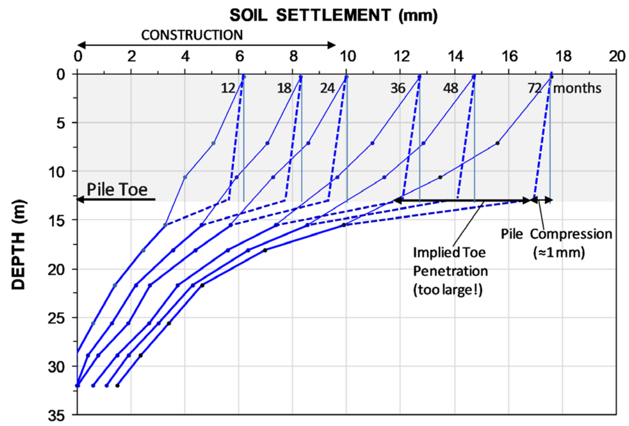


**Figure 10. Distribution of soil settlement at different times**



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The Okabe (1977) and Cooke *et al*. (1981) care studies showed that the response of the interior piles in a wide group cannot be calculated as that of a series of single piles mobilizing shaft resistance starting from below the pile raft and, eventually, mobilizing toe resistance. Instead, the mobilization of the resistance has to commence from the pile toe and progress upward with the analysis considering that the upward movement of the soil is equal to the penetration of the pile toe (downward movement) as in a bidirectional test.

**Hansbo (1984)** reported a case history monitoring the response of two adjacent four-storey apartment buildings in Göteborg, Sweden, both supported on piled foundations. The soil at the site was a thick deposit of soft clay with a water content of 60 to 80 % and a Liquid Limit of about 60 %. The clay was very compressible; the Janbu modulus number was about 5. Building 1 was constructed on a grillage of concrete beams (contact area was not reported) and Building 2 on a 400 mm thick raft. Both foundations were cast on engineered fill. The footprint areas of the buildings were 700 and 900 $m^2$, respectively. The foundation piles comprised an upper 8 m length of square 275 mm precast concrete pile extended by a wood pile to 26 m depth. Building 1 was supported on 211 piles under the grillage beams and Building 2 on 104 piles evenly distributed at about 3.0 m spacing. The Building 2 pile group had a width of five rows, which places the pile group at the border line of narrow to wide pile group. The footprint ratios, FR, were 2.3 and 0.8 %, respectively.

The nominal total average load over each building footprint corresponded to 66 and 60 kPa, respectively—quite similar values. The estimated average sustained loads for the two designs were 220 and 520 kN/pile, respectively—quite different values. The conservatively estimated pile "capacity" was stated to be 330 kN/pile. At the end of construction, the average measured pile loads were about 150 and 280 kN/pile for the two buildings, respectively. The differences, 70 and 240 kN/pile, respectively, between measured load and calculated nominal sustained load can be assumed to represent contact load.

For Building 2, the 240 kN difference in contact load correlates to 28 kPa average contact stress, reasonably close to the average measured contact stress at Building 2 of about 40 kPa. The 280 kN axial force at the pile head combined with about 30-GPa E-modulus correlates to about 100 $\mu\varepsilon$ axial strain. That strain combined with a 40 kPa contact stress correlates to a $E_{soil}$-modulus of 400 MPa; large even for the engineered fill, but far from commensurate with the E-modulus of the soft clay below, which means that, in the clay layer, much of the "contact load" would have been transferred to the pile.

Figure 11 shows that the buildings settled on average about the same amount, about 40 mm, over a 13-year period. The calculated equivalent-pier shortening was smaller for Building 1, because of its smaller average pile load, toe penetration, and, larger pier EA-parameter, but because of its larger average stress over the footprint, this difference was compensated by the settlement below the pile toe level being larger.

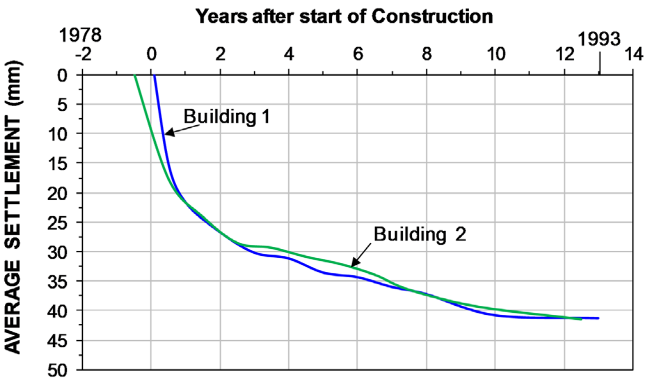


**Figure 11. Settlement measured for the two buildings over 13 years (Hansbo 1993)**

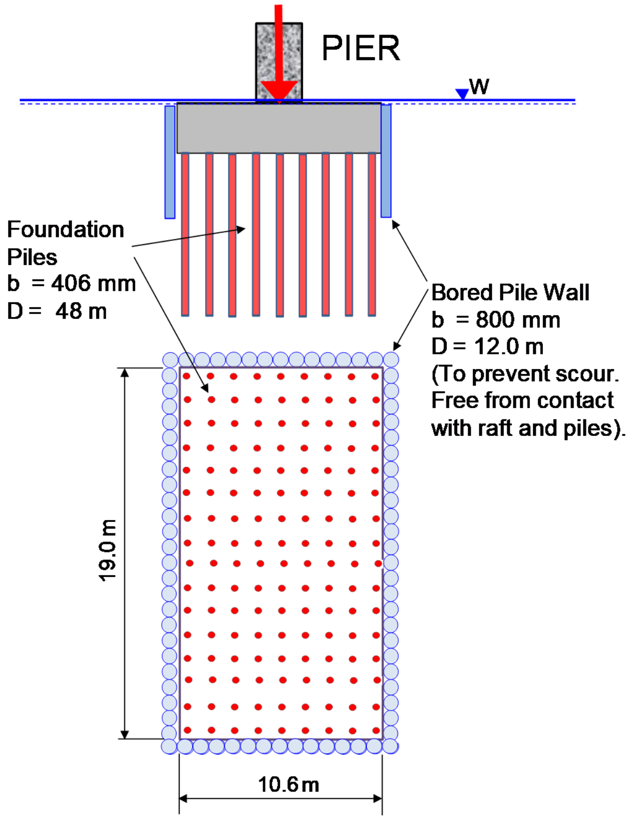


**Figure 12. Piles for the Garigliano River Bridge. Section and plan (after Russo and Viggiani 1995)**

The case history indicates very clearly that, for a wide piled foundation, the bearing of a single pile is irrelevant to the foundation response to the supported load.

**Russo and Viggiani (1995)** presented a case history of a wide piled foundation supporting the main pier of a cable-stayed bridge over the Garigliano River in Southern Italy constructed in 1991-94. Figure 12 shows the section and plan of the pier and layout of the piles.

The soil profile consisted of about 10 m of clay on about 10 m of dense sand underlain by normally consolidated soft

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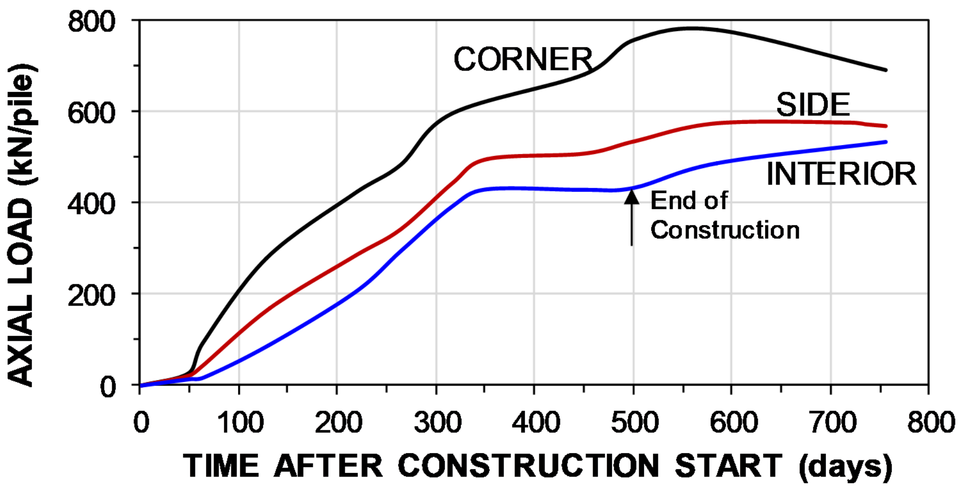


**Figure 13. Axial pile loads measured during and after construction (Russo and Viggiani 1995)**

clay deposited at about 48 m depth on a very dense sand and gravel bed. A small regional subsidence affected the area. The piled foundation comprised 144 mandrel-driven, then concrete-filled, steel pipe piles, 406 mm diameter, 48 m long, uniformly distributed over a 3 m thick raft 10.6 m by 19.0 m. The piles were driven into the very dense sand and gravel layer. The pile configuration was rectangular, comprising 9 rows and 16 columns, and the pile c/c distance was 1.2 m (3.0 pile diameters). The footprint ratio, FR, was 9.2 %. Enveloping the raft, a wall of 800-mm diameter bored piles to 12 m depth was constructed to protect against scour. This pile wall was free from contact with the raft and the pipe piles. The nominal unfactored load from the pier was 115 MN (800 kN/pile), which corresponded to a factor of safety of 3.0 on a pile capacity stated to have been determined in a static loading test.

The foundation was instrumented to monitor the pile axial load in 35 piles and the contact stress between the raft and the soil in eight earth-stress cells, as the bridge was constructed and over about 10 months afterward. Figure 13 shows that interior piles under the raft carried 60% of the load carried by perimeter piles after the construction of the bridge had been completed.

The softer response of interior piles is because the rigid raft could not adjust to the bowl-shaped deformations resulting from the fact that piles in the center settle more than piles at the periphery of a loaded raft. After construction, the general subsidence imposed drag force and downdrag on the perimeter piles, softening their response. It is likely that the development of downdrag was lessened by the presence of the enveloping pile wall. With time after end of construction, the needed balance of force and settlement (including axial compression) resulted in a decreased load on the perimeter piles (side and corner) and a corresponding increase of load on the interior piles. As the figure shows, the post-construction interactive effect was prominent for the corner and interior piles. The total load on the pier did not change.

The authors did not report raft-soil interface (contact stress) measurements in the eight earth stress cells. The study did not include measurements of settlement.

The Russo and Viggiani (1995) records show similarly to the Cooke *et al*. (1981) records that the perimeter piles take on a larger load than the interior piles and, as also reported by Okabe (1977), that presence of general subsidence will reduce that difference.

## Conclusions

The technical literature includes a few additional important case histories, for example: Auxilia *et al*. (2009), Badellas *et al*. (1988), Broms (1976), Fellenius *et al*. (2019), Georgiadis *et al*. (1989), Kakurai *et al*. (1987), Liew *et al*. (2002), Mandolini *et al*. (2005), Savvaidis (2003), and Yamashita *et al*. (2011). These support the observations in the here revisited papers. There are also papers ostensibly addressing wide piled foundations that do not present a clearly defined layout of the pile group, contain limited reference records, or, at times, even any, or concentrate on numerical analytical procedure rather than of the records.

The four case histories demonstrated that a perimeter piles responded much the same way as single piles to an applied load: the bearing in the form of shaft plus toe resistance followed the same rules and that the response is mobilized from the pile head downward for both. Also in regard to the effect of drag force and downdrag from general subsidence, the response of the perimeter pile was similar to that for a single pile. In contrast, the interior piles responded very differently to the single pile. As first stated by Franke (1991), the interior piles response to load was from the pile toe upward (Fellenius 2019; 2025) and showed neither negative nor positive resistance until close to the pile toe.

The revisited case histories showed convincingly that design of a wide piled foundation cannot be analyzed on bearing response—ultimate limit states, but must be based on analysis of settlement—serviceability states. The settlement is the sum of the axial compression of the "pier" comprising


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the soil and the piles, the pile toe penetration, and the compression of the soil layers below the pile toe level. The first is approximated to the equivalent pier compression—compression of pile and soil within the pile head and the pile toe for the full amount of applied load to the raft. The second is determined from the q-z function of the pile-toe mobilization—known from a static loading test or experience from previous work and tests in the area. The third is the settlement determined in conventional calculation addressing an equivalent raft at the pile toe level—knowledge of the soil parameters below the pile toe level is a necessity, of course, as is incorporating the effect of other changes, if any, to the effective stress underneath the pile toe level.

Indeed, the average settlement of a wide piled foundation depends on the applied stress, the compressibility and length of the pile-soil body (the pier, which E-modulus is the pile footprint ratio times the pile E-modulus), the pile-toe penetration, and the compressibility of the soil below the pile toe level.

The case histories showed that an analysis needs to consider special aspects, such as the presence and, if so, effect of general subsidence, the effect on the rigidity due to pile raft thickness, footprint ratio, and influence of bearing walls on the raft. It is particularly important to consider the pile length and toe stiffness. Long interior piles will have the shaft resistance engaged along a relatively shorter distance above the pile toe. Short interior piles may have the full distance to the raft engaged by shaft resistance, more so if the pile toe response is soft (small toe stiffness), which affects the compression of the equivalent pier and toe penetration. Moreover, there is no contribution of contact stress to the pile group bearing (unless the total raft area would widely exceed the area of the piles), everything else equal, and whether or not the raft is on ground or in the air, or the pile heads are connected by a raft, a mat, or a grid of beams makes no difference to the average settlement of the piled foundation.

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