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Hansbo Lecture
The Unified Design of Piled Foundations

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ABSTRACT. The unified method for analysis of the interaction of forces and movements that governs the settlement of a foundation supported on a single pile or on a small group of piles is presented. It is shown that the response to loads applied to foundations supported on single piles and narrow pile groups differ from that of foundations supported on wide pile groups and that the settlement of the wide group can be analyzed using the simple model of an equivalent pier placed on an equivalent raft at the pile toe level. The effect of downdrag acting along the perimeter piles of a wide pile group is indicated and referenced to measurements. The conditions for presence of a contact stress below a pile cap and below a wide piled raft are discussed. The recommendations of the ISSMGE TC212 for design of piled raft foundation is quoted and the implication of the assumptions applied is highlighted.

1. SINGLE PILES AND SMALL PILE GROUPS

The unified method of design of piled foundation is based on designing foundations considering actual and acceptable settlements, as opposed to basing the design on a pile "capacity" reduced by various factors of safety or resistance factors. The unified method is a logical method because it considers actually occurring loads, deformations, and movements, whereas the conventional design means calculating forces for an ultimate condition that supposedly will never develop. The main approach to the unified method was proposed more than 30 years ago (Fellenius 1984; 1988). However, many have still difficultly in taking the step from the conventional "capacity-reasoning" to the more rational "deformation-reasoning" of the unified method. The following notes aim to explain the basics of the unified design.

Consider a hypothetical case of a single 300 mm diameter, round, concrete pile installed through 25 m of clay and 5 m into an underlying sand. Figure 1 shows typical load-movement curves determined from a hypothetical static loading test on the pile calculated using the UniPile software (Goudreault and Fellenius 2014). The input data are from Case 9 of the examples in the software manual, although slightly simplified. The test is assumed to have been carried out in equal load increments (125 kN) until large significant pile toe movements were recorded. The pile head load-movement curve shows the load (1,400 kN) that corresponds to the Offset Limit and the load (1,600 kN) that gave a 30-mm pile toe movement. (I have found that the two load levels are useful when comparing the response of different piles to load). Coincidentally, the 30-mm toe movement is

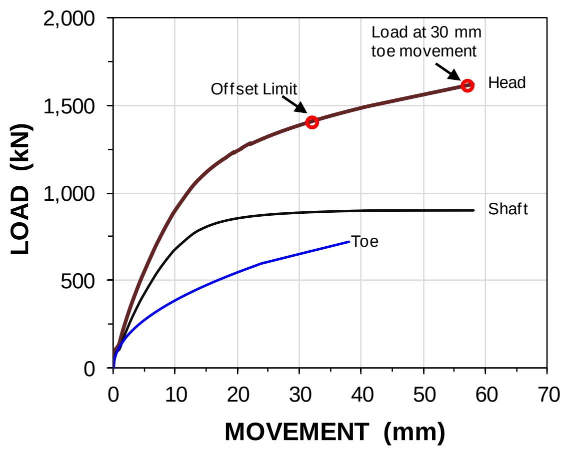

Figure 1. Typical results of a hypothetical static loading test on the hypothetical pile.

also 10 % of the pile toe diameter. The load applied to the pile head that resulted in a movement equal to 10-% of the toe diameter is frequently used as a definition of capacity. This definition originates in a misconception of a recommendation by Terzaghi (Likins et al. 2011).

The hypothetical pile is assumed to have been instrumented for measuring the distribution of load down the pile during the static loading test. The hypothetically measured distributions for the applied loads are shown in Figure 2, as calculated using UniPile. The assumed t-z functions for the shaft (clay and sand) and q-z function for the pile toe are indicated. The figure also shows the hypothetical distribution of settlement at the site assumed to be caused by a small lowering of the groundwater table or by a similar change of effective stress triggering a consolidation process. Notice that the soil below the pile toe level was assumed sufficiently dense or stiff not to develop any appreciable settlement due to the groundwater table lowering or to the increase of stress from the load transferred to the soil below the pile toe level.

The shaft resistance t-z curves represent the shear-movement response of the soil along the pile. Depending on piles and soil, the response in any given case will differ from that of another case. Responses may exhibit large and small movement before a peak shear resistance, before continuing in a strain-hardening, strain-softening, or plastic mode. Normally, the shear it is not associated with volume change, although, it is conceivable that, on occasions, the soil nearest the pile surface can contract or dilate due to the shear movement, with corresponding slight effect on the single-pile t-z curve.

At the pile toe, however, the downward movement of the pile (per q-z function), displaces the soil both below the pile toe level, to the side, and—to a limited height—also up along the side of the pile. The q-z function incorporates both effects and the q-z curve combines the effects of soil being displaced and the soil volume being changed due to the shear forces that develop around the pile toe, where both compression and dilation can occur.

The conventional approach is to determine a "safe" working load by applying some definition of "capacity" to the pile head load-movement curve. (The definitions actually used in the engineering practice for what constitutes "capacity" differ widely). The working load is then determined by dividing the "capacity" with a factor of safety larger than unity or, in LRFD, multiplying it with a resistance factor smaller than unity. Conventionally, it is assumed that the service-ability (settlement aspect) of the piled foundation is ensured by this approach.

When the long-term settlement of the soil surrounding the pile is small, the approach usually results in a piled foundation that does not experience adverse deformations for the applied working load. On the other hand, when the soil, as in the subject case, settles around the pile, a drag force and a downdrag will develop. Some codes and standards, e.g., the AASHTO Specs and the EuroCode, add the calculated drag force to the working load. When the drag force is correctly estimated (as opposed to underestimating it, which is a common mistake), this approach often results in that the pile, as originally designed, will seem to be unable to carry the desired working load and, therefore, the design is changed to employ larger,

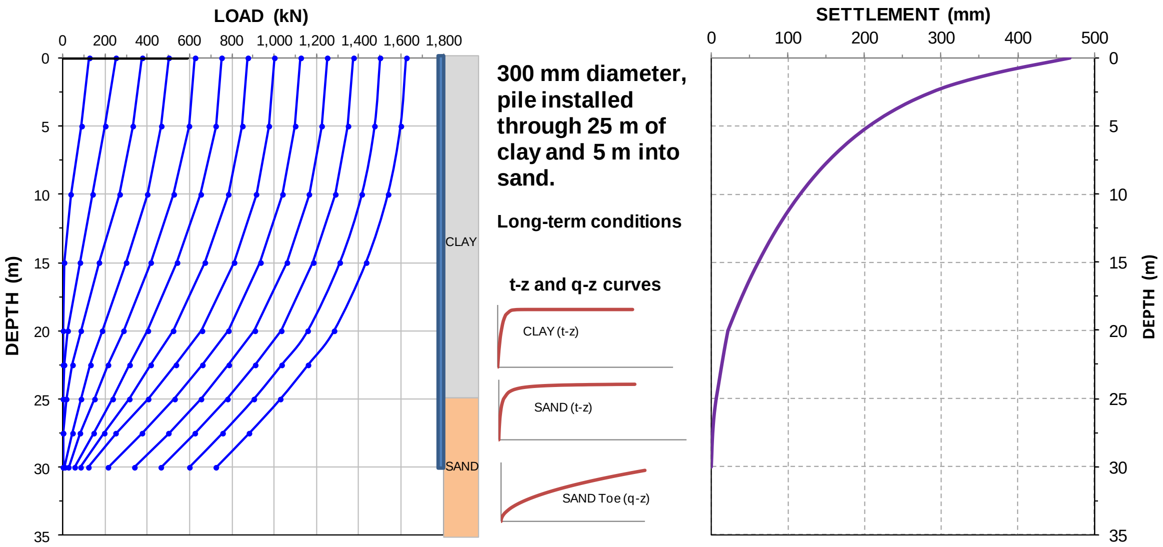

Figure 2. Distributions of axial load in the pile and settlement of the soil around the pile.

longer piles, and/or adding piles. More enlightened codes and standards, e.g., the Canadian Bridge Design Code, the Australian Building Code, US Corps of Engineers, etc., recognize that this approach is not just ignorant, but costly, and, that it yet does not ensure a safe foundation (Fellenius 2014a; 2016). The drag force is not the issue, the downdrag is, and the action of the settling soil has to be assessed in a settlement analysis.

The unified design considers the pile and soil deformations (settlement) and recognizes the fundamental reality that forces and movements are related and cannot be considered separately from each other. Thus, design of piled foundations according to the unified method involves matching the force and settlement interaction. A force equilibrium is determined as the location where the downward acting axial forces (dead load and drag force) are equal to the upward acting forces (positive shaft resistance below the equilibrium depth and toe resistance). The settlement equilibrium is determined as the location where the pile and the soil settle equally (the direction of shear forces along the pile changes from negative to positive at this location). When the shaft shear response is correctly identified, the two equilibriums occur at the same depth, called "neutral plane".

For the hypothetical case considered, as the supported structure is constructed, it will introduce a permanent working load (the dead or sustained load), say, 600 kN. The transient (live) load for the case is assumed to be about 100 kN. The loading test indicates that the load transfer movement due to the 600-kN load will be smaller than 10 mm. The purpose of the settlement analysis per the unified method is to determine the magnitude of the additional settlement that will develop in the long-term to follow the application of the load.

Figure 3 repeats Figure 2 and adds a curve to the load distribution diagram labeled "Increase of load due to negative skin friction", which mirrors the shaft resistance reduction of axial load with depth. The curve starts at the pile head at a load equal to the 600-kN permanent working load for the pile. Each intersection between the curve labeled "Increase of load due to negative skin friction" and the load distributions curves is a potential force-equilibrium neutral plane. A series of horizontal lines that intersect with the settlement curve has been added from each such intersection and each such intersection of these lines with the settlement distribution is a potential settlement-equilibrium neutral plane. At each potential settlement-equilibrium neutral plane, a slightly slanting line is drawn representing the pile shortening for the axial load in the pile. At the pile toe level, the distance between this line and the soil settlement at the pile toe level represents the pile toe penetration for the particular location of the settlement-equilibrium plane. Each intersection of the slanting lines with the line at the pile cap level indicates the settlement of the foundation. The task is to determine which of these would represent the long-term settlement of the supported foundation.

The figure shows several potential locations of force-equilibriums and settlement-equilibriums. However, there is only one location (depth) that is true, that is, only one location for which the pile toe force determines a location of the force-equilibrium that is at the same location (depth) as the settlement-equilibrium producing a pile toe penetration that, according to the pile toe load-movement curve, corresponds to the pile toe force in the load distribution diagram.

The true neutral plane location can be determined by trial-and-error as illustrated in Figure 4. A first-attempt toe force is assumed, and the load distribution from this force is extended upward to intersection with the drag force curve. A horizontal line is then drawn to intersect with the settlement distribution curve. If this intersection is the settlement-equilibrium depth, then, the pile line will determine the pile toe penetration. The corresponding pile toe resistance is determined by correlation with the pile toe load-movement curve. As shown in the figure, this first-attempt resistance does not match the originally assumed toe force, the starting toe resistance. A new starting toe resistance is therefore selected and the process is repeated. After two or three attempts, a match (red loop) is obtained as shown in Figure 5.

The purpose of matching the force-equilibrium and settlement-equilibrium to the pile toe movement and the pile toe force (never choose one without the other) is to determine—predict—the settlement of the single pile or small pile group. There is a misconception around that the movement measured for a specific applied load in a static loading test directly represents the settlement of a pile for the load. Note, however, that the static loading test does not measure settlement, but movement and, often, just the accumulated compression of the pile for the applied test load. Of course, knowing the movement response of a single pile for an applied load, in particular the

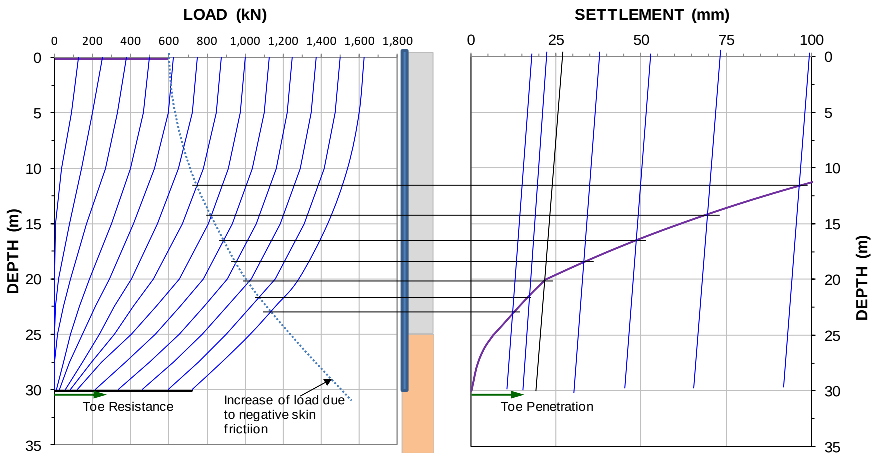

Figure 3. Figure 2 with added information: Increase of load due to negative skin friction building up drag force, intersections between this curve and load distributions suggesting potential force-equilibrium neutral planes, and construction of pile toe penetration from assuming that the settlement-equilibrium location is that the same depth as the depth of the potential force-equilibrium neutral plane.

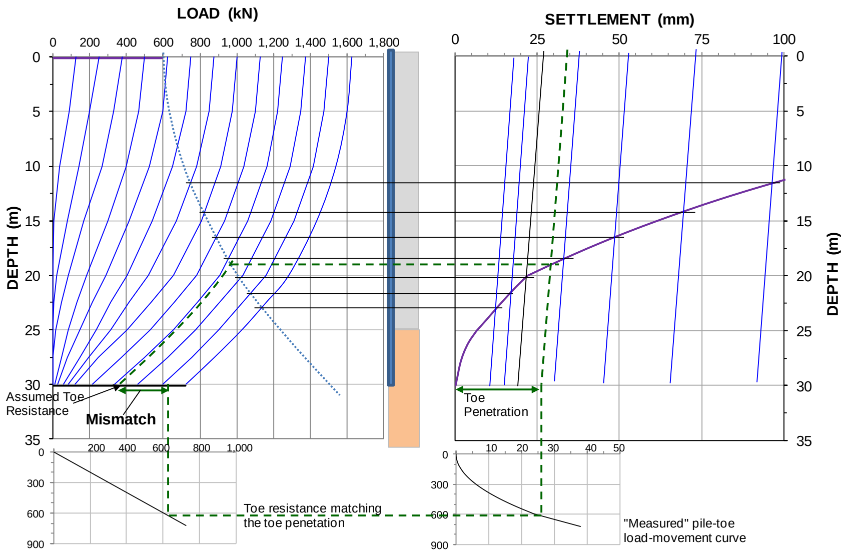

Figure 4. First attempt to find the true neutral plane.

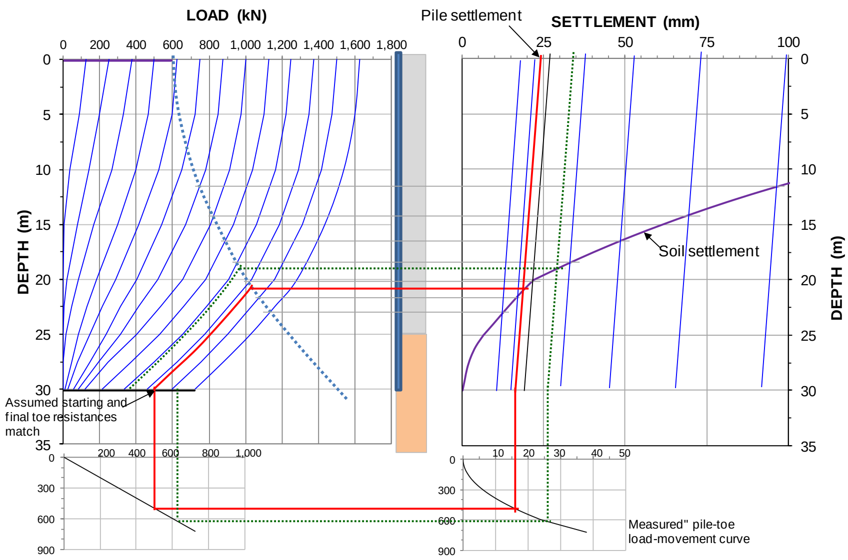

Figure 5. Final match between starting and finished toe resistance and determining the pile settlement.

pile-toe load-movement, is a vital part of determining the settlement for that pile, as illustrated in the forgoing.

The analysis shown in Figure 5 produced a calculated pile head long-term settlement of about 25 mm, which is satisfactory for most piled foundations. Depending on the height of the transition zone (transition from negative to positive shaft shear directions), the drag force will amount to about 350 through 400 kN. It will prestress the pile and it will essentially be a beneficial load. The maximum axial load at the neutral plane will be about 1,000 kN, which is well within the pile structural strength. When, as in this case, the loads from the supported structure also includes transient (live) loads (here 100 kN), these will just replace a similar magnitude of drag force. There will be a small pile shortening, but it is recovered when the live load is gone and does not add to the long-term settlement.

For a single pile, the portion of load reaching the pile toe influences only a small volume of soil, and its compression due to the increased stress is included in the pile-toe load-movement response, the q-z relation.

Piles making up a small group not wider than 3 or 4 rows of piles, will normally respond with a load distribution in the long-term similar to that of a single pile and develop a similar neutral plane location. However, the accumulated loads applied to a small (or narrow) group will add stress to the soil underneath the pile toe level that can result in settlement in addition to that due to other factors, local fills, other foundations, groundwater table lowering etc. The stress increase below the toe level is the effect of the toe resistance and the shaft resistance between the neutral plane and the pile toe level (for a narrow group), which will add stress to an area wider than the group footprint. The shaft resistance can be considered to originate from a string or assembly of pile elements between the neutral plane and the pile toe. Thus, the closer one such element is to the pile toe, the less wide the area affected by the shaft resistance of the element.

Fellenius (2016) suggested that settlement below the pile toe level due to the shaft resistance on a small group of piles functioning as single piles should be calculated over an equivalent raft that is wider and longer than the footprint by a width equal to 20 % of the distance between the neutral plane and the pile toe level. That is, the equivalent raft at the pile toe is widened by 1(H):5(V) from a raft at the neutral plane equal to the pile group footprint (envelop) area, as illustrated in Figure 6. The load stressing that widened toe raft is then equal to the total load applied to the pile group.

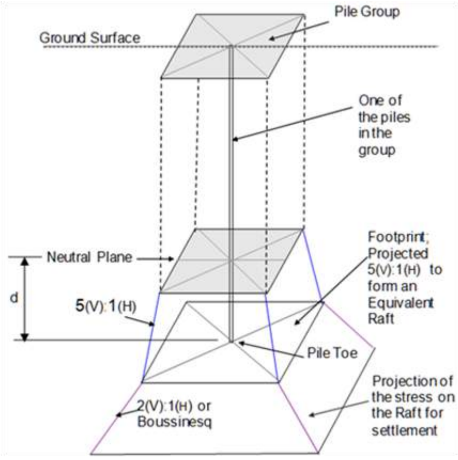

Figure 6. Distribution of stress below the neutral plane for a group of piles. Only one pile is shown.

1.1 Discussion

What factor of safety, F_s, does the 700-kN load represent for the single pile example? If the about 1,400-kN Offset Limit marked in Figure 1 would be taken as the pile capacity, then, the F_s would be 2.0. As mentioned, some like to define capacity as the applied load that gave a pile toe movement equal to 10 % of the pile diameter, which load is 1,600 kN here, and, thus, would indicate an F_s of 2.3. (A capacity based on a certain toe movement is rational. This notwithstanding that basing that movement on the pile diameter is most irrational; what movement and settlement the structure can accept is not a function of the pile diameter). Whether or not "the capacity" is defined as 1,400 kN or 1,600 kN, the long-term settlement will be the same, but, as no surprise, increasing the total load by 100 kN permanent load to 800 kN would increase the long-term settlement. How much is easily determined from the procedure illustrated in Figures 4 and 5; the new settlement-equilibrium neutral-plane would coincidentally be about where the dashed green lines intersect in the Figure 4 settlement diagram. That is, the calculated pile long-term settlement would increase to about 35 mm. Whether this larger settlement would be a satisfactory outcome of the design or not, the structure supported on the pile would be the party telling, the factor of safety has no say.

Indeed, the definition of capacity and choice of factor of safety has little to say about the long-term response of the foundation. The proper design question to pose is "what is the magnitude of the acceptable settlement?". Moreover, a margin against unacceptable settlement cannot be determined by repeating the analysis for an applied load increased by a single "serviceability" factor. The unified method might show for one case that the new settlement due to a so-increased load would still be acceptable for the structure supported, while in another case having some different conditions of soil, the increase would show to be beyond acceptable. In short, the assessment using a factor of any kind would require considering a combined effect of a load increase and changed location of the neutral plane. The important requirement for a design of a foundation supported on a single pile, or on a small group of piles, is to indicate the long-term pile head settlement associated with the working load—unfactored.

In many approaches to modeling the response of a single pile to load, the shaft and toe resistances are assumed to be "ideally" elastic-plastic, i.e., that both have an ultimate resistance for a specific movement (different for the shaft and toe) beyond which the resistance stays constant, i.e., the ultimate resistance is reached. However, the reality is that the shaft resistance response is rarely ideally plastic even at large movement, but can instead be both strain-hardening and strain softening. And, the pile toe resistance never reaches a plastic state, but is always strain-hardening. This means that the capacity determined from the shape of the pile-head load-movement curve (as established in a static loading test) is different from the one determined as the sum of the resistance of the individual pile shaft and toe elements. The fact that many theories use the latter and then verify the relevance of the results to the former does not instil much confidence in a design of a piled foundation based on a capacity reasoning.

The rational approach to the uncertainty of the foundation design lies primarily in the settlement analysis, notably the settlement distribution, which in effect is a prediction of the most probable future development. The settlement distribution, therefore, should be conservatively estimated with regard to the factors that increase the effective stress and to the compressibility parameters employed in the settlement analysis, as well as to other potential geotechnical and geological facts of the site. The pile load distribution, even if from measurements, needs to be carefully assessed, in particular, the pile toe load-movement response, as it is the most critical component of the load response. However, a settlement/deformation analysis is by far more reliable than the "capacity" approach. For example, most people would initially think that if the static loading test is performed before full set-up has occurred, the additional shaft resistance—increase of capacity—developed over time would assure a hidden extra "safety". However, if in the illustrated single pile case, set-up would increase the long-term shaft resistance in the clay beyond that measured in the "test", the effect would be a lifting of the neutral plane and, potentially, a larger long-term settlement of the piled foundation. That is, a design based on the "capacity" approach might then result in a less serviceable foundation.

2. PILE GROUPS

2.1 General

When designing a foundation as a conventional piled foundation group of piles, the common approach is to perform the usual bearing capacity/factor-of-safety calculations considering the group response to be somewhat smaller than the response of the same number of single piles, which then is accounted for by applying a so-called "efficiency coefficient" smaller than unity. However, this disregards the fact that, while a single pile imparts but little stress to the soil below the pile toe, apart from the rather small volume nearest the toe, a pile group will distribute load and stress to a much larger volume, determined by the width of the pile group.

The surface area of the perimeter (the envelop) of a uniform pile group is much larger than the surface area of an equal number of single piles. For example, a group of piles at a 3-diameter spacing, has an envelope surface area about ten times larger than the sum of the surface area of all the piles in the group.

The bending stiffness of a foundation raft controls the forces in a raft resulting from differential settlement. According to O'Brian et al. (2012), the raft stiffness, K, is linearly proportional to the ratio between the E modulus of the raft and that of the soil, E_r and E_s, respectively, times the ratio (K) between the raft thickness (t) and width, (B), raised to the power of 3: $(K = t/B)^3$. The authors indicate a raft stiffness ratio of four orders of magnitude when going from an infinitely flexible through an infinitely rigid raft. Considering the large difference in E-modulus between concrete and soil, the E -modulus ratio,

(E_r/E_s), is about 10^3 to 10^4, and the fact that the stiffness ratio, K, is usually smaller than 1, it is clear that, typically, the usual raft stiffness is on the flexible side. The question whether or not a portion of a raft is on the rigid side depends mainly on the presence or not of a shear wall on the raft placed perpendicularly to the bending axis considered.

If the settlement in the surrounding soils is small, the long-term effect has no other consequence than some inconsequential drag force developing in the perimeter piles. On the other hand, if the settlement in the surrounding soils is appreciable, a significant downdrag will develop for the perimeter piles unless they are installed so that the pile toes are in very competent soil capable of building up toe resistance at small movement. If, on the other hand, significant downdrag would affect the perimeter piles, some downdrag could even occur for the next row or column of piles. It may then be advisable to have the perimeter piles longer than the interior piles in order to minimize the otherwise larger downward movement of the perimeter piles. Either way, the soil conditions will trend to minimize the differences so the foundation raft responds more like a flexible raft.

For a large (wide) piled foundation, the resistance along the perimeter can be disregarded. Thus, all the applied load will be transferred to the pile toe level. However, in contrast to the small pile group or a group of widely spaced piles, there will not be a separate action of the pile toes being pushed into the soil, but the entire body ("equivalent pier") of piles and soil will act as a unit and the load will be distributed between the piles and the soil, notwithstanding that there is shaft resistance interaction between the piles.

As is the case for the foundation on a single pile, for a foundation on a group of piles, the key design task is to determine the future settlement of the foundation. However, the result of a static loading test on a single pile has little direct bearing on the design of the settlement for a foundation on a pile group. Therefore, the frequently applied way to calculate settlement as equal to the accumulated movements from the equivalent load-movement response of each pile in the group, as illustrated in Figure 7, is a fallacious approach that has led to many more or less complex methods for calculating settlement of a piled foundation, none correct.

If a load applied to the pile cap results in that each individual pile in the group mobilizes only a portion of its available total pile resistance, and

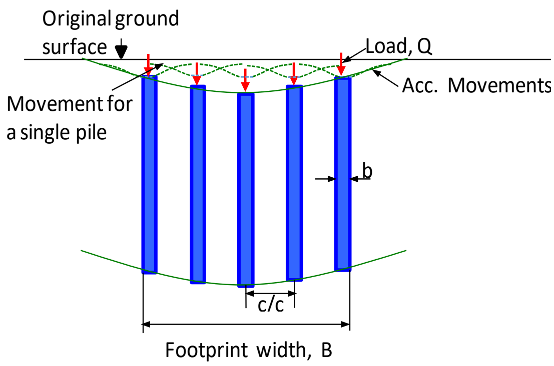

Figure 7. The misconception of calculating pile group settlement treating each pile as a single pile and accumulating the movements from an equivalent static loading tests on each pile

minimal contact stress under the pile cap. The piles will respond by a shaft resistance distribution and some toe resistance. Such piled foundation is then labeled a "pure" piled foundation, as opposed to a "piled raft", which is when there is a contact between the surface of the soil and the underside of the pile cap. Some call the latter foundations "hybrid foundations", "pile-enhanced rafts", "raft enhanced-piles", or "combined pile-raft foundation". While there is little consensus of what term to use, the variety of terms imply a consensus on expectations of a difference between a pile raft with a contact stress as opposed to a "pure" pile foundation. However, be the foundation a "pure" foundation or a "piled raft" foundation, the actual distributions of axial load down the piles in the group are more complex than for a single pile. Moreover, for "pure" pile group foundation on long piles, the pile compression may result in a downward movement of the pile head large enough for a contact to develop, but contact stress?

In the long-term, the soil in between the piles will undergo volume changes, mostly reduce and, consequently, move downward. The movement causes negative skin friction accumulating to a drag force in the piles. The maximum drag force that can act on the interior piles in a group is the weight of the soil in-between the piles. In contrast, the perimeter piles can experience more or less fully mobilized drag force, and, more important, downdrag. The drag force is in most cases inconsequential, but the downdrag can result in uneven stress distribution on the piles and undesirable bending stress in the slab.

Papers discussing pile group response to load characterize piled foundations as small or large, narrow or wide ("narrow" and "wide" refer to width, which is always shorter than the length). A narrow or small pile group can be defined as a group being no wider than 3 or 4 rows of piles and it can be assumed to respond to load much in the same way as an equal number of single piles. The response of groups composed of large number of rows and columns, will be significantly different, however, depending on the actual number of rows and columns, but more important, on the spacing between the piles, because the piles will interact via the shaft resistance, which effect will be enhanced by deeper pile embedment.

Publications sometimes label a piled foundation as small or large according to a definition proposed by Randolph and Clancy (1993), who separated small pile groups from large by a pile-group Aspect-Ratio, R, defined as $R = \sqrt{(ns/D)}$, were "n" is the number of piles in the group, "s" is center-to-center distance between the piles, and "D" is the pile embedment length. A small group has $R < 3$ and a large group has $R > 3$. A group of 12 piles with a 300 mm diameter to 10 m depth placed at a center-to-center distance of 3 pile diameters would have $R = 1.0$ and be a small group. I am reluctant, however, to accept that the pile groups comprising 38 piles to 40 m depth with Aspect Ratio of 1.9 and the 144 piles to 48 m depth with Aspect Ratios of 1.2, reported by Mandolini et al. (2005) and Okabe (1977), respectively, as detailed below, would be defined as "small".

A "pure" foundation alternative would normally comprise longer piles than a "piled raft" foundation. Often a comparison—per settlements or costs—between the two types is based on the "pure" piled foundation being designed conventionally per bearing capacity calculations and factors of safety, whereas the "piled raft" is designed by different analytical methods emphasizing settlement and relying on support from contact stress. First, this is not an apple-to-apple comparison. Second, the "bearing" effect of contact stress, if at all significant, would reasonably be diminishing with depth and be negligible for all but for groups comprising rather short piles.

Pile in a small (narrow) group will respond to an applied load much in a way similar to that of single piles, but for the effect of the accumulation of stress imposed to the soil below the pile toe level. If such a group would be tested with a common cap resting on the ground, of course, a significant contact stress could develop as the test progressed. The pile cap is a resistance element that functions much like a pile toe or a helical plate in a screwpile. UniPile models the cap or element by input of a q-z relation with its individual stiffness response. The software can simulate—match—an actual loading test from start to finish. However, for long-term conditions of a narrow piled foundation designed with a conventional factor of safety larger than unity, the soil will settle away from the pile cap, and there will be no contact stress contribution to the pile response.

One piled foundation variation is to have the piles not connected to the foundation slab, but functioning as soil reinforcing units. The terms used for this type of foundation are "piled pad" or "disconnected footing", and others. For case history examples, see Pecker (2004) and Amini et al. (2008). Such designs will have to consider the potential for horizontal displacement in the upper soils layer; the flaring out to the pile, which then will cause the foundation to settle.

I have found the concept of Footprint Ratio useful, i.e., the ratio between the total area of all piles over the footprint area of the pile group defined by the envelop around the piles. A spacing of 3 pile diameters in a symmetrical placement with equal spacing for all piles in a wide foundation (triangular configuration) corresponds to a 10-% Footprint Ratio. If, instead, the pile configuration as well as the piles are square and the spacing refers to the pile face-to face diameter, the Footprint Ratio is 11 %, which is about the same value. A 2-diameter spacing results in Footprint Ratios of 22 and 25 %, respectively. (On an aside, I have found that Footprint Ratios for wide pile groups larger than 15 % frequently will result in construction difficulties unless the piles are very short). Usually, piled foundations are designed with the center-to-center spacing between the piles in a group equal to 3 pile diameters or larger. As the spacing increases, the difference in response between interior and perimeter piles will diminish. Perhaps, for wide groups with a 2 %, or smaller Footprint Ratio, i.e., a spacing beyond about 7 pile diameters, the pile response will be similar to that of single piles, somewhat depending on the length of the piles. For example, the Footprint Ratios for the below mentioned cases reported by Mandolini et al. (2005) and Okabe (1977) were 9 % and 22 %, respectively.

The Footprint Ratio approach is best suited for foundations that have at least an 8-row width. Consider, for example, a square foundation on four piles, also square (side = b) placed at a spacing equal to 3b. The side of the area enveloping the piles is 3b + b and the total area is $16b^2$ (not the $9b^2$-area of an infinitely wide group). Thus, the Footprint Ratio is $nb^2/16b^2 = n/16 = = 25$ %. If instead the number of piles would be 9, 16, 25, 36, 49, 64, 100, and 400, the Footprint Ratios would become 18 %, 16 %, 15 %, 14 %, 13.5 %, 13.2 %, 12.7 %, and 11.9 %. A square pile group containing an infinite number of square piles placed at a three face-to-face diameter spacing would have a Footprint Ratio of 11.1 %. If instead the piles are circular, all the mentioned ratios would reduce by $\pi/4$ and the Footprint Ratio of the infinitely large group would reduce from 11% to 9%. Obviously, the Footprint Ratio concept is not suitable for narrow groups. Although an exact number can be obtained for each specific case, it is best considered as an approximate reference number to aid designer judgment in the design of wide piled foundations.

Depending on pile embedment length in a wide piled foundation, even with the piles spaced out across the foundation footprint, at Footprint Ratios larger than about 4 %, i.e., a pile spacing smaller than about 5 diameters, a piled foundation will tend to act as a pier or block, be an equivalent pier, composed of piles and soil responding together—acting as a unit. The "equivalent pier" has a stiffness, EA, determined by a modulus, E_{pier}, proportioned between the soil modulus and the pile modulus, as indicated in Equations 1 and 2. The pier compression will be entirely governed by the so proportioned stiffness (pile-soil combination above the pile toe level) and, thus, the pile and the soil will work in unison.

The combined E-modulus of the pile and soil body, the equivalent pier, is expressed in Equation 1. The compression contribution to the foundation settlement is then expressed in Equation 2. Because the E modulus of the pile material is either 200 GPa (steel piles) or about 30 GPa (concrete piles) and that of the soil is rarely more than about 50 MPa and frequently much smaller, the combined modulus depends mainly on the pile modulus and the Footprint Ratio. Thus, the soil modulus has negligible influence on the combined modulus.

$$E_{pile+soil} = FR \times E_{pile} + (1-FR)E_{soil} \approx FR \times E_{pile} \qquad (1)$$

where $E_{pile+soil}$ = combined E-modulus
FR = Footprint Ratio
E_{pile} = E-modulus of the pile
E_{soil} = E-modulus of the soil

$$\Delta L = (Q \times L)/(E_{pile+soil} \times A_{total\ footprint}) \qquad (2)$$

where ΔL = compression of the equivalent pier
Q = load applied to the foundation raft
L = height of equivalent pier
$A_{total\ footprint}$ = footprint area of the raft
$E_{pile+soil}$ = combined E-modulus

The distribution of load at the pile cap level depends on the combination of the bending stiffness of the pile cap (the raft or slab) and the response of the soil. Usually, the pile cap on a small pile group can be assumed to be essentially rigid and, therefore, all pile heads will deform in equal measure. N.B., because the pile response to movement can differ from pile to pile, depending on the pile length and resistance distribution, the equal deformation does not necessarily mean that the loads at the pile cap are equal. Wide pile groups, pile caps, rather, with small thickness to width ratio, will be flexible and the pile head deformation will vary between the piles. The load may still vary from pile to pile. However, inasmuch that the most of the deformation is due to 'elastic' axial shortening of the piles, the differential settlement of the foundation will be moderate. The major part of the differential settlement will be due to conditions below the pile toe level and settlement of the soil surrounding the pile group which may impose additional settlement in the perimeter piles due to downdrag.

In common practice, settlement analysis would be according to the 1948 Terzaghi-Peck approach assuming an equivalent raft placed at the lower-third point and disregarding the stiffening effect of the pile length below that raft. However, as shown in case records reporting settlement of wide pile groups (Fellenius and Ochoa 2016, Fellenius 2016), the pile group settlement of any type of piled foundation is better modeled by placing the equivalent raft at the pile toe level.

A foundation raft, whether it is a raft with no piles located at the ground surface or an equivalent raft at the pile toe level, will transfer the applied load as stress to the ground immediately below the raft. If the raft is infinitely flexible, the stress is more or less the same across the raft area. If, on the other hand, the raft is infinitely rigid, the stress will vary across the raft and inasmuch the soil response is elastic, the perimeter stress will be the largest. The stress distribution at depths below the raft can be calculated according to general principles of immediate compression, consolidation settlement, and secondary compression. Boussinesq equations, and sometimes Westergaard, equations, are common tools for determining the stress distribution below the raft and to its sides. For settlement calculation under rigid raft, the calculated stress underneath the characteristic point is used as this is where it is equal for flexible and rigid rafts (Fellenius 2016).

The last two decades have seen several numerical methods developed, incorporating interaction between the pile cap (raft), the piles, and the soil. For example, the ISSMGE Committee TC212 has produced guidelines for the design of a piled-raft foundation as a "combined pile-raft foundation (CPRF)" as foundation elements with the interaction between these and the soil (Katzenbach and Choudhury 2013). The publication includes several references on the subject.

The principles of the design according to the TC212 guidelines are illustrated in the following three figures. Figure 8 shows the forces acting on the piled raft from the supported structure and the response on the raft from the piles and the contact stress underneath the raft.

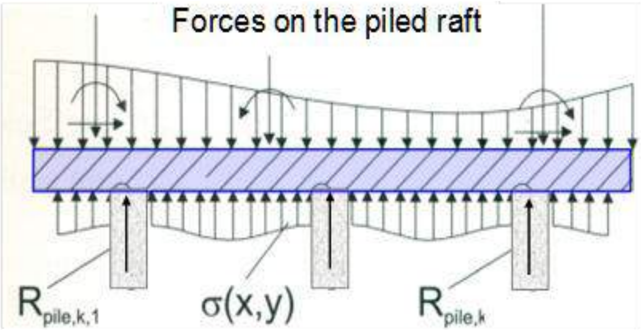

Figure 8. Piled raft affected by supported load and pile response and contact stress (after Katzenbach and Choudhury 2013).

Figures 9A and 9B show the pile and soil response model, which assumes that each pile is subjected to shaft and toe resistances. The figure suggests that the shaft resistance increases with depth as were it governed by an overburden effective stress relation. Additional resistance to support the foundation is assumed to be derived from the contact stress over the foundation area between the piles. This will result in a pile load distribution that depends on the distance between the pile (pile spacing, c/c), as well as on the location of the pile within the pile group (e.g., center, side, or corner) and degree of rigidity/ flexibility of the raft. The end result is the calculated settlement of the foundation. Note that the displacement due to stress increase below the pile toe level is not included in the DR; it needs to be determined separately.

To use in estimating settlement (displacement), Katzenbach and Choudhury (2013) defined two ratios: a Resistance Ratio (RR) and a Displacement Ratio (DR).

The RR is the ratio between the sum of all pile shaft and toe ultimate resistances over the total piled raft resistance (the same pile resistances plus the raft ultimate contact resistance). For a "pure piled foundation" (no contact stress), the RR is 1.0 and for a raft without piles (a "regular spread footing" with the same footprint and same load), it is 0.0. Anything in between is a "piled raft".

The DR is the ratio between the displacement (settlement) of a "pure" piled foundation with no contact stress to that of a raft with the same footprint and same load and with a presumed contact stress carrying part of the load. When the DR is close to 1.0, the contact stress would be very small. If almost all the load would carried by contact stress—a highly unlikely case—the DR would be close to 0.

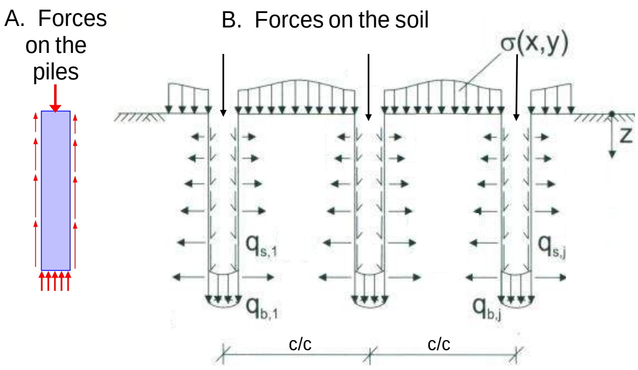

Figure 9. Shaft and toe resistance acting on a pile (A) and forces acting against the soil (B) (after Katzenbach and Choudhury 2013 with labeling modifications).

Figure 10 shows the relation between RR and DR for varying degree of piles "enhancing" the foundation., the RR would then be close to 0.0. The RR and DR for a raft incorporating piles within the raft footprint, compared to the same raft placed above the ground (i.e., with a space between the raft bottom and the ground surface; say, to avoid affecting a permafrost layer) would both be somewhere between 0.0 and 1.0. That is, lie within the band shown in the figure.

It is obvious that a raft without piles has contact stress. A raft that incorporates only a few piles ("few" here means large spacing and a small Footprint Ratio) is labelled an "enhanced raft" foundation, and a contact is assumed to exist between the raft and the soil (provided the factor of safety would be close to unity or less). It will have a RR close to zero and a DR close to unity. If the

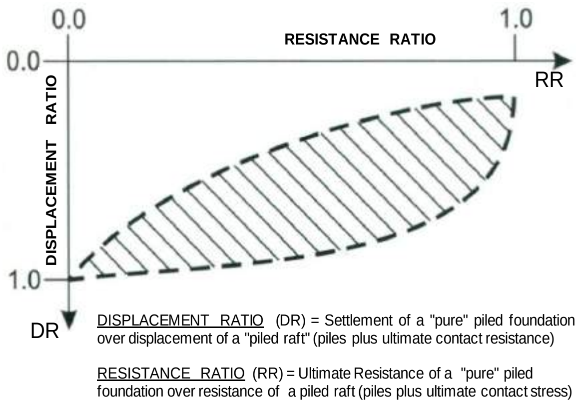

Figure 10. Relation between the two ratios (after Katzenbach and Choudhury 2013).

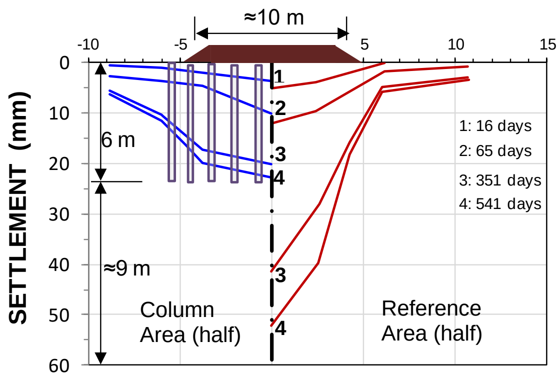

Figure 11. Settlement of embankments on soft compressible clay (data from Broms 1976).

number of piles is increased (pile spacing reduces and Footprint Ratio increases), the RR would increase and the DR would decrease. If the piles would be able to carry the load on their own, albeit with large movement, the RR would reduce faster than the DR. However, were the pile "enhancement" actually necessary in order to reduce settlement, the DR would have to increase rapidly. This means that the more probably development of the interaction between the RR and the DR in an actual case would follow the lower envelope curve in the figure.

Additional discussion of the DR for various combinations of rafts and piles is available in Phung Duc Long (1993; 2010).

2.2 Case Histories

Methods of analysis need to be verified by correlation to results of full-scale tests. The following presents a few case histories reporting measurements of pile group settlement. The case histories are often referenced in support of various methods.

Broms (1976) reported settlement measured for two square embankments on a 15 m thick deposit of compressible soft clay. One of the two embankments had a grid of 500-mm diameter, 6 m deep lime-columns placed at a center-to-center spacing of 1.4 m (2.8-column-diameter) and an about 10 % Footprint Ratio. Figure 11 combines the measurements from the two embankments taken at 16, 65, 351, and 541 days after placing the embankment.

"Column Area" indicates records under the embankment supported on 6 m long lime-columns and "Reference Area" indicates an embankment with no columns.

Most of the settlement occurred below 6 m depth and the settlement within the column depth was not only limited, but also more uniform than under the no-column, reference area.

Although the axial stiffness of the lime column is many times smaller than that of a similar size concrete pile, it is still many times larger than that of the soft clay. It is, therefore, rational to draw a parallel between the embankment supported on the lime columns and an embankment, or flexible raft, supported on piles. A suitable model for analyzing the two embankments is to assume for the Column Area an equivalent pier with a flexible equivalent raft at the column depth using Boussinesq stress distribution to calculate the settlement at the center of the area and outward from the center. The calculated settlement is the sum of the consolidation settlement for the equivalent raft and the compression of the lime-column reinforced upper 6 m of clay. The settlement of the reference area is calculated as a flexible raft placed at the ground surface.

Matching calculated settlements to the settlement measured at the center of the embankment is simple. The same parameters for the soil below 6 m depth were used under the column and reference areas. Then, on shifting the calculation location to the other measuring points within and outside of the footprint without making any other change than that imposed automatically by the Boussinesq distribution, the calculation results using the UniSettle software (Goudreault and Fellenius 2011) were found to match also those observations.

The settlements measured outside the column area have been stated to indicate that the column-reinforced pier or block transferred load through shaft resistance acting along its enveloping perimeter that imposed consolidation in the surrounding clay. However, as the calculations using the UniSettle software show, the settlements outside the two footprint areas are caused by the stress from the embankment load acting at the toe of the lime columns or at the ground surface of the reference embankment.

Okabe (1977) reported results from a series of investigations undertaken to study the effect of drag force on driven foundation piles installed at a low-lying wet paddy field through a compressible sandy silt undergoing regional settlement. The unfactored working load was 800 kN. The soils consisted of soft compressible sandy silt to about 40+ m depth and the area was expected to settle due to fill being placed across the site. The study involved a "pure" piled foundation supporting a 30-MN bridge pier supported on 38 piles. The piles were 700-mm diameter, 40 m long, closed-toe steel pipe piles, joined by a common cap. The piles were placed in the corners of equilateral triangles with a 1.5 m, i.e., 2.1 pile diameters center-to-center spacing (Footprint Ratio = 20 %). The layout is shown in Figure 12 indicating the location of four test piles for which axial strain was monitored and evaluated to axial pile load at four depths over 1,040 days. Three of the test piles were interior piles and one was a perimeter pile. A fifth test pile, a single 600-mm diameter, closed-toe steel pipe pile was driven away from the group and to 43 m depth into dense sand to serve as a reference pile. It also was instrumented.

Figure 13 shows the load distributions in test piles. The distributions in the three interior piles were quite similar to each other, but differed considerable from the perimeter and reference piles. The paper reporting the study did not include any measurements taken before the casting of the foundation slab. It is probable that some axial residual force developed in the piles from the driving and from the soil reconsolidation. This would explain why the measurements did not indicate any increase of load with depth, i.e., no shaft resistance. The tendency of the distributions to reduce below about 25 m depth is commensurate with presence of locked-in (residual) force load.

The dashed straight line represents the per pile soil weight with depth. Note, however, that, while the perimeter pile was fully affected by the settling soil and showed the same "negative-skin-friction" development as the single pile, the interior piles did not show a build-up of drag force; the main message, here.

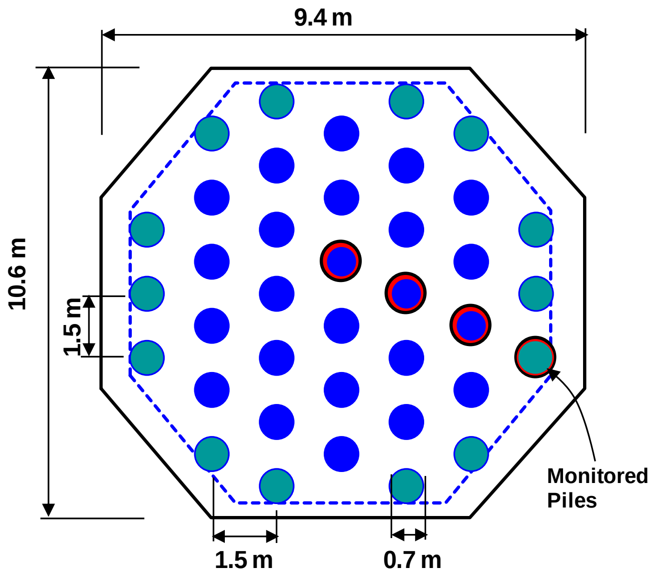

Figure 12. Layout of piles for pile-group study (data from Okabe 1977).

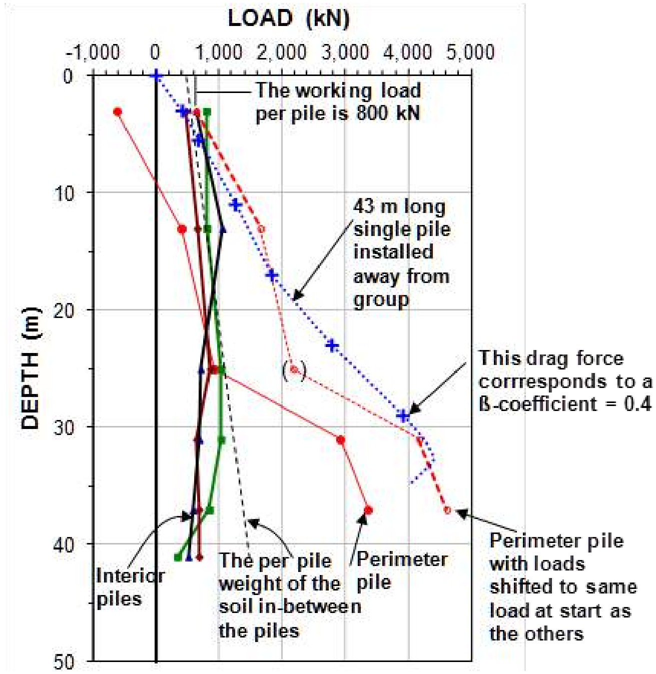

Figure 13. Load distribution in the three interior and perimeter piles and the reference single pile (data from Okabe 1977).

Generally, other than for a small piled foundation, when load is applied to the pile cap, the piles are not pushed individually through the soil, but they and the in-between soil start to move together—move as a unit. Because the perimeter piles have to face the outside soil, their response is different to the interior piles as illustrated in the quoted study.

The study was directed toward the drag force, which in the state-of-the-practice of the times, was considered the key factor. Unfortunately, the settlements of the pile and the soil were considered less important and were not measured (reported).

O'Neill et al. (1982) compared the load-movement response of a single pile to that of a nine-pile group. The piles were 273 mm diameter closed-toe steel pipe piles with 9 mm wall driven to 13 m embedment into a thick deposit of over-consolidated stiff salty clay. The pile group configuration was a square grid with 1.64 m side measured center-to-center of the piles. The pile cap was above the ground surface. The pile group is small by any definition. For groups as small as this, indicating spacing in terms of pile diameter and Footprint Ratio is not meaningful.

Figure 14 shows the layout of the group. Static loading tests were first performed on the single pile and on the nine-pile group. Thereafter, the four corner piles and the center pile were tested together, with the mid-side pile not loaded. Then, the four mid-side piles were tested together. The purpose of the tests was to study group effect, i.e., load-movement of a single pile v. a group of piles.

Figure 15 shows the average pile-head load-movement response of the single pile and the test on the nine-pile group. Compared to the response of the single pile, the group responses are much softer. The results have been used to correlate a group "efficiency" factor and in models incorporating the pile spacing and number of piles, or by some more sophisticated analysis incorporating the same along with aspects of interaction between the piles, soil module, and shear zones. Either approach was then rationalized by the larger movement for the same load—softer response—of the nine-pile group as opposed to that of the single pile.

As the test on the corner piles plus the center pile and on the mid-side piles are re-loading tests, comparing their results to that of the single pile are not really an apple-to-apple comparison. However, comparing the results of the four-, five-, and nine-pile groups is.

Using the UniPile software, I fitted the load-movement of the single pile with input of t-z functions for the shaft resistance and q-z function for the toe resistance forcing a fit to the measured curve, incorporating the measured residual force in the pile. The fitted curve is added to the figure. I then calculated the settlement for an equivalent raft placed at the pile toe level and determined a reloading modulus number of the clay below the pile toe that fitted (gave) the measured 2 mm settlement difference between the single pile and the nine-pile group for the 700 kN maximum test load. Next, without changing any other input than

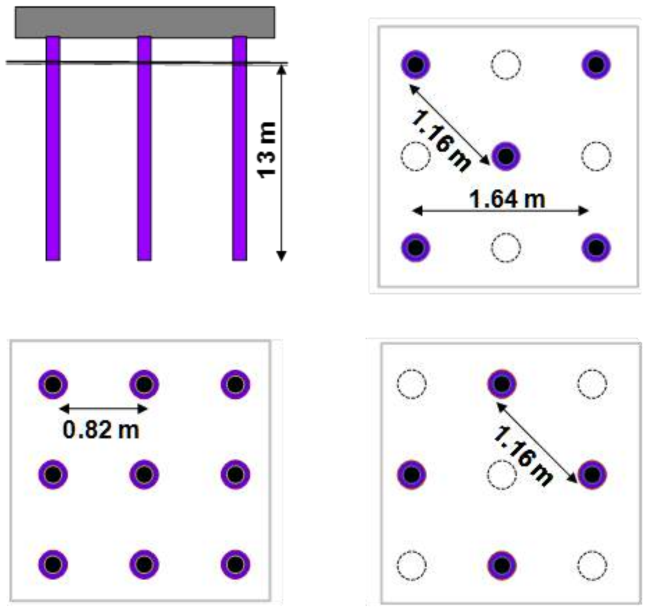

Figure 14. Layout of the 9-pile group and the configuration of the tests on the 9, 5, and 4 piles (data from O'Neill et al. 1982).

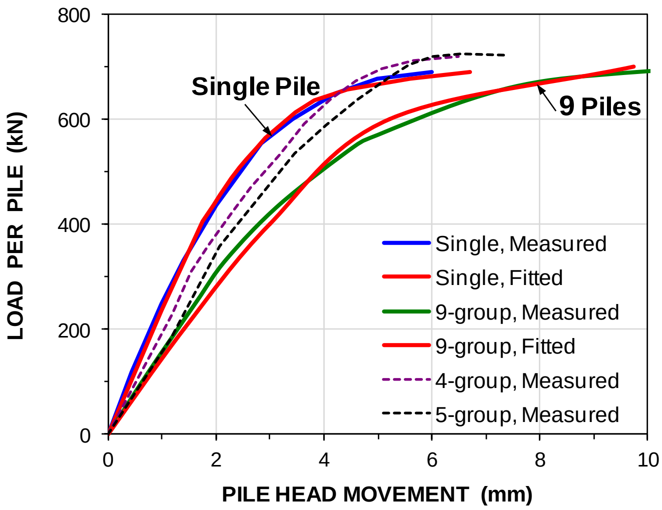

Figure 15. Pile-head load-movement curves for the single pile, the nine-pile group, and for the reloading of five and four piles in the group (data from O'Neill et al. 1982).

load, I calculated the equivalent raft settlement for the other tests. The so-calculated pile-head load-movement, also shown in the figure, indicates agreement with the measured load-movement curve.

The good fit to the nine-pile group curve does not prove that the equivalent pier plus equivalent raft is the correct method of analysis. However, it does support that a reason for the softer response of the group as opposed to that of the single pile, could well have been that the soil volume below the piles was affected by the applied load and not due to interaction between the piles in the group.

Badellas et al. (1988) and **Savvaidis (2003)** presented a case history of settlement measurements for a 38 m diameter, liquid storage tank in Greece supported on a piled foundation. The soil profile consisted of 40 m of soft compressible soil followed by dense coarse-grained soil. The groundwater table was at about 1.5 m depth. The tank bottom consisted of an 800 mm thick concrete raft and the total dead weight of the empty tank was 70 MN (about 60 kPa stress). The foundation was designed as a "pure" piled foundation and comprised a total of 112, 1,000 mm diameter, 42 m long bored piles. The Footprint Ratio was about 8 % and the average spacing was about 3.6 pile diameters. No results of any static loading test was reported.

Figure 16 shows the results of measured and calculated settlements during a hydrotest (just before unloading). The settlement calculations were performed using UniSettle and assumed an equivalent pier placed on a flexible raft at the pile toe level. The software input was adjusted until the output matched the settlement measured at mid-point including shortening of the piles due to the applied hydrotest load. The critical input was the compressibility (E-modulus) of the soil below the pile toe level. The match determined the representative modulus and the software was then used to calculate the settlement also along a diameter of the tank without changing of any parameters so that the only change was from imposing a Boussinesq stress distribution. The agreement between the calculated and actually measured settlements indicated that the assumption of flexible raft (Boussinesq stress distribution) fitted the records well.

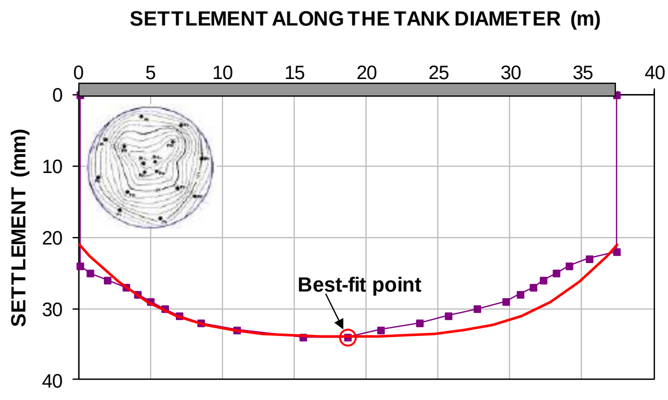

Figure 16. Measured and calculated settlements for the hydro tested tank (Fellenius and Ochoa 2016 with settlement data from Badellas et al. 1988, Savvaidis 2003).

The analysis results of the tank records are from the equivalent pier and equivalent raft analyses reported by Fellenius and Ochoa (2016), who also back-calculated the records from two additional case histories of wide piled foundations and showed that the distribution of settlement across a piled foundation diameter could be fitted to an equivalent pier and raft analysis.

Briaud et al. (1989) performed static loading tests on a group of five closed-toe, 273-mm diameter, pipe piles driven to a 9-m embedment and a single 9 m long pile serving as a reference pile. The piles were strain-gage instrumented. Figure 17 shows the pile layout in plan and profile.

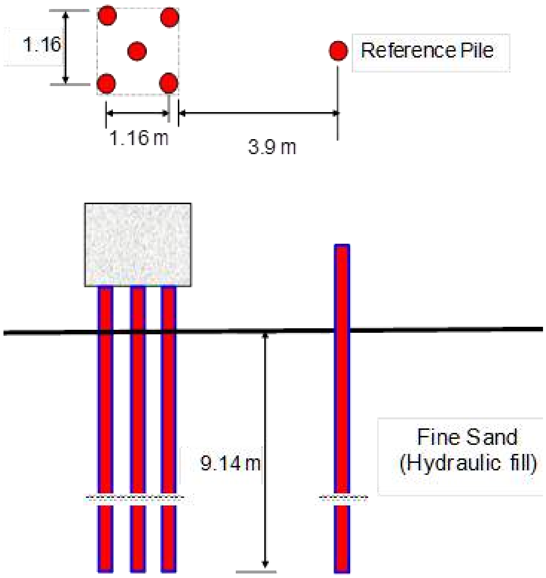

Figure 17. Pile layout in plan and a profile (data from Briaud et al. 1989).

The results show that the driving of the five group piles resulted in a compaction of the hydraulic sand fill, densifying the sand around the piles and loosening it under the pile toe. Moreover, the authors also report that the driving left the piles with appreciable residual force. The reported load distributions are therefore "true", as they refer to the axial load in reference to the conditions before the driving, as opposed to the common situation where the load distributions reported are only those loads imposed during the loading test.

Figure 18 shows the shaft and toe resistances of pile-head load-movement measurements in two corner piles, the center pile, and the single pile, the "reference pile". It is very noticeable that the group piles responded in a like manner, whereas the response of the single pile showed a larger shaft resistance and a smaller toe resistance than the individual group piles.

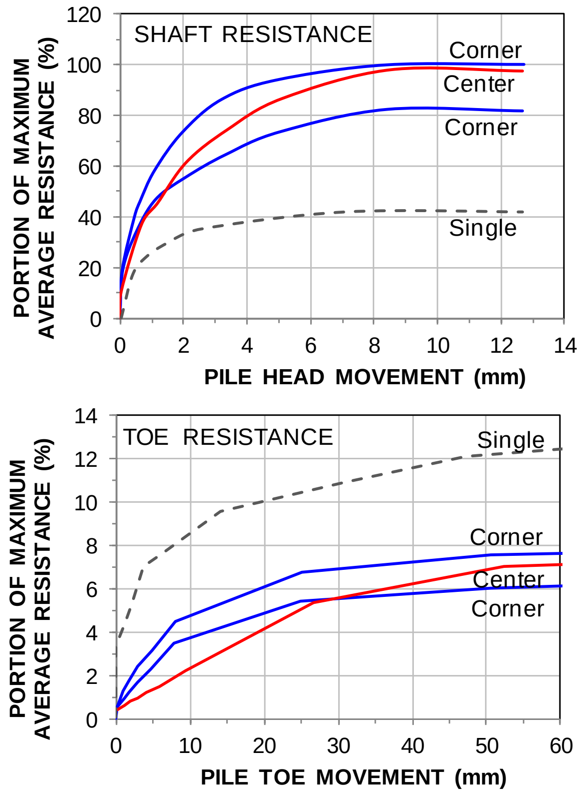

Figure 18. Results of the static loading tests separated on shaft resistance and toe resistance (data from Briaud et al., 1989).

Figure 19 shows the portion of the total applied load for each of the five piles in the group. The curves are so similar that the conclusion must be that each of the five responded as a single pile. Definitely, the response of the "reference pile" was different from that of the group piles, but this was due to the compaction effects and not to any single pile response versus group response to load. The case history is justifiably well-recognized and is frequently referenced. However, I think that those using the records of the test to verify methods for prediction of single pile settlement versus pile group settlement have stretched the rather limited results a mite.

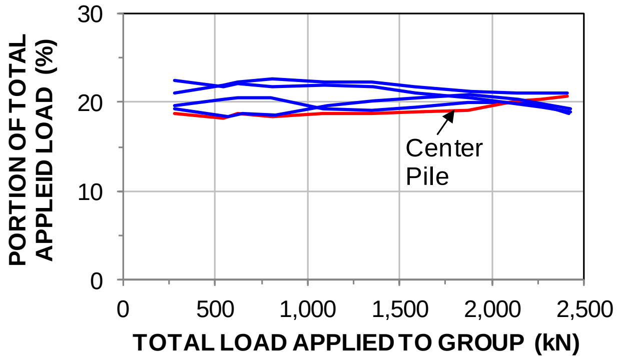

Figure 19. Distribution of load between the five piles in the group (data from Briaud et al. 1989)

Russo and Viggiani (1995) and **Mandolini et al. (2005)** reported a case history of a "pure" piled foundation of the main pier of the cable-stayed bridge over the Garigliano River in Southern Italy; constructed in 1991-94 and founded in deep compressible silty clay. The piled foundation comprised 144 mandrel-driven, then concrete-filled, steel pipe piles, 48 m long, 406-mm diameter, uniformly distributed in a 10.6 m by 19.0 m raft (Russo and Viggiani 1995), as shown in Figure 20. 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 was 9 %. Enveloping the raft, a wall of 800 mm diameter bored piles to 12 m depth were constructed to protect against scour. These piles were free from contact with the raft and the pipe piles. The unfactored load from the pier was 800 kN/pile, which incorporated a factor of safety of 3.0 on pile capacity as verified in static loading tests during the design. The foundation was instrumented to monitor the pile axial load in 35 piles and the contact stress between the raft and the soil at eight locations as the bridge was constructed. The monitoring continued for about ten years following the construction. Settlements were monitored by survey.

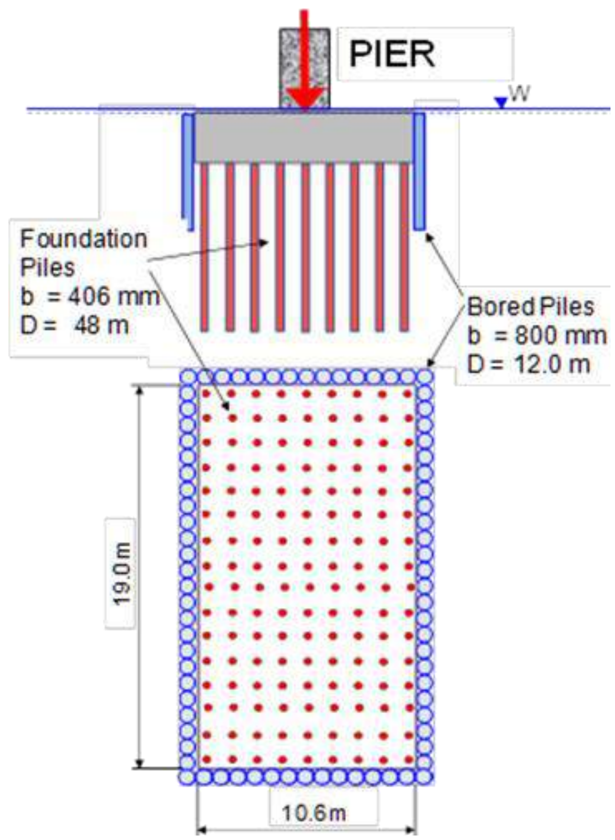

Figure 20. Pile layout at the main pier of the Garigliano bridge (after Russo and Viggiani 1995 and Mandolini et al. 2005).

At first, the measurements showed larger loads on the perimeter piles than on the interior, but, with time, the load on the perimeter piles decreased, while the load on the interior piles increased. Throughout the construction, the total load measured in the pile gages corresponded closely to the net weight from the raft and the pier. At the end of construction, the average settlement was about 42 mm. Ten years later, it had increased to 52 mm. Because of the rigid cap, no differential settlement developed.

The earth cells measuring the contact stress registered only negligible values, during the construction of the raft and the pier above. The authors suggested that the absence of significant contact stress was due to "the cells not working properly". No discussion of a possible reason for the "malfunction" is included.

The authors also suggested that the reduction of the load measured in the perimeter piles was due to creep of the reinforced concrete raft. However, I believe the settlement of the soil surrounding the pile group (the magnitude was not included in the paper) resulted in drag forces on the perimeter piles, causing the perimeter piles to be partially unloaded from the raft. In other words, the response of the interior and perimeter piles was very similar to that of the foregoing case history. The bored-piles were short, 12 m, and were not able to shield the 48 m long perimeter piles from the settling soil below 12 m depth.

Lee and Xiao (2001) compiled test records from Caputo and Viggiani (1984) on the results of static loading tests on three single piles, Piles 1, 2, and 3. While measuring the pile-head load-movement on the test piles, for each test, they also measured the movement of an unloaded, "passive" pile a short distance away. Piles 1 and 2, active and passive piles, had 400 mm diameter and 8.6 and 8.0 m embedment. Piles 3, active and passive piles, had 500 mm diameter and 20.6 m embedment. The side-to-side distances between the active and passive piles were 0.8, 1.2, and 3.0 m, respectively.

Figure 21 shows that loading the active piles induced small downward movement on the passive piles. For the rather small 6-mm pile head movement of the active piles, the passive pile movements were about 0.2 to 0.3 mm regardless of the distance between the active and passive piles.

For the wider and longer pile, Pile #3, the passive pile moved 0.8 mm when the active pile moved 30 mm. The movements of both active and

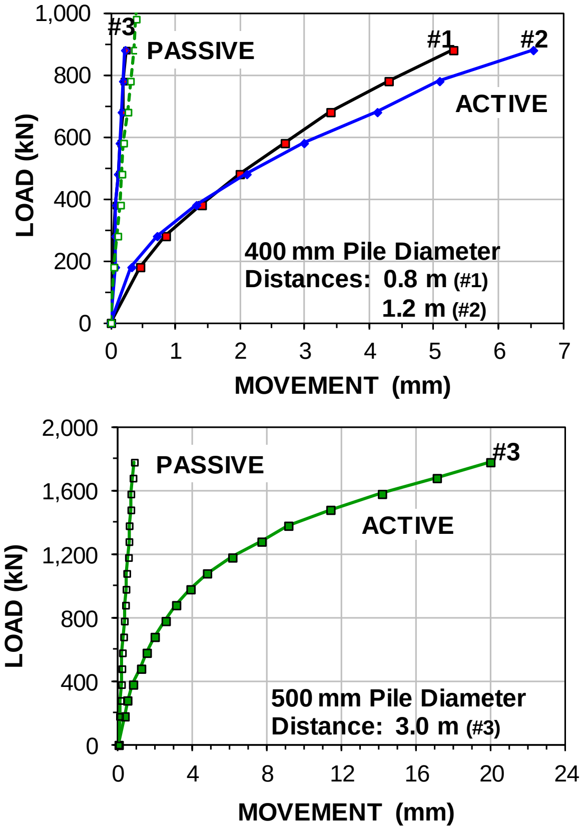

Figure 21. Movements induced in passive piles a distance away from test piles. (Data from Lee and Xiao 2001, Caputo and Viggiani 1984).

passive piles were probably measured against a reference beam that may or may not have been common to the piles, and whether the load was applied by jacking against a loaded platform or reaction piles, the reaction arrangement might have affected the measurements. The actual movement values matter less. The key point is that the tests showed that even when piles are separated by distance equal to several pile diameters, they will interact. Presumably, the presence of a passive or equally active pile near a loaded pile will have an effect on the axial loads and displacements of the loaded pile. However, to my knowledge there are as yet no such measurements available.

van Impe et al. (2013), van Impe (2016), and **Fellenius (2014b)** analyzed a case of settlements of three 33,000 m^3 in volume, 19 m in height, oil tanks, each supported on 422 piles. The piles were 460-mm diameter, 21.6 m long screw piles (Omega piles). Figure 22 shows that the soil profile consisted of a 15 m thick old fill of sand with clay deposited on about 4 m of silt and clay and 5 m of sand on a tertiary, slightly overconsolidated stiff clay at 24 m depth that continued for about 100 m.

Each pile cap was a 49 m wide and 600 mm thick reinforced concrete slab. The total load from the filled tanks was about 330 MN, giving an average maximum pile load of 780 kN and an about 200 kPa average stress over the tank footprint. The Footprint Ratio was 4 % and the average center-to-center pile spacing was 2.2 m (about 5 pile diameters). The pile raft was very flexible, but as the free length from pile to pile was short the slab can be considered capable of bridging the 200 kPa stress with minimal bending of the raft.

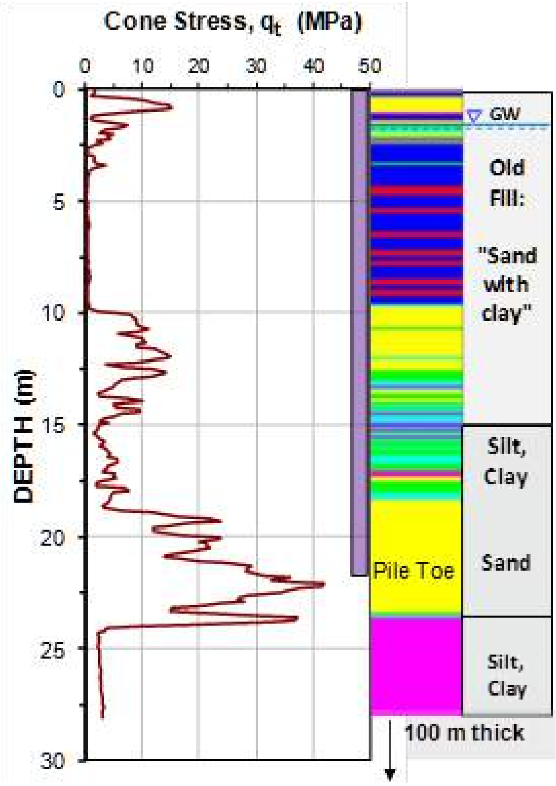

Figure 22. Cone stress diagram and soil profile.

Figure 23 show the pile head, pile shaft, the pile toe, and the pile compression load-movement curves of a static loading test. The results indicate no obvious ultimate shaft resistance. The pile capacity can be estimated to range between 2,750 kN at a pile head movement of 40 mm through 2,930 kN at about 70 mm movement. It is obvious that all the load applied to the tank will be carried by the piles with no load going toward a contact stress. Indeed, the settlement within the pile length will be small; as indicated by the measured pile compression, it would be only a millimetre or two. Therefore, settlement of the tanks will be governed by the compression of the tertiary clay underneath the sand "cushion" immediately below the pile toe level.

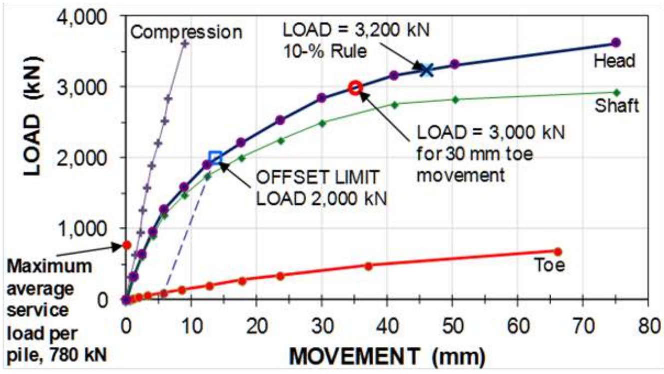

Figure 23. Load movement curves for head, shaft, toe, and compression (from Fellenius 2014b).

Hydro-testing of the 3 tanks was performed in April 2013, by filling the tanks with water to a height of about 18 m, which took about 3 days. The maximum water level in the tanks was kept constant for about 4 days, the tanks were then emptied over 3 days. The free distance between the tanks was 17 m, which is smaller than the 22-m depth to the pile toe level. Tank 1 was filled and emptied first. Tank 2 was then filled. This meant that the water load in Tank 1 was preloading the soil under Tanks 2 and 3; more under the side closest to Tank 1 than for the away side. Similarly, the water in Tanks 2 preloaded the soil under Tank 3.

After the four days of maintaining the maximum water height in Tank 2, it was emptied by pumping the water over to fill Tank 3. This procedure means that the filling of Tank 3 started when the stress from Tank 2 was present under Tank 3 and that the stress reduced at the same rate as the stress induced from the load in Tank 3 increased.

Figure 24 shows the observed settlements for Tanks 1 and 2 at maximum load and remaining settlement of Tank 1 after unloading. The settlement for the #2 gages (benchmarks) in Tanks 1 and 2 are about equal, while the settlement for #10 gages show larger settlement for Tank 1 than for Tank 2, possibly, because the preloading effect reduced the settlement under the Tank 2 side closest to Tank 1.

Figure 25 shows the observed settlements for all three tanks (settlement of Tanks 3 added) at maximum load and the remaining settlement of Tank 3 after unloading. No preloading effect similar to that shown in Figure 24 is noticeable.

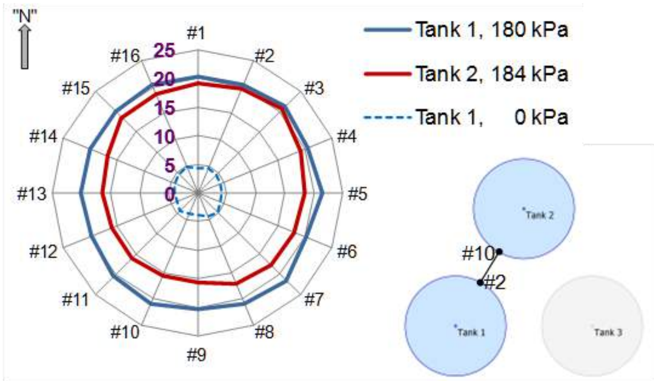

Figure 24. Perimeter settlements for Tanks 1 and 2 at maximum load and remaining settlement of Tank 1 after unloading. The North direction is assumed vertical. (Data from van Impe et al. 2013).

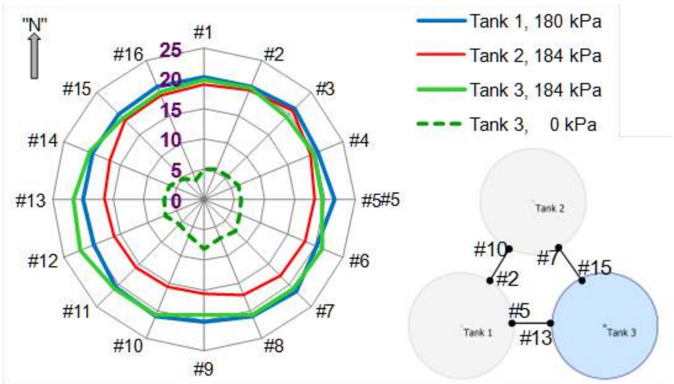

Figure 25. Perimeter settlements for Tanks 1, 2, and 3 at maximum load and remaining settlement of Tank 3 after unloading. The North direction is assumed vertical. (Data from van Impe et al. 2013).

Unfortunately, no samples were taken for testing from the clay, so no laboratory tests were carried out. For the here purpose of addressing the analysis method for the wide pile group, such values are not necessary. It is simple to assume that the three tanks can be analyzed as three equivalent piers each with an equivalent raft at the pile the pile toe level and calculate what modulus would fit the measurements. The so-determined modulus was used to find that the settlement underneath the centers would be about twice that of the perimeter. Of course, a four-day hydrotest does not provide much information on long-term settlement and without representative soil parameters, calculation for long-term development is not meaningful.

Gwizdala and Kesik 2015 reported settlement records taken on the Third Millennium Bridge in Gdansk, Poland. This is a cable-stayed road bridge spanning the Dead Vistula River and links the Northern Port of Gdansk with the national road network constructed in 1999 - 2001. The main bridge component is a single tower, a 100-m tall reinforced concrete pylon, consisting of a 52.4 x 22.4 m concrete slab supported on 50 bored piles of 1,800 mm diameter with 30-m embedment. The total unfactored load was 480 MN and the unfactored working load per pile was 9,600 kN.

As shown in Figure 26, the piled foundation has the form of two square grids of 25 piles each with a 5.8 m, 3.3 diameter center-to-center spacing and an about 12 % Footprint Ratio.

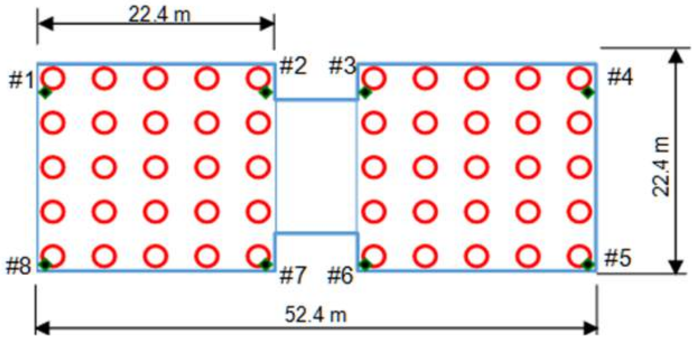

Figure 26. Pylon pile layout for the Third Millennium Bridge, Gdansk. (Data from Gwizdala and Kesik, 2015).

Figure 27 shows a vertical section of the foundation and the geometry toward the river. The foundation lies close to and parallel to the river. Over time, starting after the casting of the slab, the pylon settlement was monitored at eight locations around the slab, Points #1 through #8 Figure 26).

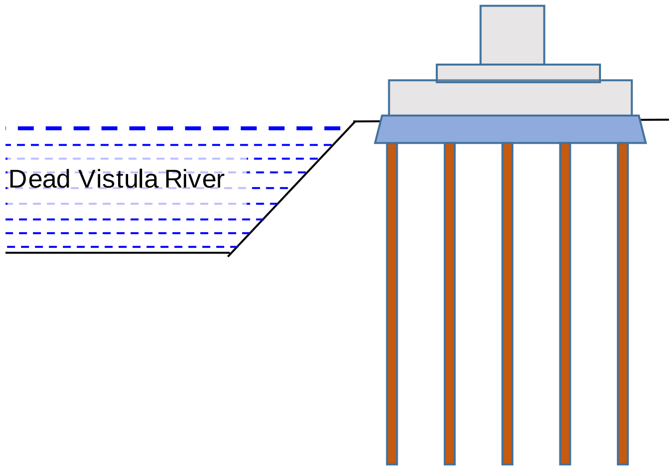

Figure 27. Vertical section and geometry of the pier.

Figure 28 shows that tower had a slight tilt, i.e., the measured settlements differed between the land side and the river side. Using the UniSettle software, I input the geometry, assumed values of compressibility of the pile and soil above and below the pile toe level, and modeled the piled foundation as an equivalent pier on a flexible raft at the pile toe level. I then calculated the calculated settlement at about 700 days (end of construction) and at about 1,700 days (after the following about 1,000 days of consolidation) and matched the result to the average measured settlement considering it to occur at the center of the foundation. The input geometry included modeling the river as an excavation, which resulted in an unloading of the deeper soils that was, of course, more pronounced on the river side than the land side, as reflected by Boussinesq distribution, which presupposes a flexible raft. As the bridge pier is definitely more rigid than flexible, strictly, the stresses should therefore be calculated for the characteristic point toward the two sides, which is where the stresses under flexible and rigid foundations are equal (Fellenius 2016). Either way, on matching the average calculated settlements to the average measured settlements, the simultaneously calculated Land-side and River side settlements were similar to the measured values. This indicates that the assumption of equivalent raft can model the settlement of the piled foundation.

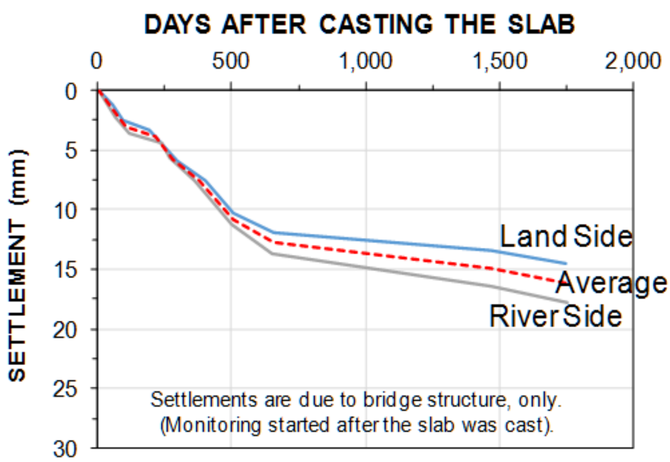

Figure 28. Measured settlements (data from Gwizdala and Kesik 2015).

The authors applied the Polish code to their analysis, which includes an equivalent pier approach where the bottom raft is wider than the actual raft at the foundation level. The widening approach is similar to that described above in Figure 6. By the Polish Code, the widening does not begin at the neutral plane depth, but starts at the pile head. The authors also applied an E-modulus taken from the geotechnical investigation and assumed it increased with depth to back-calculate the average settlement of the two rafts. The main difference between the two analyses is my inclusion of the unloading effect of the river.

3. CONDITIONS FOR CONTACT STRESS

Piles, be they single or in a group, are usually connected to a pile cap or a raft that transfers the load from the structure. In the long-term and for the sustained working load, a single pile and a group of piles in a narrow piled foundation will develop a force equilibrium at some depth, which means that the soil immediately underneath the pile cap will settle more the pile. Therefore, there will be no contact stress between the pile cap (raft) and the soil. When the conditions are such that the neutral plane lies right at the underside of the pile cap (the "factor of safety" would then be equal to unity or less), it is often thought that a contact and a contact stress would develop that would assist in supporting the applied load. If so, however, the strain developed in the piles by the applied load must be equal to that in the soil. Ordinarily, the strain introduced in the pile is approximately 100 microstrain. Most soils surrounding a pile would have a modulus that is three to four orders of magnitudes smaller than the modulus of the pile material. The corresponding soil stress for the imposed strain is therefore negligibly small and no contact stress is transferred to the soil. It is conceivable that some stress will be induced to the soil from the pile further down, much like the interaction and interplay of stress between the reinforcement and the concrete in a reinforced concrete element. However, any axial load that is shed to the soil is then transferred from the soil to a neighboring pile that, in turn sends some of its load to the first pile or to other piles. Similar to the case reported by Okabe (1977), there is then no reduction of load due to shaft resistance.

When load is applied to a wide piled-supported raft, the pier made up of the piles and the soil in-between the piles will compress for the load more or less responding as a single body (pier) affected by a uniform stress. (For interior piles under a wide raft, we may disregard the influence of the shaft resistance along the perimeter piles). At the pile toe level, the upward directed stress acting on the soil in-between the piles will cause an upward push—the soil immediately above the pile toe level will compress and the toe-level boundary will move upward in relation to the pile. The compression will appear as a pile-toe load-transfer movement that generates a shaft resistance up along the piles, gradually reducing the vertical stress in the soil by transferring the stress to the piles. This is illustrated in Figure 29, showing a pile-supported wide raft (say, 1.0-m diameter, square, concrete piles at a 3.3-m spacing). The in-between soil can be considered (analyzed) as an upside-down "soil-pile" with its "head" at the pile toe level, its shaft resistance along the pile (the square) in the center, and its "toe" at the cap level.

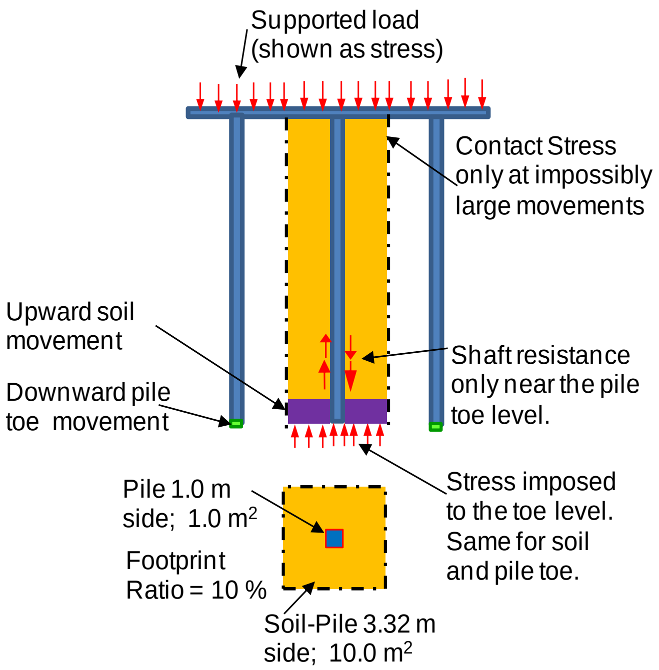

Figure 29. Section and geometry of piles and soil.

At the pile toe level, in contrast to the condition at the underside of the raft, the strain in the soil is independent of the strain in the piles. The soil will compress some distance up from the toe level until there is an equilibrium between the stress (coming from below) required to compress the soil and the shaft resistance built-up along a corroborating distance up the piles. The mechanism is similar to the that of the soil core inside an open-toe pipe-pile being pushed upward inside the pipe, generating a "core resistance" along a limited length above the pile toe (Fellenius 2015). The length is limited to the pile toe load-movement (i.e., the apparent pile toe penetration), which distance is very much smaller than the compression necessary to generate a stress in the soil all the way up to the pile head and pile cap level.

The apparent pile-toe movement and the maximum compression of the soil-pile are equal. Even with a very large toe movement of 30 mm or more, the compression of the soil would not extend to anywhere near the pile cap level.

4. A PILED FOUNDATION EXAMPLE

To expound the procedures of the Unified Method, the method is applied to a typical example of a wide piled foundation, say, a storage tank, as follows. A hypothetical site comprises a 15 m thick layer of soft compressible clay followed by 25 m of dense silt and sand on hard soil at 40 m depth. The clay layer is subject to a gradual regional subsidence. A structure imposing a 64-MN sustained load (no live load), will be constructed at the site. The foundation consists of a hexagonal piled raft with a 3.0-m side-to-side distance. The designers have decided to use 91 round precast concrete piles with 300-mm diameter, placed at a 3.0 diameter center-to-center equilateral spacing. Figure 30 shows the raft footprint and the soil profile.

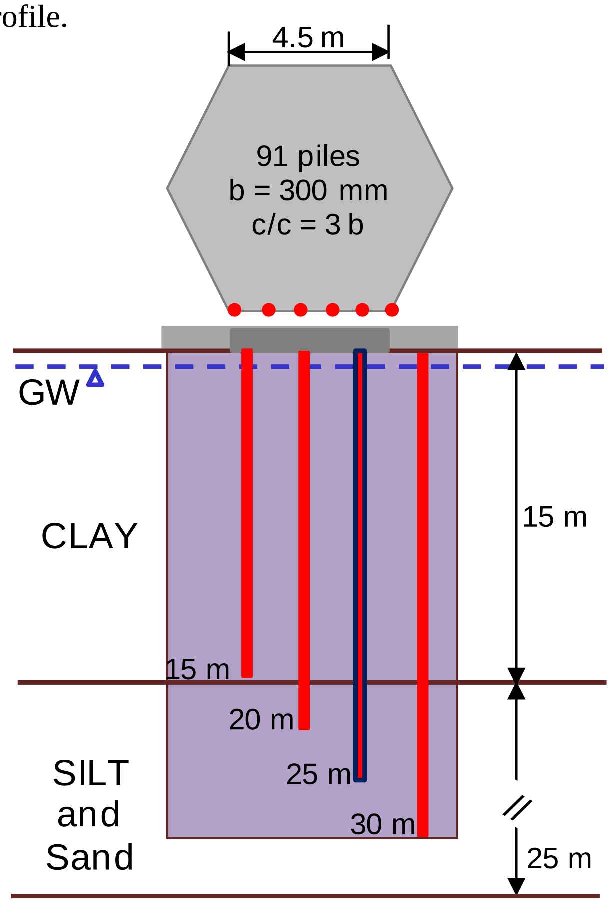

Figure 30. The raft footprint and soil profile with alternative pile lengths.

To assist the hypothetical design, four test piles are assumed installed to embedment lengths of 30, 25, 20, and 15 m. Figure 31 shows the hypothetical pile-head load-movement curves of the tests. The pile-toe response was assumed to be the same for all four tests and the pile-toe load-movement curve is included in the figure. The tests were terminated when the pile toe movement was 30 mm.

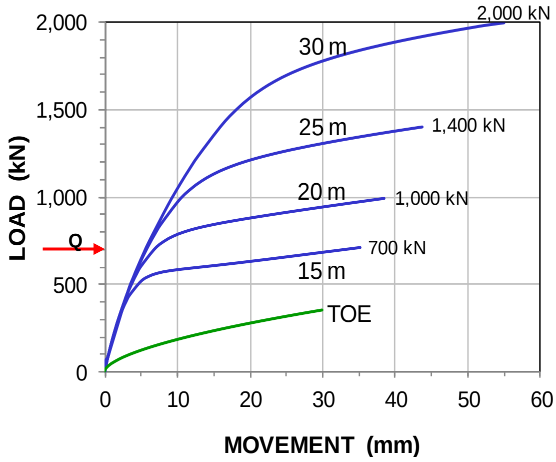

Figure 31. Hypothetical load-movement curves

Storage tanks frequently have outside appurtenances that need to be supported on single piles and small pile groups. If the working load (Q) applied to such piles outside the raft is the same as the average raft piles, i.e. Q= 700 kN, then, the required pile length appears to be 25 m. Not because the "capacity" of the 25-m pile appears to be around twice the working load, but because the long-term settlement is acceptable, as indicated by the results of the unified-method analysis shown in Figure 32 (only showing the load distribution for the match of the toe force and toe movement).

A single pile would be subjected to a drag force of about 150 kN, which would be of no consequence. The drag force would be smaller for a shorter pile, but that pile would have the neutral plane up in the settling clay and suffer excessive downdrag and be unsuitable. To design for a longer pile would be spending more money than warranted.

The various methods employing analysis and relations such as the RR and DR would provide estimates of the long-term settlements for the pile raft. Here, I will apply the equivalent pier and equivalent raft methods I used to back-calculate the case histories quoted in Section 2.

Equations 1 and 2 determine the pier stiffness. (The Footprint Ratio is 12 % and the pile modulus is assumed to be 30 GPa). The height of the equivalent pier is the pile length and, thus, the calculated compressions of the equivalent piers for the 15, 20, 25, and 30 m pile lengths are 5, 6, 8, and 10 mm, respectively.

The dominant settlement is that developing in the silt and sand layer below the pile toe level and it is a function of the compressibility of that layer. I made use of the effortless freedom of a hypothetical example and simply selected the compressibility that gave a 25 mm total settlement for the piled raft supported on 25 m long piles.

Using UniSettle, I then calculated, employing the same compressibility, the settlement of equivalent rafts placed at the other three depths. Figure 33 shows, as a function of pile length, the settlements calculated for the characteristic point of the raft of the equivalent pier, the equivalent raft, and the two combined, i.e., the total settlement for the piled raft. The equivalent raft values are shown as a bar, where the center of the bar is the settlement calculated for the characteristic point, which is representative for a rigid raft. The left and right ends of the bar are settlements calculated (using UniSettle) for the raft side and center, respectively, for the Boussinesq stress distribution,

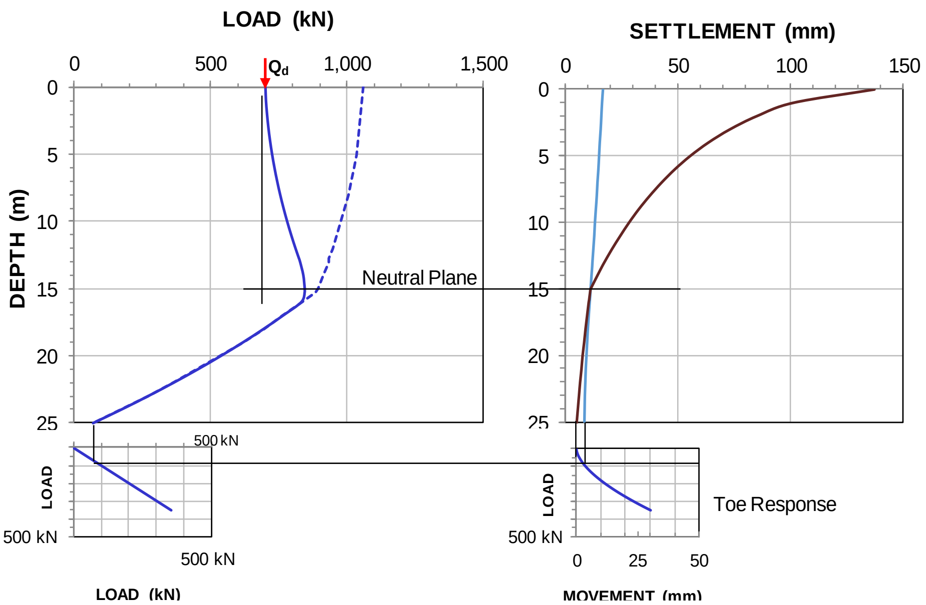

Fig. 32 Load distribution and match between toe resistance and settlement.

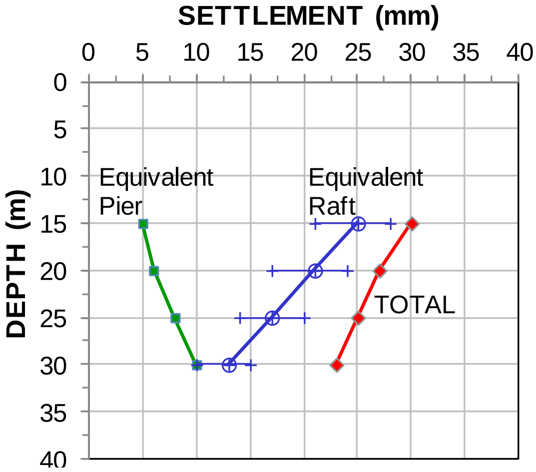

Figure 33. Settlements versus pile depth.

which is considered representative for a flexible raft. Thus, the bar indicates the range of the differential settlement of the flexible equivalent raft, and, therefore, also of a flexible piled raft.

Two results are evident from the figure. First, the settlement contributed by the equivalent raft for the example below the pile toe level is larger than that of the compression of the equivalent pier. Second, designing the piled foundation on shorter piles appears to result in very small additional settlement beyond those of the 25-m "pure" piled foundation. Obviously, were it possible to use fewer piles, potential savings could be realized. For example, if the spacing would be increased to 3.75 pile diameters, while keeping the dimensions of the raft, the number of piles needed would reduce to 61. The corresponding reduction of the FR to 8 % would result in an about 30 % increased calculated pier compression, a moderate value. The settlement of the equivalent raft would be the same. The total settlement for such an enhanced piled raft would increase to about 30 mm from the about 25 mm value for the "pure" option.

4. CONCLUSIONS

The key aspect to consider in foundation design is settlement. Past practice assumed that, if the factor of safety against bearing failure was adequate, then, the settlement due to the load would be acceptable to the structure supported. However, even when compensating for the fact that bearing failure, i.e., "capacity", is for most situations a very approximate and imprecise condition, this assumption is not always true. In contrast, a piled foundation for which the settlement, properly analyzed, is acceptable, will also have adequate safety against failure by any definition of the latter. A foundation design should therefore be directed toward determining settlement and letting capacity reasoning take second place. The unified method satisfies the requirement by employing interaction of forces and movements to determine the short- and long-term settlements for a single pile or a narrow pile group.

In regard to foundations supported on single piles or narrow pile groups, how the loads are transferred to the soil along the pile shaft and the pile toe together in interaction with the settlement in the surrounding soil governs the settlement of the foundation. In a homogenous soil deposit thicker than the length of the piles, the settlement for a piled foundation on short piles will be larger than that for a foundation on long piles, everything else equal. For both, the presence and magnitude of downdrag may decide whether or not the foundation is acceptable. (Note that issues of drag force, however, are only of concern with regard to the pile axial structural strength and are irrelevant to capacity and settlement considerations).

The case histories reported by Briaud et al. (1989) and O'Neill et al. (1982), indicated that small-group piles (5 and 9 piles, respectively) do not interact, but respond as single piles to the load.

The here quoted case histories and several similar cases presented by Fellenius and Ochoa (2016) have been back-calculated by several authors simulating the measured settlement response by means of various numerical methods. My review of the case histories show that the same records can be back-calculated employing the concept of an equivalent pier placed on an equivalent raft at the pile toe level employing only a response to the increased stress in the soil and including no ultimate resistance methods. That is, a wide piled foundation, whether it is considered as a "hybrid" raft, an "enhanced raft", a "piled raft", a "piled pad", or a "pure piled foundation" can be modeled as an equivalent pier on an equivalent raft placed at the pile toe level. The settlement is then calculated as the sum of the compression of the pile and soil making up a pier with a stiffness proportioned between the pile and the soil plus the settlement of the equivalent raft. The here quoted case histories provide interesting observations pertinent to some issues of wide foundations, but no analysis method applied to the case histories is shown to be clearly advantageous, superior, or more "correct" than another, including the equivalent pier and equivalent raft method.

The conventional understanding of the difference between a wide "pure" piled foundation and a piled raft is that the piles in the former have a "capacity", by some definition or other, that significantly exceeds the average working load applied to the foundation, whereas the "capacity" of the piles in the latter is at most about the same as the working load, i.e., the factor of safety is about 1.0, or even smaller. It is generally thought that a contact stress between the raft and the soil would develop for the latter type and that the contact stress would provide additional bearing resistance to the raft foundation as opposed to no contact stress (where the raft is above the ground level).

The contact stress analysis shown in Section 3 indicates that whether or not contact stress develops is a function of the pile shaft resistance, not the total resistance: contact stress would only develop if the applied load per pile is larger than the shaft resistance (defined per some movement criterion)—otherwise not. However, for a wide pile foundation, the strain in the pile (at the raft underside) must be equal to the strain in the soil immediately underneath the raft. The amount of strain decides the stress in the piles as well as the stress in the soil, that is, the contact stress. The strain imposed in the pile by a usual working load rarely exceeds 100 $\mu\epsilon$. That same strain in a natural soil with a 50 MPa upper limit of E-modulus, which would be a stiff soil, indicates a soil contact stress of no more than about 5 kPa. It may be quite possible that the contact stress cells reported by Russo and Viggiani (1995) and Mandolini et al. (2005) were working properly, after all.

Furthermore, the condition for the presence of shaft resistance along the pile surface, i.e., presence of a shear zone, is that the pile moves down relative the soil. However, near the pile underside of a wide piled raft (N.B., in contact with the soil), the pile can neither move more nor less than the soil. Right at the raft underside, therefore, there cannot be a relative movement between the pile and the soil, i.e., the pile does not slide past the soil, and no shaft resistance can be mobilized. If the contact stress would be a significant, the soil strain would be correspondingly large, but it could still be no larger than the pile strain, that is, the stress cannot be "significant", but must be negligible. Deeper down below the raft, the shaft resistance is governed by the movement due to the difference between the soil strain accumulated to compression (large) and the pile compression (small).

To mobilize shaft resistance, a pile has to move in relation to the soil, which it does not do at the connection to the pile cap (the raft). However, does the pile move against the soil further down the pile or is the soil just there for the ride? The case history by Okabe (1977) indicated that shaft resistance mobilized along interior piles would be smaller than that of a single pile, or perimeter piles, as it should be; it cannot be larger than the weight of the soil in-between the piles.

The observations reported by Lee and Xiao (2001) showed that a single passive pile reacted to the loading of another pile (active pile) located up to 3 m away, indicating that one pile will interact with neighboring piles in a group. The interaction may result in the piles and soil responding together as a single pier or block. This would enable the foundation to rely on shaft resistance along the perimeter of the pile group, that is, the foundation would benefit from the many times larger surface of the pier as opposed to that of the sum of the surface of the individual piles. It would be an interesting experiment to remove the soil along the outward facing side of the perimeter of a pile group and measure the resulting change. Say, by excavating a trench enveloping the group.

As also actually successfully designed and constructed enhanced piled foundations bear out, e.g., Tan et al. 2005, Poulos (2009; 2013), Katzenbach et al. (2009), Ching-Han You (2011), and Rudianto (2016), significant savings can be realized by selecting an enhanced piled foundations—a "piled raft employing fewer and shorter piles. However, there is a dearth of full-scale case histories reporting settlement measurements on wide pile-enhanced foundations, i.e., foundations on piles with a significantly larger working load per pile than would ordinarily be considered for a single pile at the site and no full-scale cases reporting measurements of strain and relative movement between the piles and the soil underneath the raft.

The settlement in the soils below the pile toe level can be modeled by applying conventional methods of settlement analysis to an equivalent raft loaded by the permanent load from the structure supported. To obtain a reliable design requires good quality data on the compressibility of the soils under the pile toe level and the potential influence of stress changes due to adjacent features of the site.

The equivalent raft method has the advantage of making it easy to incorporate excavations, stress from adjacent structures, variations of soil layer thickness and composition across a site, groundwater table fluctuations, elevation differences, and other influencing aspects. Moreover, the method allows for the time dependency, e.g., influence of consolidation, to be included in the analysis. The approach is simple enough to apply in a spreadsheet calculation, e.g., Excel, although professional engineering software, such as UniPile and UniSettle, greatly simplify and speed up the analysis.

As most pile rafts are more or less flexible, a larger portion of the applied load will go to the interior piles and a smaller to the perimeter piles. Moreover, because the perimeter piles face the outside soil, they take on larger shaft resistance than of the interior piles, especially so the corner piles. Because of the combination of smaller load and larger shaft resistance, the perimeter piles are frequently installed shorter than the interior piles. However, perimeter piles will eventually be subjected to fully mobilized drag forces. Same length or shorter, their axial response will therefore be softer than that of the interior piles, and the perimeter piles will not take their full share of the load on the raft unless they, instead, are longer. If the surrounding soil is "non-settling", the process would take time, but it will always occur.

The case history by Okabe (1977) shows a long-term difference in drag force between perimeter and interior piles. A similar observation was made by Liew et al. (2002) who reported measurements on a 17.5 wide tank raft supported on 137 shaft bearing piles in soft compressible clay and stated that the observations showed that the interior piles provided "more support stiffness" to the applied load on the raft than the perimeter piles. The pile diameter was 350 mm, the spacing was 1.5 m (4 pile diameters), and the Footprint Ratio was 6 %.

Case histories reporting the response of pile groups to an applied load are usually limited to measurements of settlement in a few points on the pile cap, usually along the perimeter, only, and often only over a very short length of time (hydro testing of tanks, for example). None of the case histories reports separately the settlement from the zone within the pile depth from that underneath the pile toe level. Yet, the several more or less sophisticated methods in use for analysis of piled foundation response apply assumptions regarding the distributions of forces from the pile cap to the pile and the soil, and between the piles and the soil with depth. The latter take the lead from the response of a static loading test on a single pile, frequently performed without any instrumentation. Most papers presenting foundation analysis methods verify them by back-analysis of observations. Such analyses, including the equivalent pier/raft method, applied to the mentioned case histories only verify that the model can be fitted to a set of records. However, any and every method can show a god fit in a back-calculation of measurements.

Many have "predicted" the measurements reported in the quoted case histories by applying various methods of analysis, some more complex than others, and all also found an agreement between the calculated and measured settlements. However, it is not enough that a model can produce a good agreement in a back-calculation, it has to be shown to work also in a true prediction, i.e., in a design condition.

The adequacy of the design calculations is not a function of the sophistication of the model or the computer program employed, but of (1) the adequacy of the soil information and (2) the quality and representativeness of the parameters used as input to the analyses. Both are somewhat lacking in the state-of-the-practice of foundation design.

Moreover, when a theory involves more than one or two parameters, before it can be stated that it truly represents actual response so as to be useful to predict a response, i.e., be used in the design of a foundation, the calibration of the method applied to the model using the measured response must also include measurements of the relevant input parameters of the model. If contact stress is a key part of the model, the analysis must be supported by measurements showing contact stress and soil strain to exist and that the measurements are commensurate with those assumed before the test. Calling a calculation "a prediction" does not make it one. Indeed, if the analysis model depends on pile-soil interaction, it is not sufficient to just measure axial load distribution in the piles. The soil forces and soil deformations must also be recorded.

To improve the reliability of the design of piled foundations, research building up case histories must include instrumentation and monitoring of response to load applied to a pile group and single piles, including recording not just the settlement of the pile cap, but also:

- the movements between the pile and the soil at depths, in particular at the pile toe level
- the distribution of strain and movement in the soil with depth
- the earth stress against the piles
- the axial load distribution both in perimeter and interior piles
- the pile toe penetration into the soil and this compared to that of a single pile
- the settlement below the pile toe level

It is not financially possible to carry out a stand-alone research project, but detailed instrumentation and monitoring of actual, well-defined projects (e.g., wide tanks) are needed. Direct research will have to be satisfied by studying small piled foundations. Figure 34 shows the layout of a forthcoming pile-group test to be performed in the ISSMGE TC212 experimental field in Bolivia. The pile group comprises 13 piles to be tested by simultaneous bidirectional tests on all piles (Phase 1) and, then, a simultaneous head-down test on all piles (Phase 2) measuring loads, movements, and strain in the pile and in the soil surrounding the piles. The results will be presented to the 3rd Bolivian International Conference on Deep Foundations in Santa Cruz, Bolivia, in April 2017 (https://www.cfpbolivia.com).

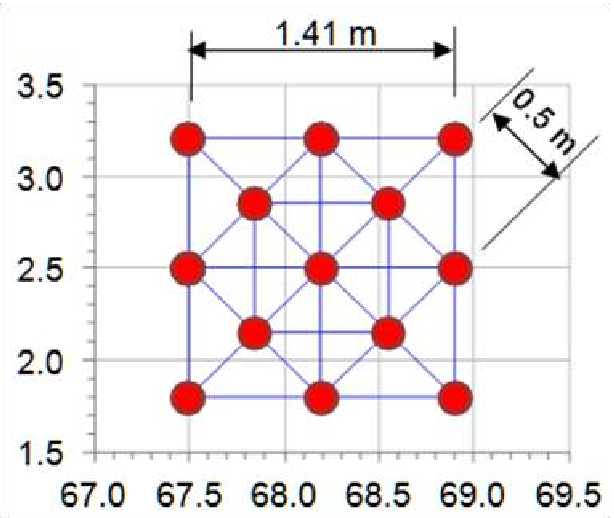

Figure 34. Layout of the pile group planned for the ISSMGE TC212 test in Bolivia April 2017.

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