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RESPONSE OF PILE GROUPS TO APPLIED LOAD AND GENERAL SUBSISTENCE PER THE UNIFIED METHOD AND US vs. EU CODES

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ABSTRACT The forthcoming update of the US AASHTO Specs will agree with the Canadian building code and the US Corps of Engineers code in regard to the analysis and design of piled foundations. Case histories on observations on full-scale piled rafts are reviewed and show that the response of interior and perimeter piles differ. Single piles and perimeter piles engage the soil from the ground downward. In contrast, interior piles engage the soil from the pile toe level upward. Interior piles and soil have strain compatibility, which determines the distribution of load between the piles, the raft contact stress, and the load-transfer movement. Particularly so in subsiding environment, because perimeter piles are subjected to downdrag and drag forces, while neither downdrag nor drag force will affect the interior piles to any appreciable degree. The difference will affect the load distribution and bending moment response across the raft, which is also governed by the degree of rigidity of the raft and by the difference in dishing at the pile toe level and in the dishing of the actual raft. Contact stress under the raft, be it large or small, is incidental to the response of a piled foundation and cannot be considered to reduce the effect of the load applied to the foundation.

Keywords: piles, settlement downdrag drag force, codes

INTRODUCTION

For the last several decades, the North American codes addressing pile foundations have shown a bewildering variety. Some are stuck to the old system of determining an "allowable load" from dividing the pile "capacity"—determined by a test or calculation—by a factor of safety, in effect, declaring that "if the capacity is good enough, then, the settlement is good, too". That some codes incorporate load-and-resistance-factor-design and, as a few do, add some serviceability factors, changes nothing. Plainly, a design aiming for a satisfactory ratio (factor of safety or resistance factor) between the sustained load and a perceived "capacity" does not necessarily establish a safe condition. The "capacity"-plus-factor approach is particularly inadequate where settlement additional to that from the supported structure is caused, for example, fills, excavations, site subsidence, etc.

Through the years, the perceived adverse effect of soil settling in relation to the piles accumulating to drag force was simply addressed by adding the drag force to the load from the supported structure, subtracting it from the "capacity", and requiring that the reduced "capacity" pile be "safely" larger in order for it to accept this load. This is the approach taken by the Eurocode, which still adds drag force to the load from the structure. However, starting in the 1960s, numerous full-scale studies of the phenomenon of negative skin friction developing drag force have been conducted showing that the mentioned approach to incorporating drag force (and downdrag) is invalid, indeed, very costly, and yet not always resulting in a safe foundation. This was recognized in the Ontario Highway Bridge Design Code 1992 (which in 2002 developed into Canadian Highway Bridge Design Code for Foundation, CSA-S6-06). The 1992 Ontario code recognized drag force as an environmental force only of importance for the pile axial strength and inconsequential for the geotechnical response of a pile. In 2012, the US Army Corp of Engineers, Design Guidelines stated that the drag force "*does not decrease the bearing capacity (geotechnical capacity) of the pile*". In contrast, the AASHTO Specs continued to add the drag force to the sustained load included the statement (terms adjusted): "*pile resistance available to support structure loads plus drag force shall be estimated by considering only the positive side and toe resistance below the lowest layer contributing to downdrag*". This unfortunate clause is applied in the US not just to the design of foundations for highway structures, but also to foundations for buildings and other structures. Regrettably, it is frequently applied also in Canada and Mexico.

The North-American situation has not been favorable to economical and safe piled foundation. To resolve the issues, the US Federal Highway Association, FHWA, established a committee to review the current design of a piled foundation in settling ground and propose a more rational approach. In 2024, the committee delivered an NCHRP report (Coffman et al. 2024), which, by the end of 2025, will be officially incorporated in the next edition of the AASHTO Specs. The main thrust of the NCHRP Report was to approach the long-term response of a pile to load and general subsidence and bring it closer to the codes of Canada and the US Corps of Engineers.

The NCHRP Report offers two methods, called Method 1 and Method 2 for the analysis of single piles. Both apply the unified method for pile analysis and design (Fellenius 1984; 1988, 2004). Method 1 accepts a simplified plastic ("ultimate") soil response for the pile shaft and pile toe resistances. Method 2 incorporates the shear-movement response of the pile shaft and pile toe resistances, so-called t-z and q-z functions. The following introduces the unified method as applied to single piles and to piled foundations comprising pile groups.

SINGLE PILES

t-z/q-z functions

The precursor to a settlement analysis of a pile is determining the movement of the pile due to an applied load and the associated distribution of axial force in the pile, making use of the available information on the pile, the soil, and, as is often needed, the results of an instrumented static loading test. A pile responds to applied load by moving a small distance, which generates shaft resistance and, when the applied load is sufficiently large, also toe resistance. Figure 1 shows six typical functions, describing the shaft and toe resistances vs. movement, called t-z and q-z functions, respectively. The functions can display quite an array of force-movements, depending on soil type and soil response. In the figure, all curves have been normalized to show both movement and force to go through an arbitrarily assigned common point, "**Target Poin**t". A single function coefficient applicable to each type of function alternative determines the shape of the selected function before and after the Target Point. The mathematics of the functions are presented in my Red Book (Fellenius 2025).

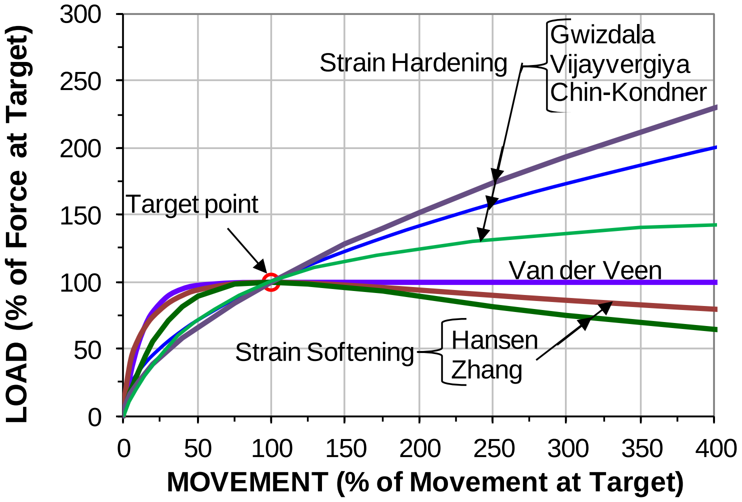

Fig. 1. Typical t-z and -z function curves

The t-z functions can range from strain-softening to strain-hardening. The simplest analysis is with the shaft resistance response assumed to be elastic- plastic similar to the vander Veen function. However, most often the actual response is hyperbolic (Chin-Kondner; Vijayvergiya) or parabolic (Gwizdala), i.e., initially steep and then gently increasing. Sometimes, the shaft resistance reduces (softens) with increasing movement (Hansen; Zhang). Sometimes, but far from always, a distinct change of slope, or curvature, appears at a force-movement, here termed "Target Point", but often not and the movement of the Target Point can be small or large. The t-z function depends on the soil type, soil densities, soil geologic origin (i.e., how created or formed), and mineralogy. Therefore, it is difficult, when faced with a design or analysis case, to choose a representative function type and coefficient, and target movement without well-founded experience, including back-analyses of pile tested under similar conditions in the particular geology.

Piles are made up of a series of pile-elements and the t-z/q-z curves represent the force-movement response of an element to an applied force. Where the response can be represented by the vander Veen, Hansen, or Zhang functions, an ultimate resistance, a logical "capacity" value, can be assumed represented by the indicated peak resistance, here the Target point. Others would be decided by some arbitrary definition.

"Capacity" assessed from the Gwizdala, Vijayvergiya, and Chin-Kondner functions have no visual relation to the shape of the force-movement curve and require a specific definition to identify an ultimate value. There are many such definitions around and most are rather randomly chosen; the profession has neither an agreed-on definition to apply, be it a specific force-movement t-z curve for a pile element or a pile-head load-movement. Note, also, that the accumulated sum of the "capacities" of the individual pile elements is not the "capacity" of the pile other than in the rare event that all pile elements have plastic resistance (vander Veen function).

The toe function (q-z) is always strain-hardening and usually best simulated as a Gwizdala function, showing ever increasing resistance with increasing pile-toe movement. The pile toe is not fully engaged until all pile elements have mobilized significant shaft resistance. Once the shaft resistance of the pile element encompassing the pile toe is mobilized, most of the additionally applied load is normally pile-toe resistance. The response of the pile-toe element is obviously very important.

The Unified Method for Single Piles

The principles of the Unified Method (Fellenius 1984; 1988; 2004; 2025) are very basic. Figure 2A shows distributions of axial force, N.B. unfactored, for a series of applied loads to 30 m long pile installed in a two-layer soil deposit. (The curves should be thought of as originating from the pile toe). The portion of the applied load reaching the pile toe, i.e., the toe resistance (R_t), is indicated at the pile toe. The dotted force curve starting from an assumed sustained load, Q_d, indicates axial force that increases with depth due to accumulated negative skin friction. Each intersection of the latter curve with a force distribution curve is a potential "force equilibrium"—a "neutral plane" (first reported observed in full-scale tests by Johannessen and Bjerrum 1965 and Endo et al. 1969). A horizontal line is drawn from each such intersection. The curves of the example trend to be parallel, indicating that the soil response is plastic once the pile toe is engaged.

Figure 2B shows the horizontal lines from the potential force equilibriums intersecting with the expected long-term settlement distribution ("soil subsidence") curve. Each such intersection indicates a potential settlement equilibrium, i.e., depth where the settlement of the pile is equal to the settlement of the soil, with the sloping straight line representing the pile compression and its value at the pile toe level and at the pile head level indicating the pile toe movement (δ_t) and corresponding pile head settlement, respectively.

A true force equilibrium can only occur at a depth where there is no movement between the pile and the soil, i.e., the settlement of the pile is equal to the settlement of the soil, the intersections of pile-force line and subsidence curve, the "settlement equilibrium". The figure shows an infinite number of force and settlement equilibriums. Only one of these equilibriums is true, which is when the toe force in Figure 2A is the toe force that results in a pile-toe movement for one of the "settlement equilibrium" intersections shown in Figure 2B is at the same depth as the force equilibrium, when the particular q-z relation of toe force and toe movement is satisfied. The depth for which this condition is satisfied is the depth of the "Equilibrium Plane" ("neutral plane").

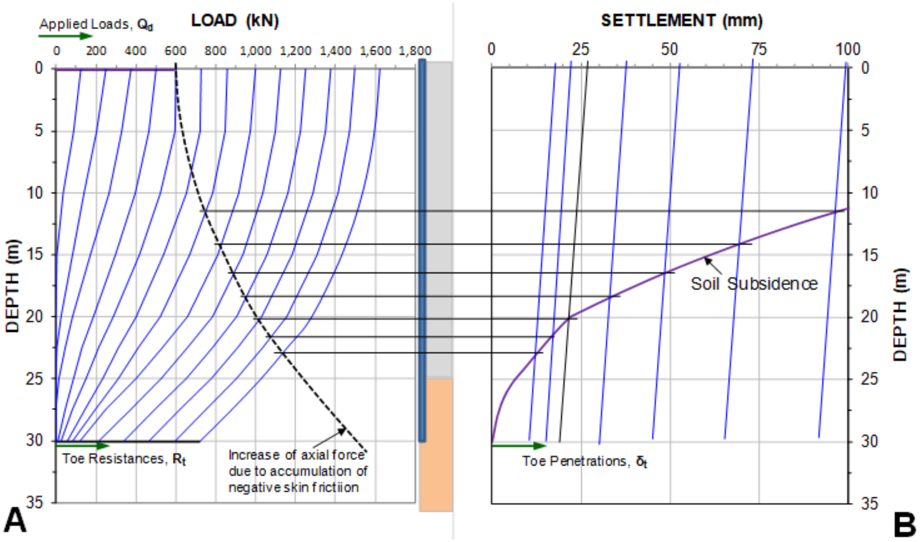

Fig. 2. Distributions of pile-force, pile-settlement, and soil subsidence

Figure 3 uses a graphical procedure to illustrate the principles of the Unified Method and how to determine the depth to the Equilibrium Plane and the long-tem pile settlement—the objective of the analysis. For conditions of plastic soil response, the force distributions curves are essentially parallel, equal, but for the fact that they rise from increasing values of toe resistance. Therefore, the analysis can be carried out using a simple spread sheet, a somewhat time-consuming effort, however.

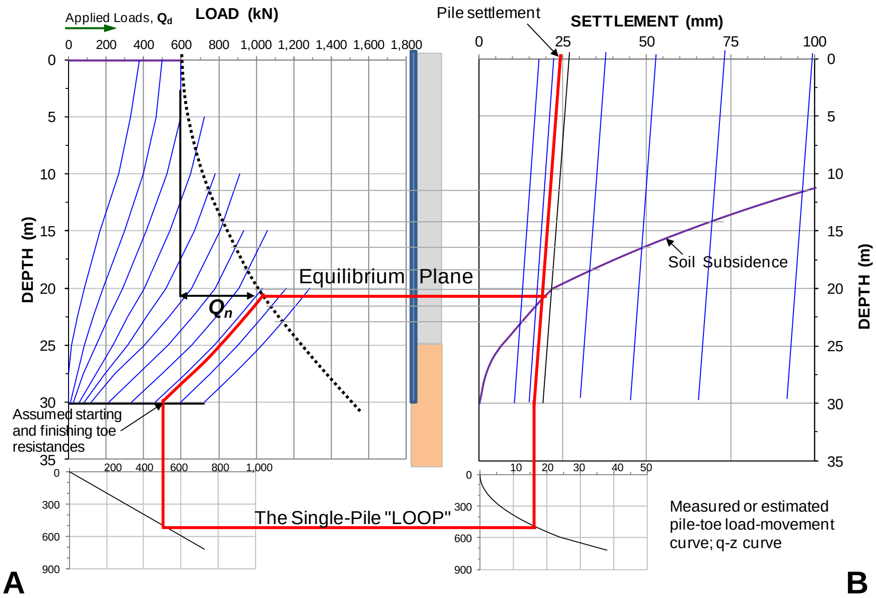

Fig. 3. Loop" determining the Equilibrium depth and the pile settlement

For any toe resistance value, the force equilibrium depth is relatively simple to determine as it is not that dependent on the t-z function. However, unless the soil response is plastic, it will be difficult to get the maximum force right, but that matters less (the maximum force establishes the drag force, but that is a factor for assessment in regard to structural strength and of no concern for the geotechnical analysis). The pile settlement, a more or less straight vertical line, is therefore easy to determine. Its intersection with the soil settlement line determines the settlement equilibrium. The key action is a trial-and-error procedure applying different toe forces and toe movements per the q-z relation to establish where the force and settlement equilibriums appear at the same depth, constituting the Equilibrium Plane. Method 1 applies plastic shaft resistance and Method 2 considers that shaft resistance follows a t-z relation, usually a hyperbolic strain-hardening function (c.f. Figure 1).

The graphical process is to, first, draw a force distribution curve downward from the pile head, i.e., a diagram of force versus depth (Figure 3A). The curve starts with the applied sustained load and increases with the load due to negative skin friction accumulated along the entire length of the pile—the dotted curve. Second, a series of force distribution curves are drawn upward from a couple of potential pile toe forces, showing the axial force increasing with accumulated positive shaft resistance. Each intersection of the downward increasing force distribution curve and an upward increasing curve is a force equilibrium and a potential Equilibrium depth. The analysis is best performed using effective stress. Stress-independent analysis (i.e., fixed shaft shear; undrained shear strength values) can give very misleading results.

Third, the soil subsidence is plotted in a diagram of settlement versus depth (Figure 3B). The soil settlement curve can be established from calculations in the same spread sheet or obtained separately from other source. (The important matter is that the settlement is calculated for the actual pile location as the soil is affected by the various fills, loads, and excavations determined by, say, Boussinesq distribution).

The Settlement Equilibrium depth is where the pile settlement line (i.e., toe movement plus compression, and it is only approximately straight) intersects with the soil settlement curve. (The pile settlement line is determined from the pile compression calculated from the axial force distribution and rising from a toe movement correlated to the pile toe force). That is, the Equilibrium depth is where the "Loop" (red lines shown in the figure), satisfies the force and movement relations for the pile, primarily, the q-z function. The 'satisfying' "Loop" starts at a pile toe force, rises to intersect with the downward force distribution, proceeds horizontally to intersect with the subsidence curve, goes downward to the pile toe, indicating a pile toe penetration that matches the force per the q-z relation. The settlement indicated at the pile head is the downdrag for the pile under the loading conditions and the double-arrow indicates the associated drag force, Q_n.

The drag force is of no concern for the response of the piled foundation; only of concern in regard to the structural strength of the pile. The transfer from negative skin friction to positive shaft resistance occurs gradually in a transition zone that reduces the magnitude of the drag force so no "kink" appears in the force curve. However, this is not shown in the figure.

N.B., even when the t-z response is plastic, the manually produced spread-sheet assisted solution will likely require investing several hours of work. Manual spread-sheet calculation for non plastic t-z relations, the more common condition, is still possible, but would need investing considerable time.

The unified method applies equally well to analysis of piled foundations where the long-term settlement is minimal, essentially comprising small secondary compression or "creep" settlement. Those cases will show small shaft resistance, in either negative or positive direction, along an upper length of the pile and some shedding of force along the lower length due to shaft resistance and, then, the balance, the toe resistance. The pile long-term settlement will be the pile-toe movement plus the pile compression.

PILE GROUPS, WIDE AND NARROW

It was once common to design pile groups by assuming the capacity of the group to be equal to the capacity of same number of single piles reduced by an "efficiency coefficient", defined as the ratio of the group-capacity to the sum of the individual piles. This approach originated in the fact that a group of piles will sometimes induce appreciable settlement in the soil below the pile toe level while well spread-out single piles do not. Thus, at equal load per pile, the group will settle more than the single pile. The "group efficiency" approach, e.g., Converse-Labarre equation (Bolin 1941) and others, attempt to adjust to this fact by an oblivious transfer from difference in settlement to a presumed difference in "capacity". Yet, the response of a single pile tested within a group is about the same as that of a single pile tested outside the group. Design by applying a group bearing efficiency factor is not realistic. For example, Terzaghi and Peck (1948) stated: *Attempts to evaluate group action by efficiency equations are likely to be misleading. ... therefore, efficiency equation should not be used*. On the same subject, in the third edition (1996) they added: *It is preferable to evaluate the settlement of a proposed pile foundation on the basis of physical properties of the soil into which the load is transmitted by the piles*. Regrettably, the neat mathematics of the efficiency approach still appeals to many, it seems, because efficiency equations still pop up in actual designs. However, the response of a pile group to an applied load, be the group small or large, narrow or wide, has to be analyzed in terms of settlement, which involves the distribution of forces between the interior and perimeter piles in the group, the shaft rigidity, the raft stress, the settlement below the pile-toe level, the interaction between pile-toe and pile-shaft, and the outside features, such as fills, excavations, site subsidence, etc. There are not many full-scale observations on piled raft settlement, but the few available do illustrate the response of pile groups to load.

Contact Stress and Strain Compatibility

That the contact stress below a piled raft cast on the ground in-between the interior piles would contribute to the bearing of the raft is a widespread fallacy. However, just like the stress distribution between rebars and the concrete in a reinforced concrete column, the distribution of the raft load to the piles ("rebars") and to soil ("concrete"), respectively, follows the principle of strain compatibility per the respective areas and E moduli of the materials. That is, the force in the piles and in the soil will be according to each total area and E modulus, the strain, ϵ, being equal for the soil and pile. The total contact load for a pile group comprising closely spaced piles will be smaller than for groups with widely spaced piles. This is, of course, because the free soil area, A_{soil}, is smaller where piles are closely spaced and larger where the piles are widely spaced, determining the ratio between the load portion on the soil and that on the piles, $E_{soil}\, A_{soil}\, \epsilon$ and $E_{pile}\, A_{pile}\, \epsilon$, respectively. For case records showing this, see Auxilia (2009) and Yamashita (2011; 2012; 2013) and Fellenius et al.(2019).

Moreover, while the E-modulus of the pile does not change with depth, the E-modulus of the soil will likely differ between the various soil layers. Therefore, the contact stress (by definition immediately under the raft) is usually large, because, there the soil is usually engineered backfill with an E-modulus much larger than that of the natural soil. Further down in the natural soil, the E_{soil} is small, therefore, the force in the soil will be smaller and the axial force in the piles will be correspondingly larger than just below the pile head. This is independent of whether the total area of the piled foundation and the total area of the piles, the Footprint Ratio, is small or large. Fellenius et al. (2019) reported a full-scale static loading test on a pile group comprising thirteen 300 mm diameter, 9.5 m long pressure-grouted bored piles installed in silty sand and connected to a rigid pile raft. Figure 4A shows a diagram of the average load-movement of the pile group and of perimeter and interior piles measured in the test and Figure 4B shows a schematic distribution of axial force from the pile head down to the pile toe for an interior pile. Note that the E-modulus in the natural soil being smaller than that of the engineered backfill, resulted in a reduction of the soil force and an increase of the strain represented by increase of axial force—i.e., a transfer of force to the piles—establishing a new strain equilibrium for the unchanged total force, pile and soil.

Distribution of Load between Interior and Perimeter Piles

Figure 4B also shows that pile shaft was engaged from the pile toe upward and that no shaft resistance developed along the upper length of the pile. Indeed, as first stated by Franke (1991), when load is applied to a group of piles, the shaft resistance on interior piles is not mobilized from the head downward, the way it is in a single pile, but from the toe to upward. This means that for an interior pile, in contrast to that for a perimeter pile, shaft resistance does not develop along the upper length of the pile. For a flexible raft and uniformly distributed load, therefore, both the compression and pile toe penetration will be larger for an interior pile than for a perimeter pile.

However, because perimeter piles are affected by shaft resistance starting at the ground surface, their response is stiffer than that of the interior piles. Therefore; in case of a rigid raft, the perimeter piles will receive a larger sustained load as opposed to the interior piles. In case of ongoing subsidence of the area around the pile group with a rigid raft, the reverse will occur: the perimeter piles will move (compress) more, due to downdrag, and the interior piles will receive the portion unloaded from the perimeter piles.

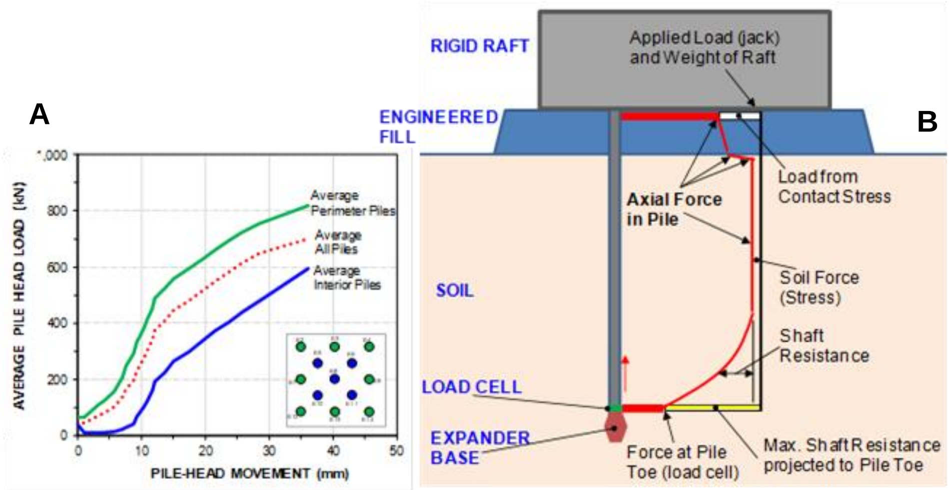

Fig. 4 Pile-head load-movement curves and interior pile distribution of axial force

The Franke principle is illustrated in the following hypothetical example comprising a wide piled-raft foundation supported on 355 mm diameter, round concrete piles at a three-diameter spacing in a square grid and constructed to 22 m depth in a soft soil transitioning to a dense sand at 20 m depth. The applied unfactored sustained load is 800 kN/pile. The live load is 200 kN/pile. A small fill placed across the area outside the foundation footprint will result in long-term subsidence at the site amounting to about 25 mm. Figure 5A illustrates the analysis procedure for where the toe force is equal to the upward movement (compression) of the soil in-between the piles. According to the Franke principle, the shaft resistance is then equal to the balance between the toe force and the applied load. The blue curve shows the pile toe resistance versus toe movement, the q-z curve, as determined in a test or by 'informed' analysis. The burgundy curve shows the applied load subtracted by the shaft resistance engaged upward from the pile toe plotted against the pile toe movement—this curve can be obtained in a bidirectional static loading test, real or simulated, as shown in Figure 5B, for the bidirectional cell placed right at the pile toe (had the test been real, the downward curve would have to be extrapolated beyond about 500 kN).

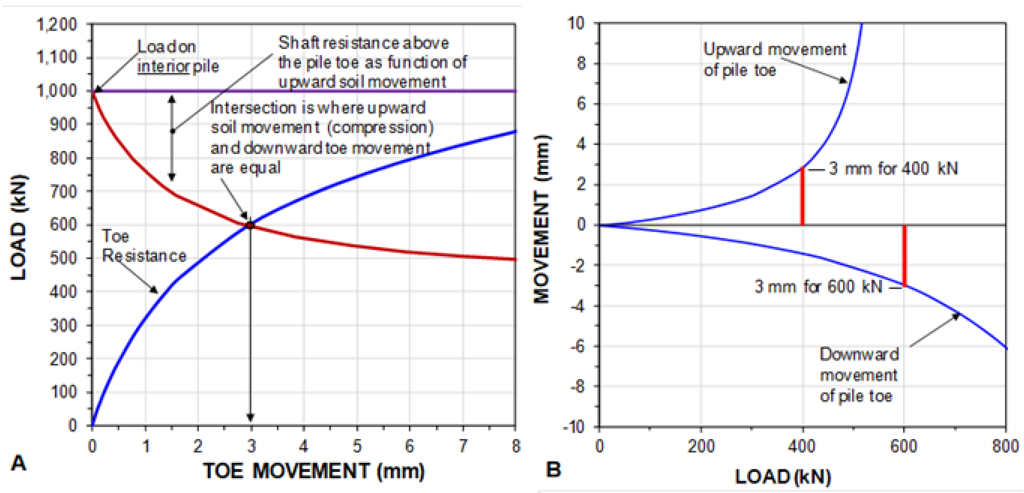

Fig. 5. The Fellenius-Franke method for determining the load-transfer toe-movement and toe-force for interior piles

The strain compatibility principle, i.e., that the strain is the same in the interior pile and the soil, means that the applied load causes no relative movement between the pile surface and the soil in the interior of a pile group. Thus, there is no shaft resistance along the interior piles and the latter piles will transfer their full share of the load to the pile toe. This is similar to absence of shear force between rebars and concrete due to the applied load in the reinforced concrete column. (The toe response is addressed below). In contrast to the interior piles, the perimeter piles will shed load due to shaft resistance.

As the load reaches the pile toe level, the pile toe moves down, which is the same mechanism as when pushing the soil upward starting at the pile toe level. The pile toe response depends on the pile toe soil stiffness (force-penetration relation, i.e., the particular q-z function). The distance the pile toe moves into the soil (the pile toe penetration) is equal to the soil's upward movement along the pile The process generates shaft resistance in the zone immediately above the pile toe, which reduces the force reaching the pile toe.

Obviously, the response of interior versus perimeter piles will be insignificant in a 3 by 3 group of piles: 8 perimeter piles versus 1 interior pile. Similarly, a 4 by 4 group will have 12 versus 4 piles and a 5 by 5 group will have 14 versus 9 piles. A design of a pile group, must therefore differ between narrow group (groups with four or fewer rows of piles) and wide groups (groups of five or more rows).

The response of an interior pile in a wide piled foundation differs from that of the piles in a narrow piled foundation. The response of a perimeter pile, i.e., the outermost row—sometimes, also the next row in—is similar to that of a single pile.

The mentioned difference was first observed by Okabe (1977) when measuring axial force distribution in interior and perimeter piles driven through compressible clay to bearing in dense sand. Figure 6 presents the records in four instrumented piles in the group and a single pile nearby after 4.5 years. The site was subjected to general subsidence due to water mining that developed general subsidence and significant drag force in the single pile. Measurements of axial force in the perimeter pile showed it to have a drag force about equal to that for the single pile. However, the interior piles were neither affected by negative skin friction nor by positive shaft resistance. Note that the perimeter pile did not support the raft, but instead showed a pull force at the foundation raft level. Also that my moving the perimeter-pile force-distribution curve to start from zero pile-head load in the figure (dashed curve less the circled point), shows that its response was practically the same as that of the single test pile.

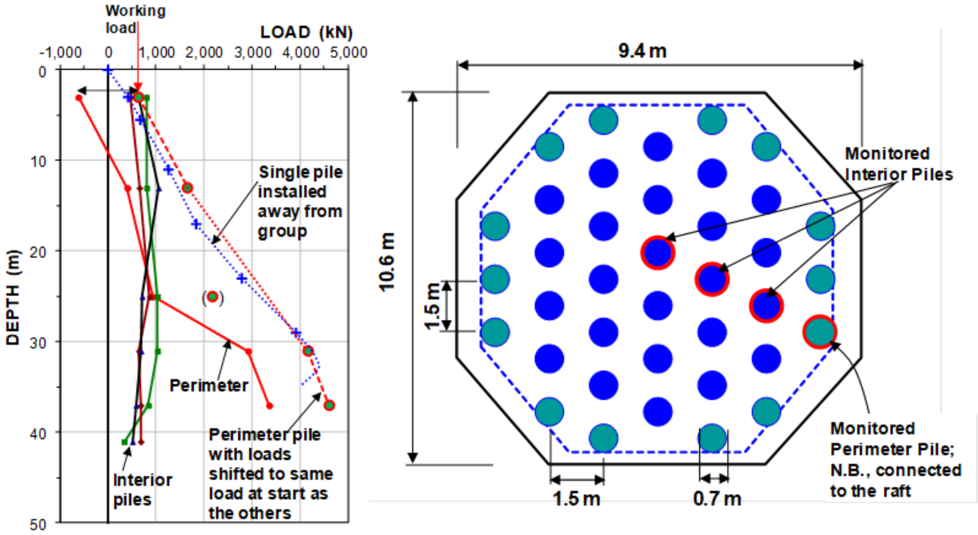

Fig. 6. Axial pile loads measured 4.5 years after construction (Okabe 1977)

A raft can be either rigid or flexible (and anything in between). Figure 7 shows typical (qualitative) load-movement response for a perimeter pile and an interior pile. For a certain total pile load applied to the raft, if the raft is rigid, the pile head movements are equal for all piles. Then, because the shaft resistance for a perimeter pile develops from the raft level (or ground surface), the response of the perimeter pile is stiffer than that of the interior piles and, therefore, the load at the head of the perimeter pile will be larger than that of the interior pile. If the raft is flexible, the loads will be equal, and, because of the response of the perimeter pile is stiffer, its movement will be smaller than that of the interior pile. (The plotted movements do not include the effect of the settlement below the pile toe level). As a raft is never totally rigid or totally flexible, the actual load of any case will be somewhere in-between the extremes, as the red circles indicate in the figure.

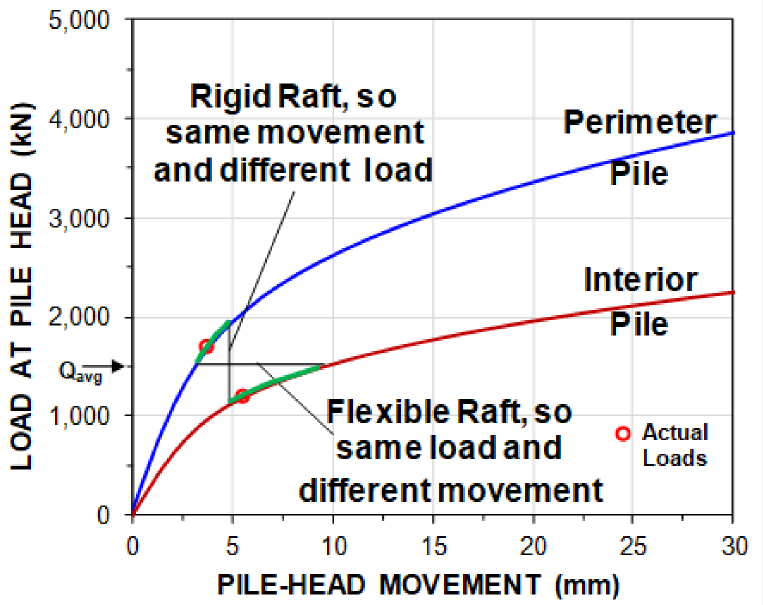

Fig. 7 Comparison of loaded distribution for a perimeter pile and an interior pile in a wide pile group

Fellenius (2019) reported a numerical analysis of load and settlement for an 8.4-m diameter, rigid piled raft supported on a wide group of 49 piles in a 50 m thick deposit of silty sand with a 40-MPa soil E-modulus and a ß-coefficient of 0.56 (from an assumed 35-degree internal friction angle and an earth stress coefficient, K_0= 0.8). Linear soil response was assumed with plastic shear after 5 mm movement. The numerical analysis (Plaxis) imposed a series of 10-mm raft movements to a 40 mm maximum raft settlement. The raft was cast on the ground surface.

Figure 8A shows the calculated soil settlement distributions and the calculated pile movement. The 40-mm pile-head movement (raft settlement) was the same across the raft (it being rigid). i.e., same for all piles. Because of the relatively small pile compression, all pile toe movements were just about slightly less than the 40 mm imposed for the pile head. For the interior piles, there was minimal movement between the pile and the soil, but for a length along the lowest part (up from the pile-toe level). In contrast, the perimeter piles moved more than the soil, thus, engaging shaft resistance and this stiffer response caused them to receive a larger portion of the sustained load and a larger pile compression

Zhang et al. (2025) did a similar numerical analysis of a large pile group, 1,600 21 m long piles, supporting an 80 m wide LNG tank (flexible raft above ground). Similarly to the foregoing analysis, the results showed that the perimeter piles engaged the soil from the pile-head downward, while the interior piles engaged the soil from the pile-toe upward. That is, both numerical analyses agreed with the presented principles of response of wide pile groups to applied load.

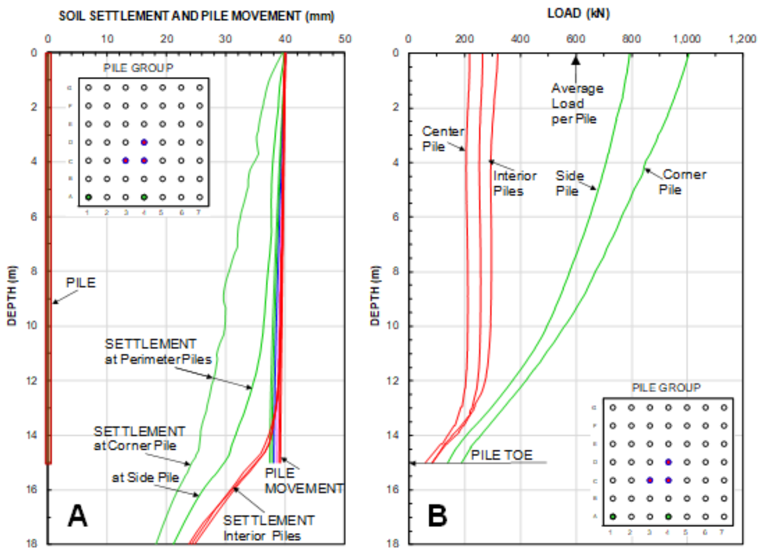

Fig. 8 Results of numerical analysis.
A. Soil settlement and pile movement. B. Axial pile loads

Settlement below the pile toe level

Be the spacing or the contact stress large or small, the response of the piled raft, of any rigidity, is compression of the equivalent pier system (piles and soil) plus settlement of the soil below the pile toe level. The compression of the pier system is determined by E_{pier}, the combined E-moduli (E_{pile} and E_{soil}) in relation to the respective areas of piles and soil (Fellenius 2016; 2025). The settlement of the soil below the pile toe level can be calculated as that of a flexible equivalent raft at the pile toe level loaded by the same stress as applied to the foundation raft, as shown in Figure 9. Note, the analysis must include the influence of stress changes due to adjacent other foundations, fills, excavations, and changes of groundwater table.

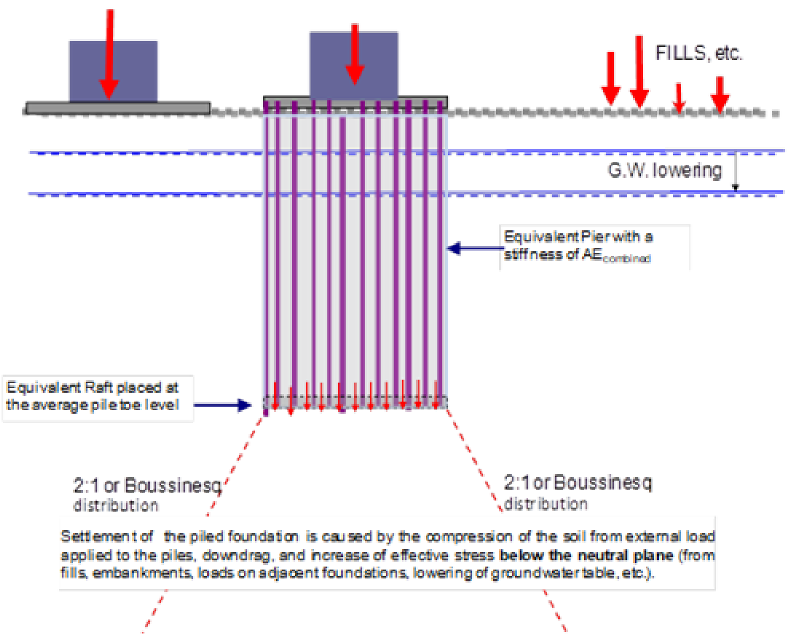

Fig. 9. The equivalent raft for calculation of settlement below the pile toe level

"Floating piles", i.e., shaft-bearing piles in soft compressible soil, can drastically reduce total and differential settlement of a raft or embankment placed on the piles. Broms (1976) reported settlement measured for two rectangular embankments on a 15 m thick deposit of compressible soft clay. One of the two embankments was supported on a grid of 500-mm diameter, 6 m deep lime-columns ("soft piles") placed at a center-to-center spacing of 1.4 m (2.8×column-diameter). The Footprint Ratio, i.e., ratio between total column ("pile") area and total footprint area, was about 10 %, an important parameter, because the E-modulus of the equivalent pier is about equal to the pile E-modulus times the Footprint Ratio (Fellenius 2019; 2025).

Figure 10 combines the measurements from the two embankments taken at 16, 65, 351, and 541 days, respectively, after constructing the embankment. "Column Area" shows records under the embankment supported on 6 m long lime-columns and "Reference Area" shows records from an adjacent embankment with no columns. The measurements established that the settlement for the column-supported embankment was not only smaller, but also more uniform than under the no column reference area. The settlement records of both embankments can be fitted in a back-calculation, using the same set of soil parameters for both and applying the equivalent raft concept.

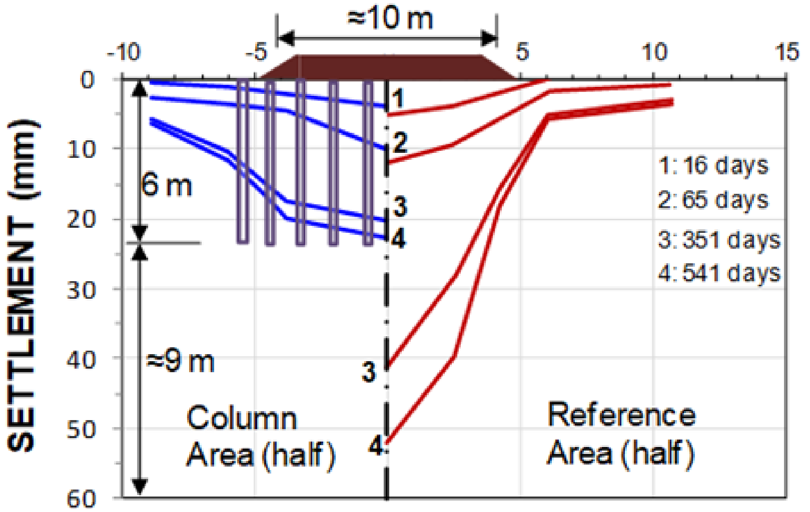

Fig. 10. Settlement of lime-column supported and unsupported embankments on soft compressible clay (data from Broms 1976)

However, Figure 11 compiles a case records presented by Badellas et al. (1988) of a flexible silo raft in Thessaloniki founded on piles, again confirming the equivalent raft approach (also addressed by Georgiadis et al. 1989 and Savvaidis 2003).

Gwizdala and Kesik (215) reported observations on the Gdansk Millennium Bridge. 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 and 30 m embedment. The total unfactored load was 480 MN and the unfactored working load per pile was 9,600 kN. The soil profile consisted of 25 m of interbedded clay and sand deposited on sand. The foundation lies close to and parallel to the river, which meant that the effective stresses on the river side are smaller than on the opposing side (Figure 12). Starting after the casting of the slab, the pylon settlement was monitored over time. Settlement analysis considering the non-equal distribution of the initial effective overburden stress agreed well with the observed differential settlement shown in Figure 13 between the wider side and the land side, demonstrating the importance of the analysis to take into account any difference in effective stress distribution below the raft area.

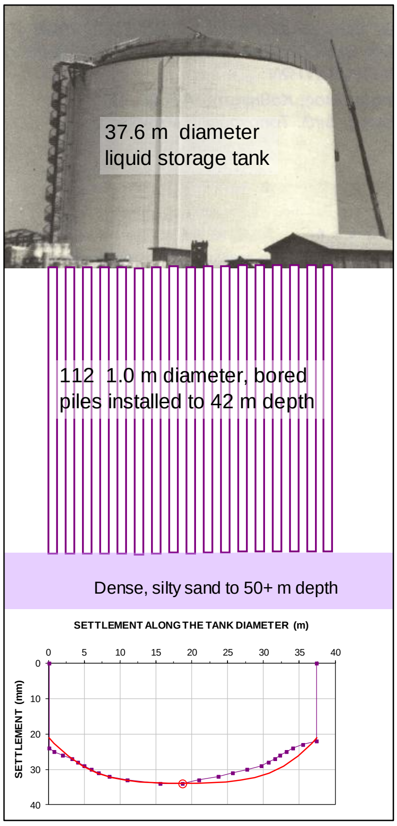

Fig. 11 Layout of the piles and measured and calculated settlements for the Thessaloniki tank along the tank diameter. (Data from Badellas et al. 1988)

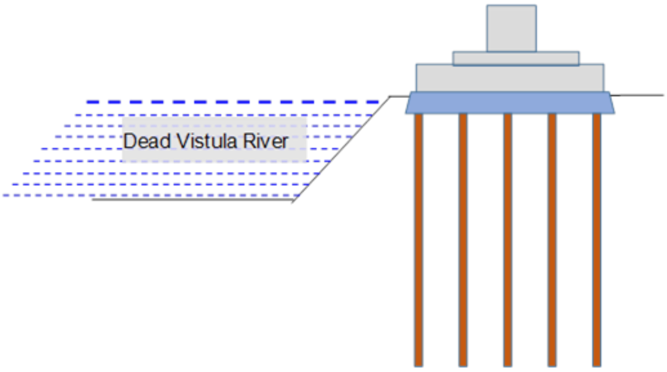

Fig. 12. Vertical section and geometry of the pier.

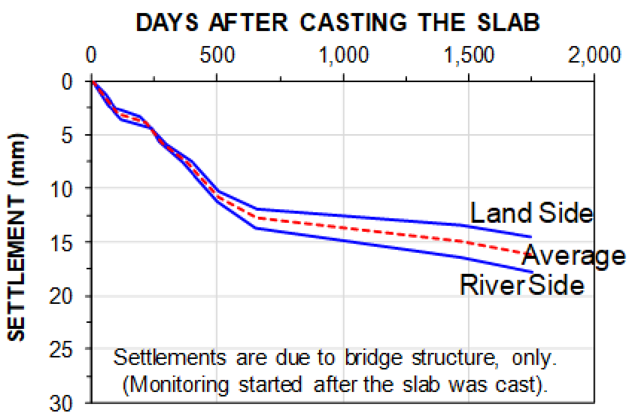

Fig. 13. Measured settlements (data from Gwizdala and Kesik 2015).

A pile group comprises perimeter piles and interior piles. Most piled foundations have more perimeter piles than interior piles. For example, to include an equal number of perimeter and interior piles, a rectangular pile group has to have 48 piles; 6 rows and 8 columns. Of the perimeter piles, the corner pile has the maximum exposure to the shaft resistance development. Therefore, it can be expected that the corner pile will, at first, take on larger load from the superstructure. Then, in the long-term, as the surrounding soil consolidates and settles, negative skin friction will reduce the shaft resistance and make the perimeter piles appear to become softer, thus, some of the load will be transferred to the interior piles.

Mandolini et al. (2005) and Russo and Viggiani (1995) reported measurements of the response of a wide pile group comprising 144 driven 406 mm diameter pipe piles supporting a bridge abutment. The pile axial load applied to 35 piles was monitored as the bridge was constructed and over about 10 months afterward. Figure 14 shows that interior piles under the raft carried 60% of the load carried by perimeter piles after the construction of the bridge had been completed. Then, because the perimeter piles were affected by general subsidence, adding drag force, pile compression increased. The rigid bridge cap ensured that some on the load on the perimeter piles, notably the corner pile, was transferred to the interior piles.

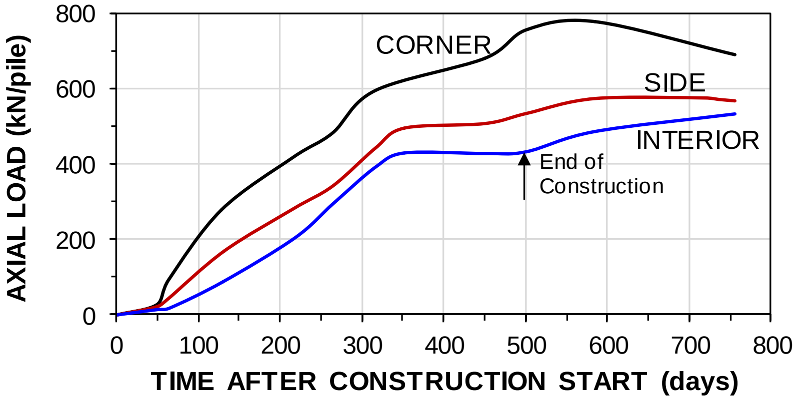

Fig. 14. Measured axial load during and after construction (from Russo and Viggiani 1995)

Cooke et al. (1981) reported similar results from monitoring a 43 m long, 20 m wide piled foundation comprising 450-mm bored piles constructed at 1.6 m c/c. The building was designed with perimeter and interior walls as bearing walls. Figure 15 shows the average pile-head loads monitored during and after the construction for eight instrumented piles: perimeter piles at corner and side piles, and interior piles. The load at both the head and toe was measured with a 360 mm wide, 100 mm thick cell cast in the pile that measured force using eight cylinders between two steel plates with vibrating-wire gages for determining imposed strain that was calibrated to force. The difference to the 400 kN/pile monitored total average load implied a total contact load of 14 MN, correlating to 16 kPa contact stress. Assuming a pile E-modulus of 30-MPa, the $E_{pile}A_{pile}$ parameter becomes 5 GN correlating the stress to 80-με axial pile strain. The soil underneath the raft, engineered fill or natural London clay, likely experienced the same 80-με strain and the 16 kPa stress correlates to E_{soil} = 250 MPa, which is a realistic value both for an engineered fill and the overconsolidated London clay. In contrast to the case of Okabe (1977), no general subsidence affected the piles.

Hansbo (1984; 1993) and Hansbo and Jendeby (1998) reported a remarkable case history of long-term response of two four-storey buildings in Göteborg, Sweden, supported on 300 mm diameter piles, driven to 28 m depth in a thick deposit of soft clay. One building was constructed on a grillage of beams (contact area was not reported) and the other on a raft. The buildings were on opposite sides of a street. The nominal total average load over each building footprint corresponded to 66 kPa and 60 kPa, respectively—very similar values. The as-designed average axial working loads were 220 kN and 520 kN/pile, respectively—quite different values. The conservatively estimated pile "capacity is stated to have been 330 kN, smaller than the sustained load per pile supported by Building 2. At the end of construction, measured pile loads were about 150 and 300 kN/pile, respectively, again, values quite different from each other. As shown in Figure 16, the measurements over a 13 year period showed that the buildings settled on average a very similar amount, about 40 mm,. The equivalent-pier shortening was smaller for Building 1, reflecting its smaller average pile load (and larger pier EA-parameter), but, because of the average stress across the footprint being larger, this difference was compensated by the larger footprint stress resulting in larger settlement below the pile toe level.

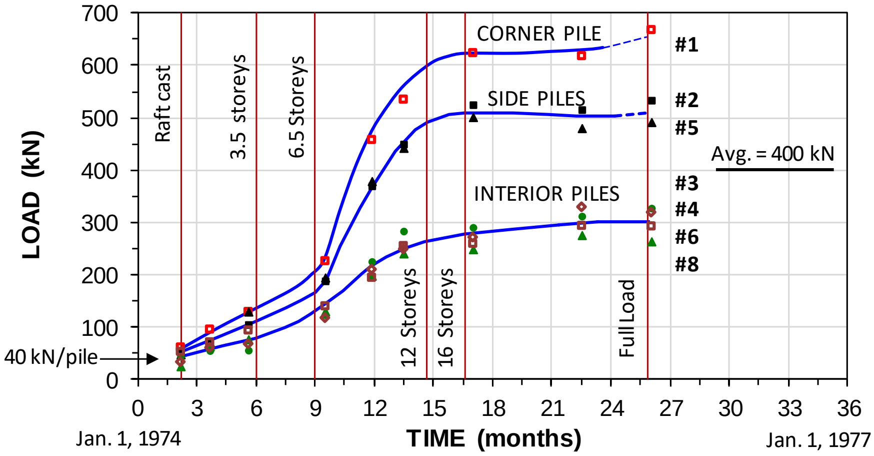

Fig. 15 Load at head of monitored piles over two years

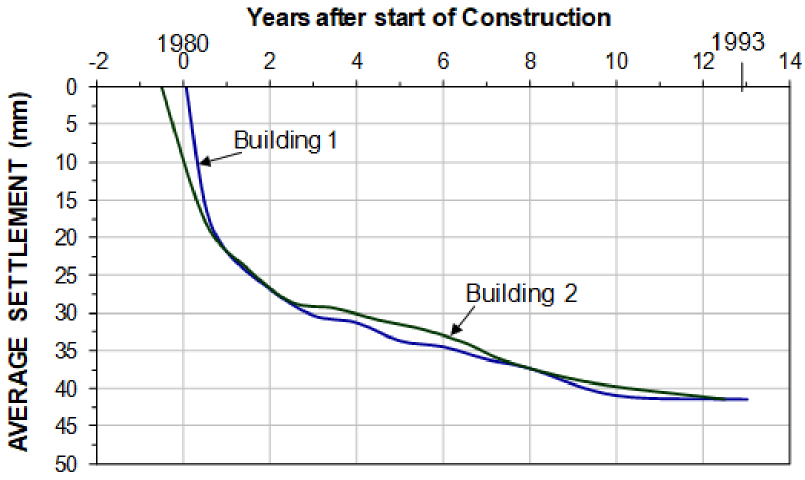

Fig. 16. Settlement measured for the two buildings over 13 years

Fellenius (2019) compiled settlement for storage tanks records of hydro testing of oil tanks 33,000 m^3 in volume, 19 m tall oil tanks, each place on a piled raft supported on 422 piles (case history by van Impe et al. 2018). The piles were 460-mm diameter, 21.6 m long screw piles (Omega piles). The average pile load was estimated to 780 kN (about 200 kPa average across the tank) and a static loading test to 3,500 kN maximum load showed no sign of failure. Tank 1 was tested first to measured 180 kPa, which caused a 20 mm, quite uniform, settlement along the tank perimeter as indicated in Figure 17. The water was then pumped over to Tank 2 resulting in measured 184 kPa stress. The settlement ranged from 15 through 20 mm with the smallest value along the side nearest Tank 1 (at Gages #10 to #11) and the largest diametrically opposite. It is obvious that the load on Tank 1 preloaded the soil under Tank 2 and caused some initial settlement, which, unfortunately, was not measured (the purpose of the tests was to the check the tanks, not to monitor settlement). Analysis of the measured settlements of the two tanks applying an equivalent raft at the pile toe depth and using the same parameter input for both analyses fitted both settlement records.

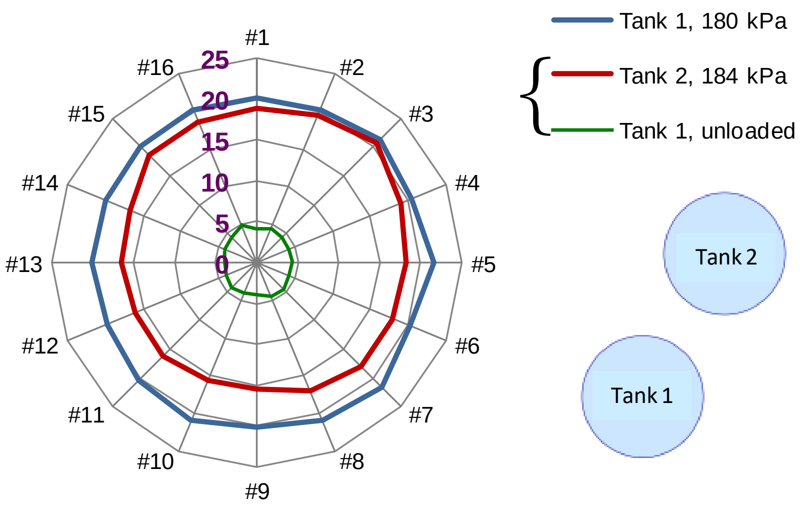

Fig. 17. Measured perimeter settlements for Tanks 1 and 2 and remaining settlement of Tank 1 after unloading. (Data from van Impe et al. 2018).

Buttling et al. (2024) reported result of nine years of monitoring a 3.0 to 3.5 m thick pile raft foundation for pile loads and settlement supporting a 200 m tall tower in Bangkok. The piles were 1,000 mm diameter, 4.5 m long bored piles. After end of construction, the load on the perimeter piles was larger loads than on the interior piles. Figure 18 shows the measured settlement across the 36 m wide raft. The load-movement during the construction was about 100 mm and, due to the general subsidence at depths below the pile-toe level, the post-construction settlement over the nine years of monitoring amounted to about 40 mm. The differential settlement across the raft half-width at the end of construction was 15 mm. Because of ongoing general subsidence, the perimeter piles were affected by downdrag and, therefore, some of the loads on these piles was transferred to the interior piles, thus, as shown in the figure, not only reducing the differential settlement, but reversing the raft from showing a sagged shape (bowl) at end of construction to become hogging some six years later.

The technical literature includes very few full-scale case histories on pile group response, but the foregoing survey is not exhaustive. For example, Inoue et al. (1977) presented an additional and equally educational case history of settlement due to downdrag. Kakurai et al. (1987) reported measured long-term distributions of axial force.

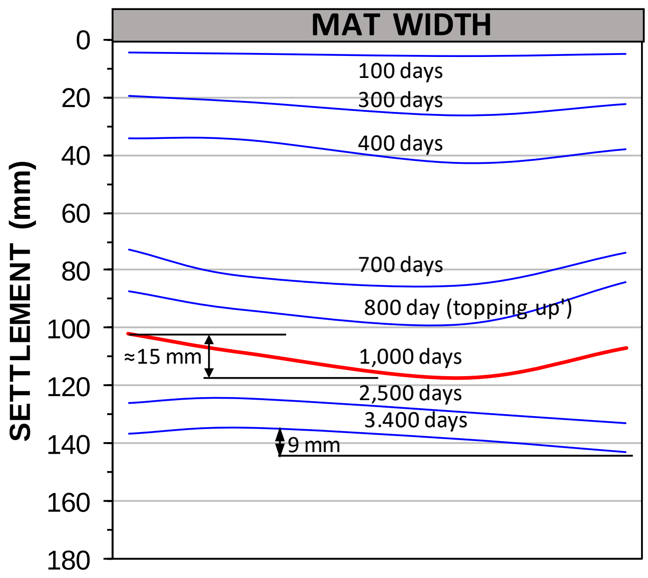

Fig. 18. Settlement measured for the tower raft over 3,400 days—9 years

CLOSURE

Designing a piled foundation based on the concept of "capacity" is fraught with much uncertainty. A more reliable design approach is to instead emphasize settlement and apply the Unified Method (Fellenius, 1984, 1988, 2025), which recognizes the particulars of single piles and of interior and perimeter piles in pile groups, and whether the groups are narrow or wide. The settlement analysis is particularly important for piles at sites experiencing subsiding soil.

Before engaging in analysis, compile all soil information (geology and mineralogy, densities, layering compressibility, GW and pore pressure distribution (also well below the expected pile toe level), and pile data. Information on past and planned excavations, fills and loads is vital.

The design will start with back-calculating the response of a pile subjected to a static loading test: the t-z/q-z functions and force distribution. Then, it involves determining the response of a single foundation-pile subjected to initial loading, long-term loading, general subsidence, etc. For a pile group, also the distribution of applied load across the raft and between piles, and the pile group settlement, total and differential must be assessed.

If a change in the distribution of effective stress around foundation piles is expected to occur during the life of a structure supported on a pile foundation, e.g., basement excavation and/or fills, this must be included in the analysis. Therefore, analysis using stress-independent shear parameters (i.e., "alpha-method"; total stress analysis; undrained shear strength; "ultimate shaft resistance") have limited use and the analysis is best performed with effective stress parameters. For example, the results of a back-analysis of a static loading test executed from the ground surface with the shaft isolated from the soil within a future excavation depth for basements will considerably overestimate the long-term conditions for the piled foundation, if the analysis is made with stress-independent parameters.

The Unified Method analysis can be carried out manually. However, for other than a very simple case, it would, even when aided by a spread sheet, involve much time and require unrepresentative short-cuts and simplifications. Software exists that will save time and allow for input of all case data, e.g., soil parameters, t-z/q-z functions, pore pressure distribution, geometry, pile type (size, length, loads, properties), area loads, fills, excavations, etc.. The output needed for design assessment comprises distributions of effective and total stress, distributions of axial force and settlement due to sustained loads and general subsidence, simulation of a static loading test, etc. The software must enable quick calculation of effect of changes in soil layering, type of pile, loading, etc. Personally, I use the software pair UniSettle and UniPile by www.unisoftGS.com. All case analyses presented in this paper have been produced with this software pair.

The one thing that a structure cares about is that the settlement not be larger than it can accept. To assure this, a design needs to be based on estimated settlement of the piled foundation related to the settlement that would bring about distress. First, such approach requires collaboration between the structural and the geotechnical engineers, Second, the geotechnical engineer should correlate forces and movement (t-z/q-z relations) and pile compression with settlement of the soils due not just an applied load but also due to other factors, such as fills, load units, excavation, groundwater lowering, etc.

Currently, the EuroCode's approach differs from the North-American approach. I strongly believe it would be advantageous for the Users of the EuroCode, if the code would be edited to address design of piled foundations in a manner closer to the unified method.

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