IPA News Letter
Volume 6, Issue 2 June 2021

Special Contribution

Long-term response of piled foundations to sustained load

Bengt H. Fellenius, Dr.Tech. P.Eng.

Consulting Engineer, Sidney, B.C., Canada

1. INTRODUCTION

I am always glad to have an opportunity to present one of my favorite topics, i.e., piled foundations. Particularly now to the IPA readership because I have benefitted so much from Japanese research as presented in milestone case history papers such as by Endo et al. (1969), Inoue et al. (1977), Okabe (1977), Kakurai et al. (1987), Kusakabe et al. (1992), and Yamashita et al. (2011, 2012, 2013), emphasizing full-scale observations on response of actual foundations. Few other countries have given this topic as much thought and provided as much insight.

Piled foundations are usually employed to transfer load through a soft and compressible soil to a competent soil at some depth, combining shaft and toe resistances. When the design relies on shaft-bearing, the piles are invariably long. Whether the project comprises single piles or pile groups, the geotechnical design practice usually aims to ensure a safety against failure of the single pile (ULS design), employing a variety of definitions of what constitutes failure or "capacity". The potential for long-term settlement is rarely considered, however. It seems to be that if the safety factor (or resistance factor) is adequate, then, there is no settlement issue, which, while often true, can be very painful when not.

Normally, the soft soil surrounding the piles will compress over time and settle, developing downdrag that could result in an unacceptable increase the settlement of a piled foundation. Practice and many uninformed codes, attempt to resolve this issue by estimating the associated drag force that is then included as a load together with the sustained load from the structure—very costly approach and yet not always safe.

A pile group can be made up of many single piles, or bents of two or three piles spaced at large distances in terms of pile diameter, say, larger than 15 diameters. At that spacing, there will be minimal interaction between the piles. A group with a common raft supported on such widely spaced piles is rare.

Piled foundations can be supported on single piles and groups of piles, small (narrow) or wide. Single piles or small bent of a few piles can be considered to have no or minimal mutual interference, whereas piles in larger groups do interact. The response of narrow groups is dominated by the response of the perimeter piles, acting similarly to single piles. The response of wide groups is dominated by the interior piles and the raft rigidity. Both narrow and wide groups are also affected by the response of the soil below the pile toe level.

This article addresses the issues for single pile, narrow pile groups, and wide pile groups in terms of settlement. Due to limits of space, bearing issues are not included. However, bearing is the less important issue for a design, because if the settlement is adequate, bearing is usually also well at hand. The opposite is not always true, however.

2. FOUNDATIONS ON A SINGLE PILE

A settlement analysis of a piled foundation begins with determining the distribution of axial force in the pile after applying a load, the sustained (dead) load, making use of the available information on the piles, the soil, and, as is often needed, results of an instrumented static loading test. When the load from the supported structure is placed on the pile, the pile moves down a small distance, which movement generates positive shaft resistance and, eventually, also pile-toe resistance. Functions describing the shaft and toe resistances vs. movement are called t-z and q-z functions, respectively. Fig. 1 shows typical such functions. The functions can have 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 a common point, "Target Point". The principles of the functions is that a single function coefficient determines the shape of the selected function before and after the Target Point (Fellenius 2021).

The t-z functions can range from strain-softening to strain-hardening. Usually, the shaft resistance response is assumed to be elastic- plastic similar to the vander Veen function. Often the response is hyperbolic (Vijayvergiya; Chin-Kondner), i.e., initially steep and then gently increasing and sometimes, the shaft resistance reduces (softens) with increasing movement Hansen; Zhang). The q-z function, is almost always strain-hardening and most often best simulated as a

Gwizdala function. The pile toe is not engaged until all elements have mobilized significant shaft resistance. Once pile toe element is mobilized, all or most additionally applied load is conveyed to the pile toe.

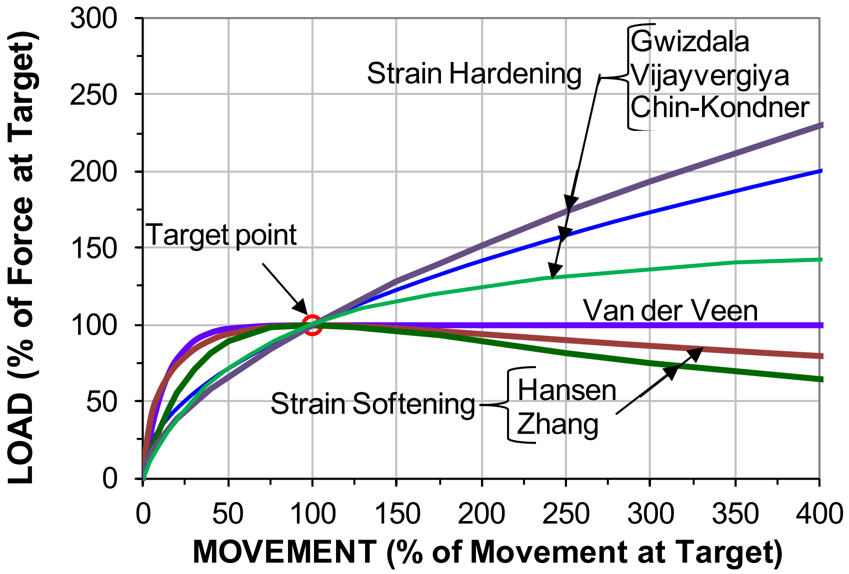


Fig. 1. Typical t-z and q-z function curves

Fig. 2A shows distributions of axial force for a series of applied loads to 30 m long pile installed in a two-layer soil deposit (Fellenius 2021). 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—neutral plane (first observed in full-scale tests by Johannessen and Bjerrum 1965 and Endo et al. 1969). A horizontal line is drawn from each intersection.

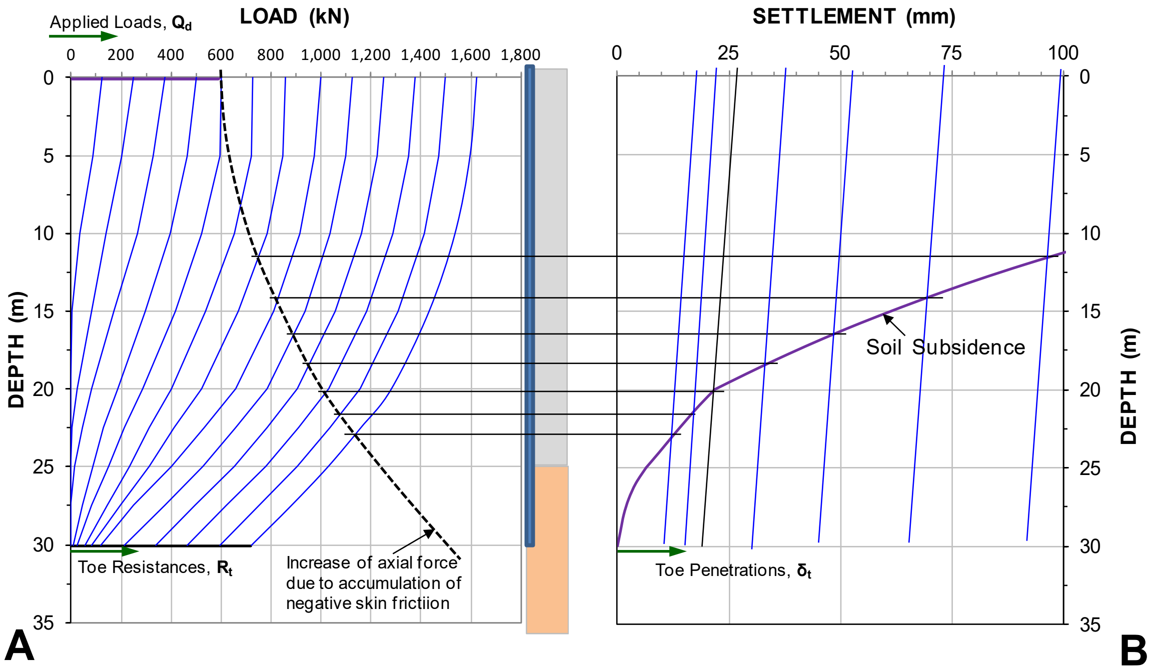


Fig. 2. Distributions of pile-force, pile-settlement, and soil subsidence

Fig. 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 force equilibrium can only occur 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 and it is, therefore, the settlement equilibrium. The figure shows an infinite number of force and settlement equilibriums. Only one of these equilibriums is true and it is that for which the toe force in Fig. 2A is the toe force that results in the pile-toe movement shown in Fig. 2B, that is, the q-z relation is satisfied. The depth for which this is satisfied is called the Neutral Plane.

Fig. 3 uses a graphical procedure to illustrate the principles of the unified design analysis according to Fellenius (1984; 1988; 2016; 2021). The method is easily computerized, e.g., UniPile by Goudreault and Fellenius (2014). The graphical process is to, first, in a diagram of force versus depth (Fig. 3A), draw a force distribution curve downward from the pile head, starting with the applied sustained load and increasing 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 curve is 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 neutral plane. Third, the soil subsidence is plotted in a diagram of settlement versus depth (Fig. 3B). The condition for a potential neutral plane to be correct is that the "Loop" shown in Fig. 3, satisfies the q-z relation for the pile. 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_d. The drag force is of no concern for the response of the piled foundation and 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 is not shown in the figure. N.B., the unified method required that all forces are unfactored.

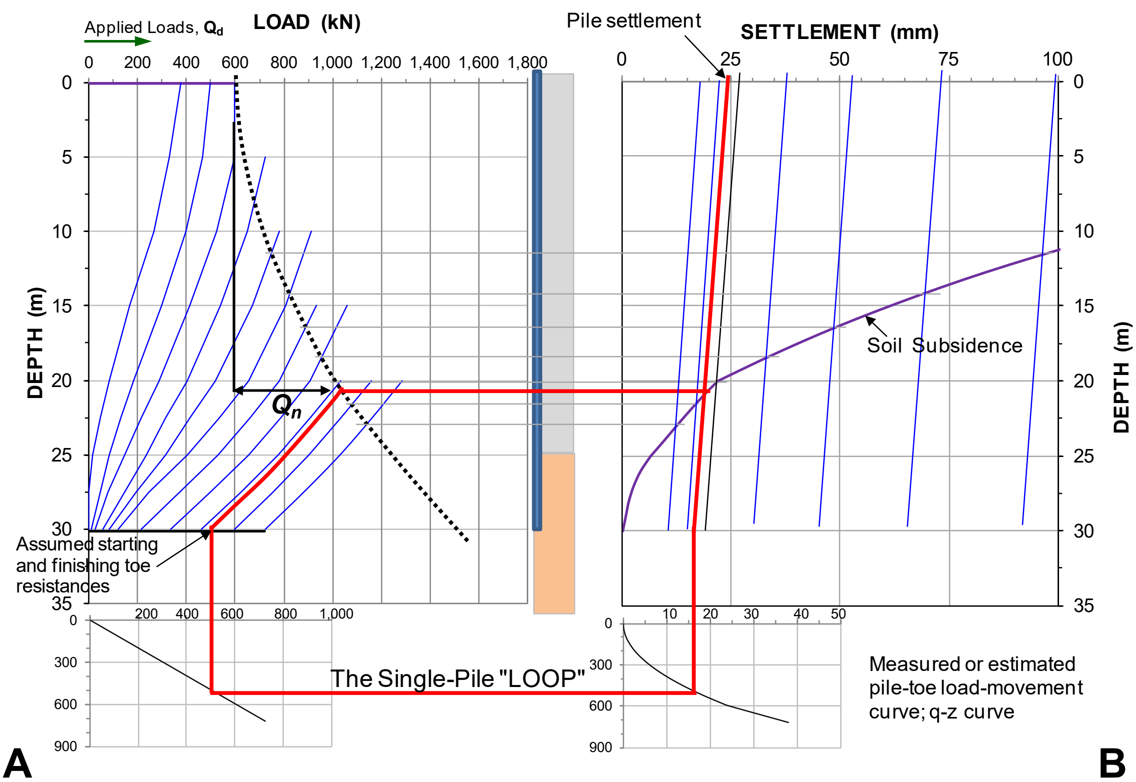


Fig. 3. Loop" determining the depth to the neutral plane and the downdrag

3. NARROW PILE GROUP

As load is applied to a piled foundation on a narrow pile group, settlement will first develop as the load is transferred to the soil. This "load-transfer movement" comprises pile axial compression and pile toe movement if a part of load reaches the pile toe. The pile toe movement is governed by the particular q-z relation for the soil at the pile toe level. Additional settlement will be caused by the soil compression (consolidation) due to the increased stress below the pile toe level. To calculate the settlement of a pile group, Terzaghi and Peck (1948; 1967) proposed that the load carried by a pile raft be assumed transferred to the soil through an equivalent raft of the same size as the raft, loaded by the same stress, and located at the lower third-point of the pile embedment depth. The settlement calculation for the equivalent raft would then be per conventional methods for settlement analysis. Actually, because the applied load starts to be distributed to the soil at the neutral plane, rather than being located at the lower third-point, the equivalent raft should be at the neutral plane. (In the long-term, a neutral plane will always develop). However, as originally proposed, the method disregards the fact that the piles enhance the compressibility of the soil between the equivalent footing and the pile toe level (the piles and soil act as a pier with a combined stiffness), which greatly reduces the settlement in this zone. Therefore, the design can just as well place the equivalent raft at the pile toe level.

The Terzaghi-Peck method usually results in settlement values that greatly overestimate the actual values. For pile groups comprising a small number of piles, no more than four rows, i.e., narrow groups, more realistic settlement values result from adjusting the equivalent raft to a larger width in recognition of the load shedding due to the shaft resistance between the neutral plane and the pile toe level. Fellenius (2021) proposed that the equivalent raft at the pile toe be widened by lines sloping 5(V):(1(H) from the neutral plane as indicated in Fig. 4.

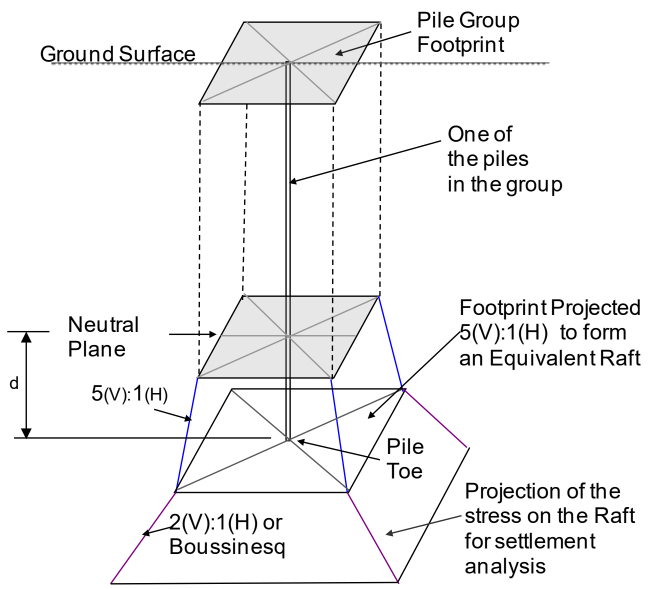


Fig. 4. Widening of the equivalent raft for a narrow group of piles

Note, that the compressibility (stiffness) of the pile-soil pier needs to be proportioned between the pile and soil E moduli and respective areas to that of an AE_{pier}. Moreover, the stress changes due to fill, adjacent foundations, and/or changes of pore pressure must be included in the settlement calculations.

4. WIDE PILED FOUNDATIONS

4.1 Contact Stress

It is common to design a pile group as equal to the same number of single piles with the average bearing reduced by an "efficiency coefficient", smaller than unity, defined as the ratio of the group bearing to the sum of the bearing of the individual piles. This approach originates in the fact that a group of piles will sometimes induce appreciable settlement in the soil below the pile toe level while single piles do not. Thus, at equal load per pile, the group will settle more than the singe pile. The "group efficiency" approach attempts to adjust to this fact. Yet, the bearing of a single pile within the group is about the same as that of a single pile outside the group. Moreover, the "group efficiency" approach only applies to small and narrow groups. For large and wide groups, a group bearing concept is not realistic. The response of a pile group to an applied load, be it small or large, narrow or wide, has to be analyzed in terms of settlement.

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, is according the principle of strain compatibility per the E moduli of the piles and soil and the areas of pile and soil. 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. Groups comprising closely spaced piles will have smaller total contact load as opposed to groups with widely spaced piles. This is because the free soil area, A_{soil}, is smaller where piles are closely spaced and the load E_{soil} A_{soil} ϵ will be large in relation to the total load carried by the piles, E_{pile} A_{pile} ϵ (Auxilia 2009, Yamashita 2011; 2012; 2013).

Moreover, while the E-modulus of the concrete 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) can be large when the soil is here engineered backfill with an E-modulus much larger than the natural soil. If spacing is wide, the free soil area, A_{soil}, will be large, and, consequently, the contact load will be large. Down in a soft soil layer with a small E_{soil}, 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 on whether the total area of the piled foundation and the total area of the piles, the Footprint Ratio, is small or large.

4.2 Perimeter versus Interior Piles

Hansbo (1984; 1993) and Hansbo and Jendeby (1998) reported a case history of long-term response of two adjacent 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 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. At the end of construction, measured pile loads were about 150 and 300 kN/pile, respectively, again, quite different values. As shown in Fig. 5, the measurements also showed that the buildings settled on average a very similar amount, about 40 mm, over a 13 year period. The equivalent-pier shortening was smaller for Building 1, reflecting its smaller average pile load (and, larger pier stiffness, EA/L), but because of its larger average stress across the footprint, this difference was compensated by the settlement below the pile toe level being larger.

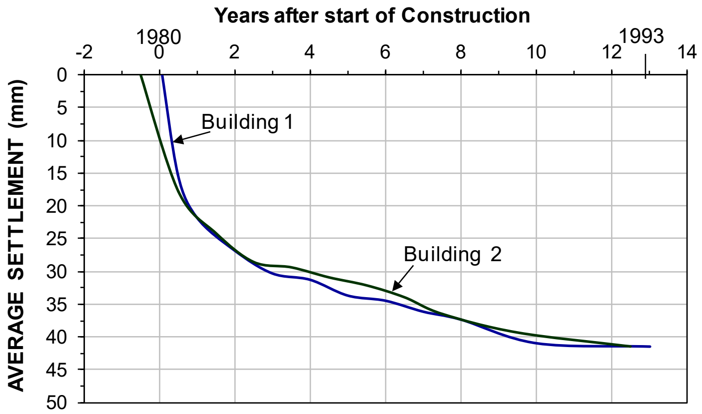


Fig. 5. Settlement measured for the two buildings over 13 years

The analogy between a pile group and a concrete column is only applicable to interior piles in a group, not to perimeter piles (side piles or corner piles) because the perimeter piles are surrounded by soil, whereas the concrete column is free standing. An interior pile is a pile having at least one row or column of piles between itself and the raft perimeter.

The strain compatibility principle, i.e., that the strain is the same in the pile and 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, the perimeter piles will shed load due to shaft resistance. 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).

Thus, the response of a wide piled foundation, differs from that of a single pile or a narrow piled foundation. The response of a perimeter pile, i.e., the outermost row—sometimes, also the next row—is similar to that of a single pile or a pile in a narrow group. However, for interior piles, the response is quite different.

The mentioned difference was first observed by Okabe (1977) when measuring axial force distribution in interior and perimeter piles over 4.5 years and comparing the records to that of a single pile nearby (Fig. 6). The site was subjected to general subsidence due to water mining that developed negative skin friction 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 the single pile. However, the interior piles were neither affected by negative skin friction nor by positive shaft resistance.

As first stated by Franke (1991), when load is applied to a group of piles, the shaft resistance on interior piles is not mobilized the way it is in a single pile, from the head downward, but from the toe to upward. The statement means that for an interior pile, in contrast to a perimeter pile, the transfer of the load applied to the raft to the soil is unaffected by shaft resistance. 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; in case of a rigid raft, the perimeter piles will receive a larger portion of the sustained load as opposed to the interior piles. The following analysis illustrates the concept, which is independent of the bending rigidity of the raft, i.e., it applies to both flexible and rigid rafts.

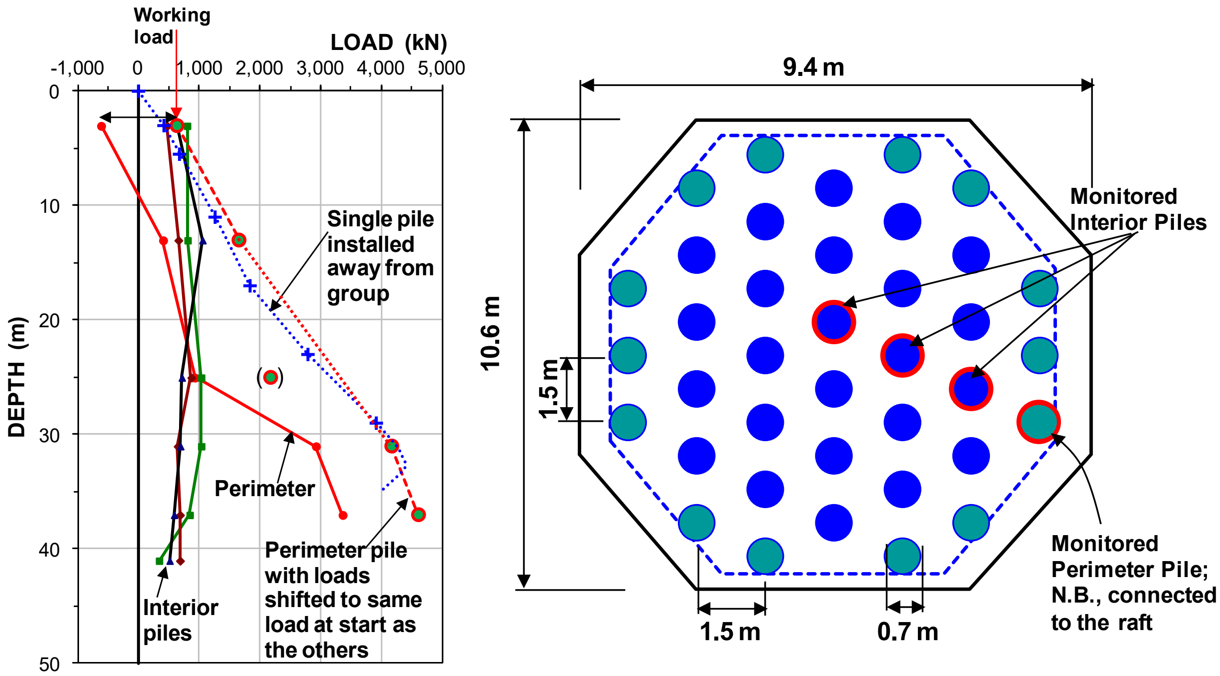


Fig. 6. Axial pile loads measured 4.5 years after construction (Okabe 1977)

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 distance the pile toe moves into the soil is equal to a soil compression—largest at the pile toe level (equal to the pile toe penetration) and diminishing upward along the pile. The process generates shaft resistance in the zone immediately above the pile toe, which reduces the force reaching the pile toe. The pile toe response depends on the pile toe soil stiffness (load-penetration relations, i.e., the particular q-z function).

The foregoing is the "Franke principle": the response of an interior pile follows the requirement that the toe penetration and force resulting from an applied load is coupled to the particular q-z function for the toe condition and t-z function for the shaft resistance immediately above the pile toe. Moreover, the movement between the pile and the soil diminishes over a distance up from the pile toe to where there is no more relative movement. A shaft resistance will develop along this length of pile, but above this length, or zone, there is no more shaft resistance along the interior pile.

The principles are 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. A small fill placed across the area outside the foundation footprint will result in about 25 mm long-term subsidence at the site. The applied unfactored sustained load is 800 kN/pile. The live load is 200 kN/pile. Fig. 7A illustrates the analysis procedure for determining where the toe movement for the toe force is equal to the upward movement (compression) of the soil in-between the piles. According to the Franke principle, the so-determined shaft resistance is 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 Fig. 7B with the bidirectional cell placed right at the pile toe (beyond about 500 kN, the downward curve is extrapolated).

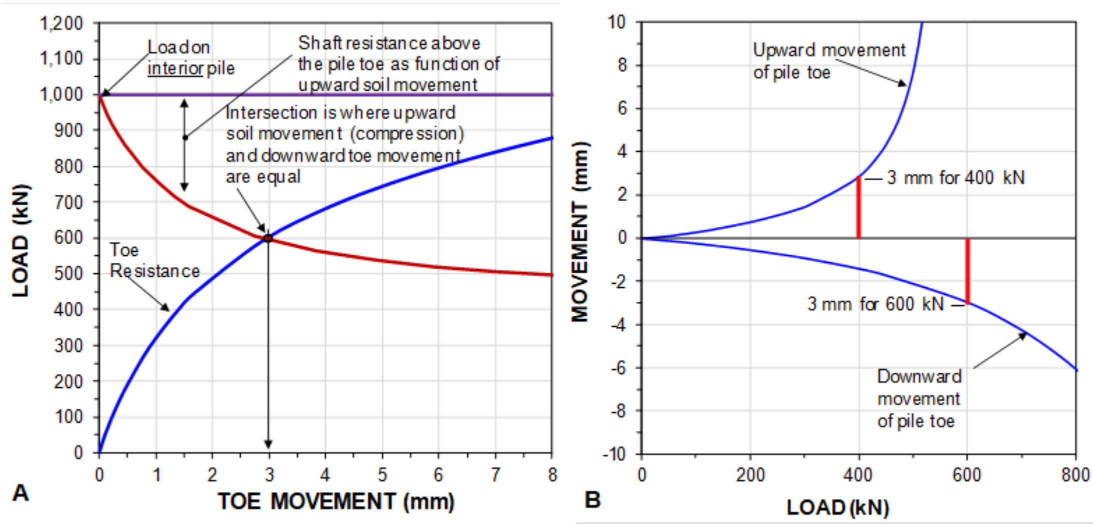


Fig. 7. The process for determining the load-transfer toe-movement and toe-force for interior piles

A raft can be either rigid or flexible (and anything in between). Fig. 8 shows 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, 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 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.

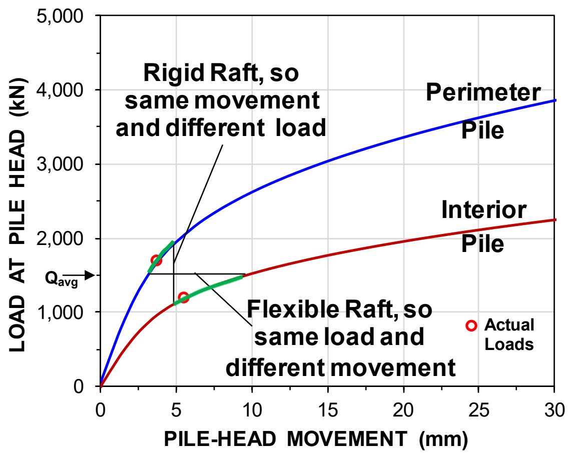


Fig. 8. Comparison of load distribution for a perimeter pile and an interior pile in a wide pile group.

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, the load will be transferred to the interior pile develop Fig. 9 shows full-scale measurements by Mandolini et al. (2005) and Russo and Viggiani (1995) illustrating the latter development.

4.3 Settlement below the pile toe level

Be the spacing or the contact stress large or small, the response of the piled raft is compression of the 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; 2021). The settlement of the soil below the pile toe level can be calculated as that of an equivalent raft at the pile toe level loaded by the same stress as applied to the foundation raft, as shown in Fig. 10. Note, the analysis must include the influence of stress changes due to other foundations, fills, excavations, and changes of groundwater table.

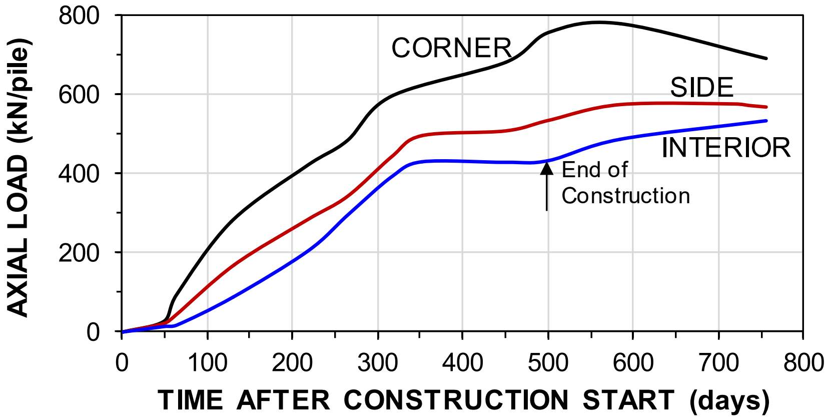


Fig. 9. Measured axial load during and after construction (data from Russo and Viggiani 1995)

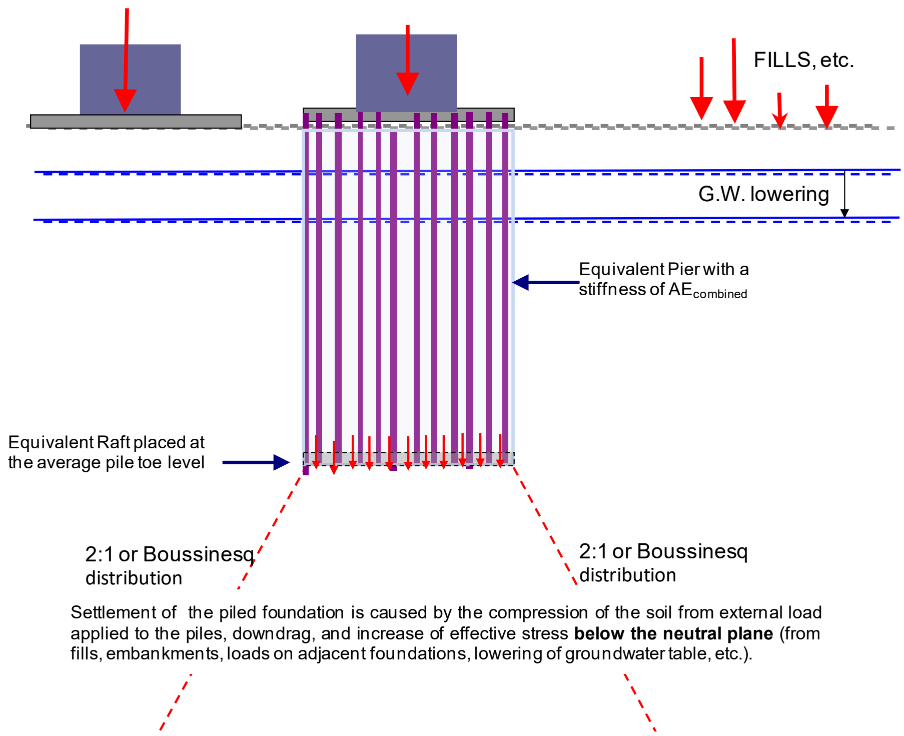


Fig. 10. The equivalent raft for calculation of settlement below the pile toe level

5. CLOSURE

Designing a piled foundation based on the concept of capacity is fraught with much uncertainty. The design should instead emphasize settlement. N.B., with due recognition of the difference in response between narrow and wide pile groups and between interior and perimeter piles. The settlement analysis is particularly important for piles in subsiding soil.

6. REFERENCES

Auxilia, G.B., Burke, P., Duranda, M., Ulini, F., Buffa, L. Terrioti, C., Dominijanni, A., and Manassero, M., 2009. Large storage capacity cement silos and clinker deposit on a near-shore sandy fill using piles for soil improvement and settlement reduction. 17th ICSMGE, Alexandria, October 5-9, 2009, Vol. 3, pp. 1181-1184.

Broms, B.B., 1976. Pile foundations—pile groups. 6th ECSMFE, Vienna, Vol. 2.1 pp. 103-132.

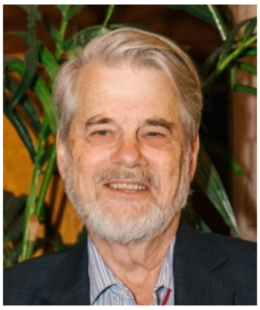
Endo M., Minou, A., Kawasaki T, and Shibata, T, 1969. Negative skin friction acting on steel piles in clay. Proc. 7th ICSMFE, Mexico City, August 25-29, Vol. 2, pp. 85- 92.

Fellenius, B.H., 1984. Negative skin friction and settlement of piles. Proc. of the Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18 p.

Fellenius, B.H., 1988. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75-82.

Fellenius, B.H., 2006. Results from long-term measurement in piles of drag load and downdrag. Canadian Geotechnical Journal 43(4) 409-430.
Fellenius, B.H., 2015. The response of a "plug" in an open-toe pipe pile. Geotechnical Engineering Journal of the SEAGS & AGSSEA 46(2) 82-86.
Fellenius, B.H., 2016. The unified design of piled foundations. The Sven Hansbo Lecture. Geotechnics for Sustainable Infrastructure Development – Geotec Hanoi 2016, edited by Phung Duc Long, Hanoi, November 23-25, pp. 3-28.
Fellenius, B.H., 2019. Observations and analysis of wide piled foundations. Canadian Geotechnical Journal, 56(3), 378-397. doi.org/10.1139/cgj-2018-0031.
Fellenius, B.H., 2021. Basics of foundation design—a textbook. Electronic Edition, www.Fellenius.net, 534 p.
Franke E., 1991. Measurements beneath piled rafts. International Conference on Deep Foundations, Ecole National des Ponts et Chaussees, Paris, March 19-21, pp. 599-626.
Goudreault, P.A. and Fellenius, B.H., 2014. UniPile Version 5, User and Examples Manual. UniSoft Geotechnical Solutions Ltd. [www.UniSoftLtd.com]. 120 p.
Hansbo, S. 1984. Foundations on creep piles in soft clays. First International Conference on Case Histories in Geotechnical Engineering, St. Louis, May 6-11, 1984, pp. 259-264.
Hansbo, S., 1993. Interaction problems related to the installation of pile groups. Proceedings of the 2nd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles, Ghent, 1–4 June, 1993, pp. 119–130.
Hansbo, S. and Jendeby, L., 1998. A follow-up of two different foundation principles. Foundations on friction creep piles in soft clays. International Conference on Case Histories in Geotechnical Engineering. St. Louis, March 9-12, 259-264.
Johannessen, I.J. and Bjerrum, L. 1965. Measurements of the compression of a steel pile to rock due to settlement of the surrounding clay. Proc. 6th ICSMFE, Montreal, September 8 15, Vol. 6, pp. 261-264.
Inoue, Y., Tamaoki, K., Ogai, T., 1977. Settlement of building due to pile downdrag. Proc. 9th ICSMFE, Tokyo, Vol. 1, pp. 561-564.
Kakurai, M., Yamashita, K., and Tomono, M., 1987. Settlement behavior of piled raft foundations on soft ground. Proceedings of the 8th Asian Regional Conf. on SMFE ARCSMFE, Kyoto, 20 -24 July 1987, Vol. 1. pp. 373 -376.
Kusakabe, O., Maeda, Y., and Ohuchi, M, 1992. Large-scale loading tests of shallow footings in pneumatic caisson. ASCE Journal of Geotechnical Engineering, 118(11) pp.1681-1695.
Mandolini, A., Russo, G. and Viggiani, C. (2005). Pile foundations: experimental investigations, analysis, and design. Proc. 16th ICSMGE, September 12 -16, Osaka, Japan, pp. 177-213.
Okabe, T., 1977. Large negative friction and friction-free piles methods. 9th ICSMFE, Tokyo, July 11-15, Vol. 1, pp. 679-682.
Russo, G. and Viggiani C. (1995). Long-term monitoring of a piled foundation. Fourth International Symposium on Field Measurements in Geomechanics, Bergamo, pp. 283-290.
Terzaghi, K. and Peck, R.B. (1948). Soil Mechanics in Engineering Practice. 1st Edition, John Wiley & Sons, New York, 566 p.
Terzaghi, K., and Peck, R.B. (1967). Soil Mechanics In Engineering Practice. 2nd edition, John Wiley & Sons Inc., New York, 549 p.
Yamashita, K. Hamada, J., Takeshi, Y., 2011. Field measurements on piled rafts with grid-form deep mixing walls on soft ground. Geotechnical Engineering Journal of the SEAG & AGSSEA, 42(2) 110.
Yamashita, K. Hamada, J., Onimaru, S., Higashino, M., 2012. Seismic behavior of a piled raft with ground improvement supporting a base-isolated building on soft ground in Tokyo. Soil and Foundations 52(5) pp.1000-1015.
Yamashita, K., Wakai, S., and Hamada, J. 2013. Large-scale piled raft with grid form deep mixing walls on soft ground. Proc. 18th ICSMGE, September 2 6, Paris, France, Vol. 3, pp. 2637-2640.

A brief CV of Dr. Bengt H. Fellenius



Fellenius obtained his M.Sc. (1962) and a Doctor of Technology (1972) from the Royal Institute of Technology in Stockholm. He has nearly 60 years of consulting engineering experience with foundation design for industrial plants, highway projects, and marine structures as well as special investigations and instrumented field tests. He was Professor of Civil Engineering at the University of Ottawa 1980-2000, where he taught and carried out research centered on site-improvement methods and analysis of single piles and wide pile groups in subsiding soil. He is an internationally active foundation engineering consultant and the author of some 400 technical papers addressing piled foundations, soil improvement, foundation settlement, codes and standards, and in-situ sounding. Dr. Fellenius has given lectures and courses to several universities and international conferences throughout North and South America, Europe, and South-East Asia. He currently lives in Sidney, BC, Canada.