DESIGN OF PILES AND PILE GROUPS

CONSIDERING CAPACITY, SETTLEMENT, AND NEGATIVE SKIN FRICTION

Bengt H. Fellenius, Dr.Tech., P.Eng.
Background Notes for Demo Example for UniPile at www.unisoftltd.com
Updated 1999

Introduction

Current practice for pile design varies and building codes differ between countries as well as within countries, indeed, even between individual engineering disciplines. The differences do not become apparent until the designer includes the effect of dragload and settlements**:** In reference to the structural strength of the pile, many do realize that the maximum load in a pile is the sum of dead load and dragload; live load does not add to the maximum load. Others, however, will consider the dragload as just another load and lump it in with the dead and live loads. In reference to the bearing capacity, some will even determine the factor of safety as the pile bearing capacity divided by the sum of the dead and live loads and the dragload! Or worse, as really has occurred by some twisted logic, first subtract the dragload from the bearing capacity and then divide the balance by the sum of the dead and live loads and the dragload. The correct approach is that the dragload is not to be included in the calculation factor of safety on bearing capacity. The factor of safety is simply the bearing capacity divided by the dead and live loads. Actually, the larger the dragload the stiffer the pile and the safer the design (provided the structural strength is sufficient).

Settlement calculations are rarely performed when designing a ordinary pile group. When it is, the methods of calculation range from those using simple rules of thumb to those incorporating detailed finite element analysis. While the design of pile capacity is often verified by full-scale field testing, design for settlement is almost totally without the benefit of full-scale verification. Worthwhile cases are scarce in the literature.

Survey of Methods Used in the Industry

In the Geotechnical News Magazine of June 1990, the author published an appeal for participation in a pile design survey. The design problem consisted of calculating the bearing capacity of a single pile in a pile group and the settlement of the pile group. Two types of piles were included, a 12.75-inch steel pipe pile and a 12-inch concrete pile. The survey case serves well as a demonstration of UniPile's capabilities. The following is abbreviated from the survey and is also limited to addressing the steel pipe pile.

The soil profile was made up of five layers. The soil profile was taken from a real case, but no SPT data or CPT data are included. The participants were asked to define their method-specific selection of soil strength parameters to use in the capacity calculations. (Basic soil information is presented in the table below).

The modulus numbers and stress exponents given in the table are those of the Janbu tangent modulus approach as detailed in the Canadian Foundation Engineering Manual (1985). Subscript "r" indicates recompression number. For Layers 2 and 4, also the conventional C_C and e_0 values are given, as determined directly from the modulus numbers by means of $C_C = 2.3\ (1+e_0)/m$. For Layers 1, 3, and 5, approximate E-moduli can be calculated from E = m times the square root of the mean effective stress.

Layer No.	Thickness m (ft)	Soil Type	Water Content (%)	Density kg.m^3 (pcf)	τ_u KPa(psf)
1	2.0 (6.6)	Silt and sand	16	2,150 (132)	--
2	8.0 (26.2)	Silty clay	27	1,950 (119)	80 (1,700)
3	5.0 (16.4)	Sandy silt	19	2,100 (129)	--
4	7.0 (23.0)	Firm clay	35	1,850 (113)	35 (700)
5	28.0 (91.9)	Coarse silt	24	2,000 (122)	--
--		Bedrock			

Layer No.	Modulus Numbers m and m_r	Stress Exponent j	OCR	Cc and C_{cr}	e_0
1	100 and --	0.5	1.0	--	0.43
2	40 and 350	0	1.4	0.10 and 1.72	0.72
3	180 and 900	0.5	2.0	--	0.51
4	20 and 200	0	2.3	0.22 and 0.02	0.93
5	100 and 700	0.5	2.7	--	0.64

Before construction, a perched groundwater table exists at a depth of 0.5 m (1.6 ft). The phreatic elevation determined in Layer 3 is at a depth of 3.0 m (9.8 ft). In Layer 5, the pore water pressure is artesian and the phreatic elevation lies 2.0 m (6.6 ft) above the ground surface. On completion of construction, the perched water table will be lowered to a depth of 2.0 m (6.6 ft). The phreatic elevations in Layers 3 and 5 will not change.

The pile group consists of 12 piles in three rows of 4 piles joined by a stiff cap of size 3.5 m by 5.0 m. The piles have been installed to a depth of 32.0 m (105 ft). The piles consist of 324 mm (12.75-inch) steel pipe piles with a 9.5 mm (0.375-inch) wall. The steel yield is 300 MPa (44 ksi). The piles were driven closed-toe and have been filled with 35 MPa (5,000 psi) concrete. These values represent steel and concrete cross sectional areas of 94 cm^2 (14.6 in^2) and 734 cm^2 (113.8 in^2). The steel and concrete E-moduli are 205 GPa (29,000 ksi) and 30 GPa (4,350 ksi), respectively, which values result in a combined modulus of 50 GPa (7,750 ksi).

The total load on the pile cap is 14.4 MN (3,240 kips) with a dead load portion of 12 MN (2,700 kips), that is, 1,200 KN (270 kips) and 1,000 KN (225 kips), respectively, per pile.

Coinciding with the pile installation, a 1.25 m (4.1 ft) thick engineered fill of density 2,200 kg/m^3 (135 pcf) is placed over a 50 by 30 m (165 by 100 ft) area with the pile group in its center. However, no fill is placed within a 9 by 7 m (30 by 20 ft) area immediately around the piles. An excavation is made to a depth of 2.0 m (6.6 ft) over a 5.5 by 4.5 m 18 by 15 ft) area around the pile group to accommodate the pile cap (the weight of the pile cap is included in the dead load).

The main purpose of the survey was to learn what method of static analysis the participants would use to determine the pile capacity (ultimate resistance) and settlement of the pile group. To this end, the following questions were asked.

1. What are the values of short-term and long-term capacities of a single pile? (The short-term capacity is capacity before backfill has been placed, but after all effects have dissipated of excavation, pile driving, and subsequent lowering of the perched water table. The long-term capacity is capacity after backfill has been placed and the soil has fully consolidated from the effects of the backfill).

2. How large is the drag load?

3. How large is the maximum load in the pile?

4. Would a reduction of the negative skin friction be desirable and, if so, by what means, how much, and over what length of pile?

5. What magnitude of settlement should be expected for the pile group (at cap level)?

6. What will the capacity be of the pile during the initial installation? This is the capacity to use in a wave equation analysis of the pile driving.

Some people wrote back with comments on the appeal**:** a US engineer, who had been invited to participate, declined because "*without N-values I do not feel I can determine the capacity of the piles*". An engineer in France said the same thing, but instead of the SPT N-values, he wanted pressuremeter data. To complete the dependency on tools, an engineer from Holland declined because Dutch cone (CPT) data were missing. Whatever happened to design by first principles and basic soil mechanics!

Some 35 answers were obtained from participants in several countries. On Question 1, the participants gave single pile capacity values that ranged from 1,100 KN through 6,000 KN, a very wide range, indeed. No serious attempt was provided to separate the capacity on the short and long-term conditions. A few participants did submit very thoughtful responses, which reflected the practices in their part of the world. However, other answers were so obviously wrong that publishing the answers would unavoidably point a critical finger at several participants. Therefore, no compilation of the answers was ever published.

On Questions 2, 3, and 4, very few gave answers. On Question 5, the values on expected pile group settlement values ranged from 2 mm through 160 mm.

No answer was received on Question 6.

Discussion and results of UniPile Calculations

In the following, a brief discussion on the design problem is offered. The UniPile program distribution disk and the Demo disk (demo.unp) contains the input data as given below where anyone can review the analysis in detail.

UniPile Input. The author's preferred method of analysis is the effective stress method known as the beta-method. It requires input of two coefficients, β and N_t. The literature (e. g., Fellenius, 1999) indicates a range of β-coefficients and N_t-coefficient based on the soil type. For the given soil profile, the suggested ranges and the author's chosen input values are, as follows.

Layer 1	Silt and Sand	0.27 - 0.60,	say 0.6;	low water content
Layer 2	Stiff silty clay	0.25 - 0.35,	say 0.35;	use of the high boundary seems proper
Layer 3	Sandy silt	0.27 - 0.50,	say 0.4;	about in the middle seems proper.
Layer 4	Firm clay	0.25 - 0.35,	say 0.3;	a one-decimal value often applied to clay
Layer 5	Coarse silt	0.27 - 0.50,	say 0.5;	use of the high boundary seems proper.

As to the N_t-coefficient, because a friction ratio of about 1 % would be expected in a CPT-test, had one been performed at the site, a ratio of 0.01 between β and N_t could be suitable, that is, an N_t-value of 50 is chosen.

Because the case is essentially fictitious, not too much thought needs to go into the selection of the above parameters. Instead of effective stress analysis (β-method), total stress analysis (α-method) could have been used for Layers 2 and 3, applying the undrained shear strength values given in the appeal. Even the lambda-method could have been applied to these layers, subject to the preference of the engineer.

UniPile Results. With the above suggested soil strength parameters as input, UniPile computes the short-term pile capacity, the dragload, and the maximum load for the final conditions to be 3,400 KN (760 kips), 1,200 KN (270 kips), and 2,200 KN (495 kips), respectively. The results are shown in Fig. 1 as load and resistance curves. The resistance curve goes from the ultimate resistance of 3,400 KN to the toe resistance value of 1,200 KN. The load curve starts from the dead load of 1,100 KN and increases by the negative skin friction down to the neutral plane at a depth of 24.3 m. Because nature abhors sudden changes — kinks — the transition from negative skin friction to positive shaft resistance occurs over a certain distance above and below the neutral plane. The distance can be a few or many pile diameters, smaller where soil settlements are small and larger where settlements are large. In this case, a transition zone of about 6 diameters is assumed. This results in a reduction of the calculated maximum load at the neutral plane from 2,200 KN to 2,000 KN, as indicated by the short vertical portion of the dashed curve.

Fig. 2 shows the distribution of toe, shaft, and total resistances with depth.

Considering the total load of 1,200 KN (270 kips), the pile has a short-term bearing capacity factor of safety of 2.8, which is normally considered adequate for a design based on static analysis.

The structural strength of the concreted pipe pile can be calculated from the data in the survey appeal, assigning factors to the steel and concrete strengths and proportioning a combined strength from the strengths of the respective areas. However, the so calculated strength is not adequate for use, as it does not consider the compatibility of the steel and concrete. The steel strength of 300 MPa (44,000 ksi) and the concrete strength of 35 MPa (5,000 ksi) are not compatible. At the limiting strain of 1.0 millistrain, the steel is well within its limit, but the concrete is overstressed. In this case, the 70 % rule takes over, that is, the concrete is only allowed to be stressed to about 70 % of its strength, which corresponds to a limiting maximum load in the pile at the neutral plane of 3,100 KN (440 kips). Still, this is more than the computed maximum load in the pile.

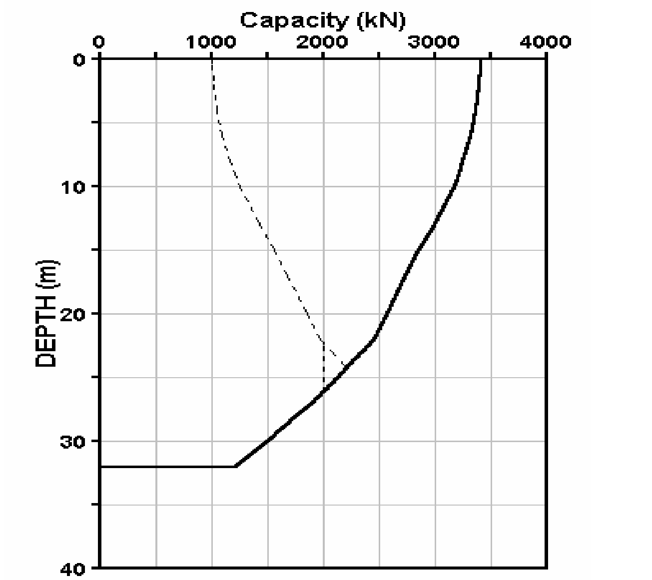


Fig. 1. Load and Resistance Curves

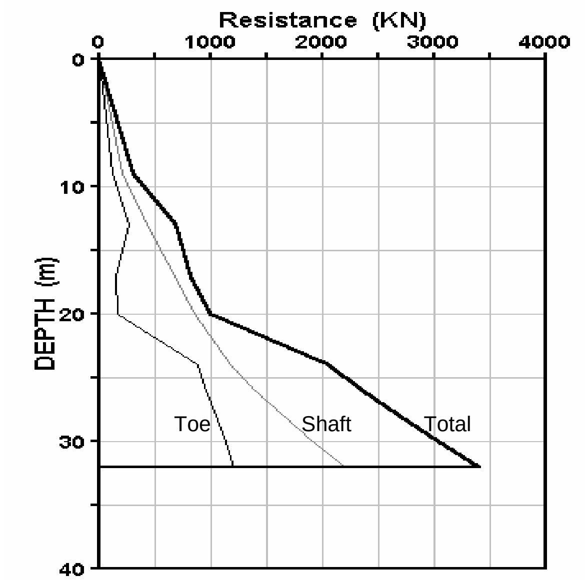


Fig. 2. Resistance versus Depth

The settlement of the pile group can be computed different ways. The data were chosen so that the combined effect of the placing of the fill, the lowering of the water table, and the excavation would result in only a very small increase of effective stress. That is, negligible settlement. The settlement of the pile group would only be caused by the load on the piles themselves. UniPile computes the settlement of a pile group as the settlement for an "equivalent footing", i.e., a footing having the same area as the pile cap with a uniform stress equal to the dead load divided by the cap area. The stress distribution is by the 2(V):1(H)-method. The user will have to decide where in the soil to place this equivalent footing. For a pile group "floating" in clay, Terzaghi and Peck (1948) suggested to use the lower third point. The lower third point for such a pile group is also where the neutral plane is located. The neutral plane is where the transition occurs between negative skin friction and positive shaft resistance and it is where the pile and the soil have no relative movement. Notice, downward net movement may occur. At the neutral plane, however, the movement for the pile and the soil are equal. It makes sense to always place the equivalent footing at the neutral plane. However, one must then consider that the soil underneath the equivalent footing is "reinforced" with the piles, or the calculated settlement based on the equivalent footing concept may become unrealistically large. For most cases, the author prefers to calculate the settlement for an equivalent footing placed at the pile toe.

For the subject example, and with an equivalent footing at the pile toe, UniPile determines the settlement of the pile group to 18 mm (3/4 inch), which is acceptable for most structures. Had the analysis been made for an equivalent footing placed at the neutral plane (the calculations show the neutral plane to be 7 m up from the pile toe) and included the reinforcing effect of the piles, the calculated settlement would have been about half this value.

The long-term capacity must consider the excavation, the fill, and the change of phreatic water table. A total stress analysis cannot handle this analysis, unless certain assumptions are made. As mentioned, the data were chosen so that the fill, the water table lowering, and the excavation would result in only a very small increase of effective stress. That is, the long-term capacity is about equal to the short-term.

WEAP Analysis

That the short and long-term capacities are about equal does not mean that these values are equal to the pile capacity during the pile driving (Question 6). The driving of the piles will induce large pore pressures, which will reduce the soil resistance to the driving. This reduced resistance is the capacity to use as input to a WEAP analysis. In the clay, for example, the pore water pressures can become almost as large as the total stress, which essentially eliminates the effective stress and any resistance to the pile advancement. In the coarse soil, the pore water pressures can about double due to the driving.

To calculate the resistance to the driving, one can change (increase) the pore water pressures to represent those induced during the initial driving. Pore pressures increase more in soft sensitive clay, less in silt, and marginally in sand. The pore pressure increase is less pronounced below the pile toe and the pore pressure method requires that the toe bearing capacity coefficients be increased. Not fully though, because the toe resistance is also affected by the driving. Alternatively, one can reduce the β-coefficients or the cohesion (alpha-method), reducing the resistance more in the soft sensitive clay, less in the silt, and marginally in the sand. The latter approach is easier to use, because it leaves the toe resistance intact and lets it be adjusted independently. However, the UniPile analysis will then require two separate soil data files. UniPile Version 4 allows the input of a constant unit toe resistance, which is independent of the effective stress at the pile toe and this option is sometimes easier to use.

For the survey case, the author suggests to adjust the β and N_t-coefficients, as shown below. As the case is fictitious, albeit realistic, the values are given to demonstrate design calculations without implying any general validity.

Layer 1	Silt and Sand	from $\beta = 0.6$ and $N_t = 10$	unchanged above the water table
Layer 2	Stiff silty clay	from $\beta = 0.35$ and $N_t = 12$	to $\beta = 0.10$ and $N_t = 8$
Layer 3	Sandy silt	from $\beta = 0.4$ and $N_t = 20$	to $\beta = 0.15$ and $N_t = 15$
Layer 4	Firm clay	from $\beta = 0.3$ and $N_t = 10$	to $\beta = 0.12$ and $N_t = 8$
Layer 5	Coarse silt	from $\beta = 0.5$ and $N_t = 50$	to $\beta = 0.30$ and $N_t = 40$

A UniPile calculation for the WEAP input shows that the capacity against which the hammer has to advance the pile at the full depth is just about 60 % of the final, only 2,100 KN (470 kips) as opposed to the capacity of 3,400 KN (760 kips) after set-up.

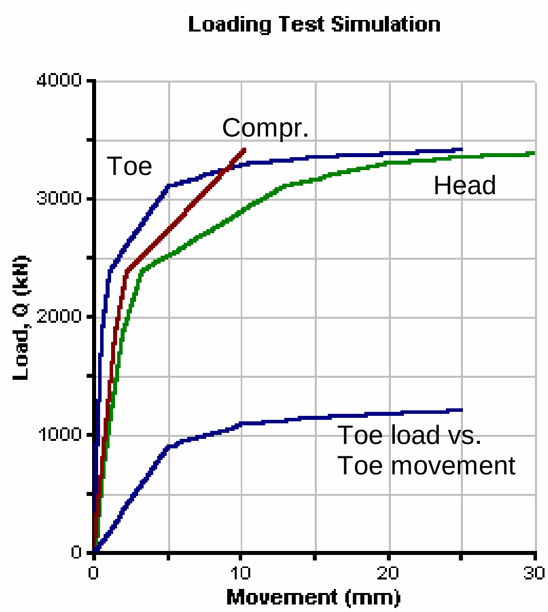
The UniPile calculation results for WEAP input are shown in the following table in a format ready to be input to the WEAP program. The shaft resistance is given both as the resistance per unit length of pile and as unit shaft resistance. This is because different versions of WEAP use either one or the other value as input. An output of WEAP is the total resistance, which is also shown in the table for reference to the WEAP output. Notice, however, that WEAP calculates the total resistance differently to UniPile (UniPile calculates the total resistance per each soil layer, i. e., independently of the depth values in the tables, whereas WEAP interpolates between the depths in the table).

Resistance vs. Depth - Initial Condition

Depth (m)	Shaft Res. (KN/m)	Shaft Res. (kPa)	Shaft Res. (KN)	Toe Res. (KN)	Total Res. (KN)
0.0	0	0	0	0	0
9.0	12	12	70	77	148
13.0	25	24	150	107	257
17.0	23	22	248	123	371
20.0	23	23	317	126	453
24.0	65	64	488	704	1192
26.0	71	70	625	770	1395
28.0	77	76	774	836	1610
30.0	84	82	935	902	1837
32.0	90	88	1108	968	2076

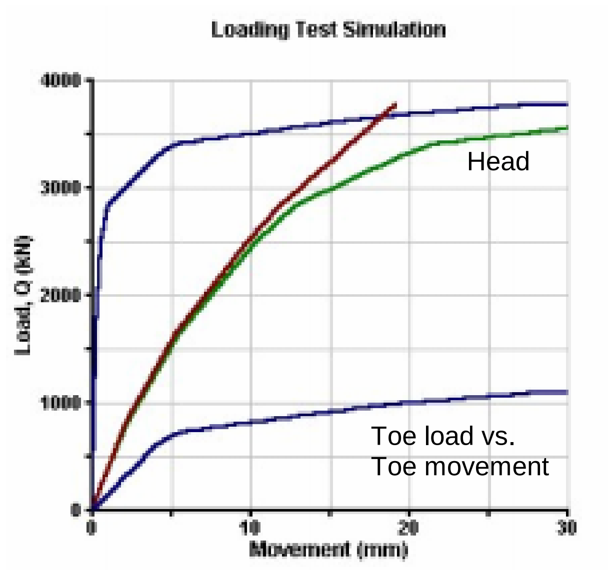
Static Loading Test Simulation

No question was asked in the survey about how the pile could be expected to behave in a static loading test. However, having the data opened in UniPile, it is easy to run a test simulation. The only input that is needed is the load-movement behavior to expect for the shaft and toe resistances. Not having any real test data to calibrate against, any choice will do as long as it is recognized that to mobilize shaft resistance requires very small movements, a millimetre or smaller, whereas mobilizing an ultimate toe resistance requires many times larger movement. In putting a toe movement of 10 % of the pile toe diameter is a reasonable value, that is about 30 mm (about an inch).



The movement dependency can be expressed in so-called t-z curves. (The term for the pile toe should really be “q-z curve” instead of “t-z curve”, but no matter). The demo file contains three shaft t-z curves (Nos. 1, 2, and 3) and two toe t-z curves (Nos. 21 and 22). As long as the shaft resistance is indicated to be mobilized at very small relative movements, either of the three can be used. This is because the shape of the pile load-movement curve is primarily a function of the toe curve and of the chosen percentage of the ultimate toe resistance to use for the toe residual load. Had the case been a real one, not just realistic—there is a difference—the simulated test behavior could have been matched to a measured behavior, which would have warranted a further discussion and comparison of different t-z curves. However, it should be clear that UniPile allows an unlimited number of trials and offers the user new insights into the analysis of piles and pile groups.

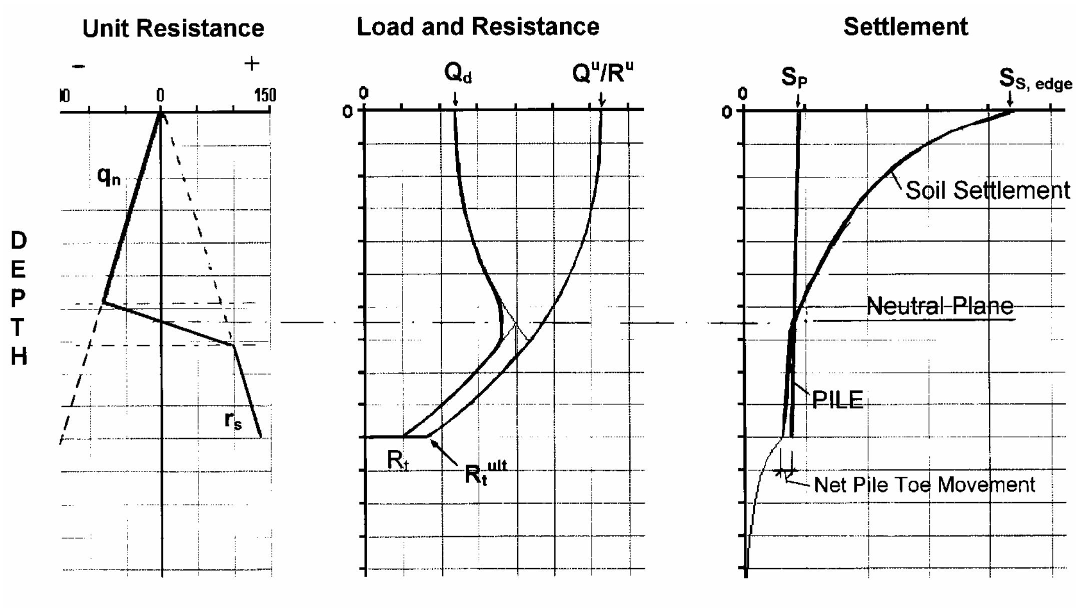
It is ordinarily not recognized that at the time of a static loading test, piles (driven as well as bored and drilled-shafts) are subjected to "residual load", which is locked-in loads. (See Fellenius 1999b, which can be downloaded from www.unisoftltd.com). Test data plotted neglecting the residual load will show a load-movement behavior for the pile toe that is similar to the one above obtained from the t-z curve No. 21 of the demo file. However, if the residual load is recognized and the more appropriate t-z curve No. 22 is used, the more correct picture shown below is obtained (the former diagram is quite similar in appearance, which is due to the incorrect t-z curve No. 21).



The Unified Pile Design

The author has published detailed recommendations for the analysis of piles and pile groups considering capacity and settlement, and dragloads and downdrag (Fellenius, 1984; 1989; and 1999a; the latter can be downloaded from www/unisoftltd.com). The method is now included with the AASHTO Specifications/Code. The principles are summarized in three diagrams shown in the following:

The first diagram of the three indicates the distribution of unit shaft resistance, r_s, and unit negative skin friction q_n. The diagram assumes, which is a reasonably correct assumption, that the magnitude of the unit shear force between the pile and the soil is the same in either negative or positive direction. The linearity is only for illustration and the distribution in an actual case would be according to the soil type and prevailing effective stress. There is no need for assuming an average soil shear.



The middle diagram shows two curves. The right side curve is the distribution of ultimate resistances: ultimate toe resistance, R_t^{ult}, and total ultimate resistance, R^u (or, ultimate load, Q^u). In long-term service, the distribution of axial load in the pile follows the left side curve, starting from the dead load applied to the pile head and increasing with depth due to negative skin friction until the neutral plane, below which the load in the pile reduces due to positive shaft resistance and mobilized toe resistance, R_t. The neutral plane is the point of equilibrium between the downward and upward acting forces. Nearest the neutral plane, the transition between negative skin friction and positive shaft resistance occurs in a zone as indicated. The height of this zone is a function of the magnitude of relative movement between the pile and the soil, and of the soil type. The height in an actual case can range from a few through many pile diameters.

The last diagram shows the distribution of settlement. Above the neutral plane, the settlement is caused by stresses imposed on the soil from fills, groundwater table lowering, footing loads at the site, etc. No part of the pile load will be transferred to the soil above the neutral plane. Below the neutral plane, the pile load will start to go out into the soil introducing stress that causes additional soil settlement. However, within the pile embedment zone, the piles will have a soil reinforcing effect and settlement will be small. A simple approach to calculating the settlement is to assume an equivalent footing loaded with the total dead load on the pile group and to perform a conventional settlement analysis for this footing including in the analysis all outside factors also affecting the change of effective stress in the soil. A typical settlement distribution is indicated in the diagram. The settlement of the pile head is indicated by s_P and the settlement of the soil by s_S. The settlement of the soil just outside the edge of a pile group, $s_{S,\ edge}$, will be greater, as opposed to inside the pile group. This will have some effect on the magnitude of the load in the piles, but if the pile cap is stiff, all piles will have essentially the same depth to the neutral plane. (Depending on details such as the pile spacing and number of piles, the inside piles will have a transition zone of greater height as opposed to the outer piles ("edge" piles), which will result in a smaller dragload on the inside piles).

The location of the neutral plane is a result of interaction between the shear forces and the pile toe resistance. Both the negative skin friction and the positive shaft resistance can be considered to require only a negligible amount of movement to mobilize fully. However, this is not true for the toe resistance, which is a function of the net pile toe movement. The values are difficult to determine. In an actual design case, when determining the maximum load in the pile (dead load plus dragload) one should assume a fully mobilized toe resistance. When determining the settlement of the pile, one should assume a less than fully mobilized toe resistance, which results in a higher location of the neutral plane and a larger calculated settlement.

The analysis illustrated in the three diagrams, can be performed by "hand" using a conventional effective stress analysis in simple spreadsheet approach or by the UniPile program.

Finally, in an actual design case, when the site and the pile conditions have been determined, the design proceeds in three steps:

1. The allowable load (dead load plus live load) is equal to the pile capacity, Q^u (ultimate resistance R^u) divided by the factor of safety.
2. The load — dead load plus dragload — at the neutral plane must be smaller than the axial structural strength of the pile divided by a factor of safety (or by similar approach to the allowable structural load)
3. The settlement calculated for an equivalent footing placed at the pile toe level or at the neutral plane must be smaller than the maximum tolerable value.

References

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

Fellenius, B.H., 1989. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75 - 82.

Fellenius, B.H., 1998. Recent advances in the design of piles for axial loads, dragloads, downdrag, and settlement. Proceedings of a Seminar by American Society of Civil Engineers, ASCE, and Port of New York and New Jersey, April 1998, 19 p.

Fellenius, B.H., 1999a. Basics of foundation design. Second expanded edition. BiTech Publishers, Vancouver, 140 p.

Fellenius, B.H., 1999b. Bearing capacity—A delusion? Proceedings of the Deep Foundation Institute 1999 Annual Meeting, Dearborn, Michigan, October 14 -16, 1999.