

Review of Piled Foundation Design—Example of a Fictitious Case

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The Terlicei Corporation is establishing a new plant near the city of West Terlicei, Tr. The site lies near the center of a former flood plain, and the soil profile, as indicated in an exploration report performed by local geotechnical engineers, comprises an upper about 9 m thick layer of compact outwash deposit (Layer 1) containing sandy silt and some organics. It is deposited on Layer 2, a 30 m thick layer of soft compressible, slightly overconsolidated ($\Delta\sigma' = 25$ kPa), marine silty clay defined by three sub-layers. Layer 3 consists of an about 1 m thick layer of compact uniform sand. Layer 4 is a thick deposit of dense to very dense residual sand and gravel with the occasional boulder.

The groundwater table lies at about 1 m below the ground surface and the pore pressure distribution is hydrostatic in the outwash soil. The pore pressure measured in the sand layer at about 30 m depth is artesian to about 2 m above the ground surface. The pore pressure distribution in the in-between clay is stated to be linear (or the clay would be consolidating). Table 1 shows results of laboratory soil tests performed to establish the compressibility of the soil layers and the soil densities.

Table 1

		Thickness	Density	Compressibility			
		h	ρ	j	m	m_r	E
		(m)	(kg/m ³)	(--)	(--)		(MPa)
Layer 1	Outwash	9	1,900	0.5	90	---	9
Layer 2a	Clay	30	1,725	0	20	200	---
Layer 2b	SiltyClay	30	1,750	0	25	200	---
Layer 2c	Clayey Silt	30	1,775	0	25	200	---
Layer 3	Sand	1	2,100	1	120	---	12
Layer 4	Residuals		2,150	1	200	---	20

Most of the plant foundations will be supported on single piles or narrow pile groups with 3-pile maximum width and all will have the foundations more or less placed at the current ground elevation. Contemporaneously with the construction, a 2.0 m thick fill will be placed across the site.

Based on their past experience from previous projects in more or less the same geology, the project engineers have designed the foundation piles to be 14-inch spun piles driven to 41 m depth. Boulders appear frequently in Layer 4. Therefore, in order to avoid piles breaking during termination driving, the piles will only be driven a short length (1 m) into Layer 4 and the termination criterion is light. The pile E-modulus is 30 GPa. The intended sustained load is 1,200 kN. There is only a small (100 kN) live load.

Starting at the time of project construction, the site will be subjected to water mining in Layer 3 which will continue for many years. A hydrological engineering study of the effect of the mining indicates that the long-term pore pressure distribution will become hydrostatic from the current level (1 m below the ground surface). Recently, concern was expressed for the soft clay layer settling over time inducing negative skin friction and a drag force—possibly worrisome—and, perhaps, excessive settlement of the foundations. The structural engineers say that about an inch total foundation settlement can be accepted.

The Terlicei engineers are local engineers (structural and geotechnical) and experienced with similar foundations and have produced many successful designs in the past. They did not see the need for a project-specific static loading test before arriving at their final design. Nor do they now ask for one. We do not know if they merely applied what appeared to have worked well on previous projects or if they had actually calculated the shaft resistance in the soil layers and the toe response. If they did do an analysis, we do not know if they applied an effective stress approach or assumed stress-independent parameters. However, some experience-based thought must have gone into their choice of pile, its lengths, and the assigned sustained load.

as being a safe one. When taking a final look at their design, they simply realized that the water mining (and the initial artisan head changing to a final hydrostatic) might infer conditions beyond what they usually encounter and they found that they like to benefit from a second opinion.

The requested second opinion should tell whether (1) everything is okay, (2) there is a problem afoot, and, if so, (3) what should be done to resolve the problem, e.g., if a static loading test or other tests would be needed to verify the design or to serve as a base to amend it.

Of course that second opinion cannot be a simple best-judgment statement regardless of how experienced and knowledgeable the person expressing it could be. It must be based on assessing results of calculations of the response of the pile and soil using the provided information. The issue of the project (as for every project) is verifying that the foundations will not settle more than can be accepted. Therefore, an analysis must address the force-movement response of the pile-soil for the applied load, as affected by the soil layer settlement that develop due to the change of effective stress caused by the pore pressure change and the surface fill. The assignment is to provide that analysis.

Note that a calculation employing stress-independent analysis (so-called total stress analysis) has little merit. Especially so for this project involving change of effective stress with time. Moreover, declaring that the required "capacity" of the pile will be a certain factor times the supported maximum load would be meaningless, regardless of what method and definition that would be used to determine the "capacity".

Both shaft and toe resistances considered for the analysis need to include the mobilizing movement for each soil layer. The analysis must establish for each pile element, the force (shaft or toe resistance) and how it depends on the movement and, then, accumulate this to show the total for the pile. It is best achieved by choosing a specific value of unit target resistance for a pile element and its specific target movement relative to the soil and, then, choosing how this resistance changes with the movement.

There are numerous functions expressing the relation between shaft and toe resistances to movement called t-z and q-z functions. For details see the Red Book, Section 8.5. Figure 1 shows three alternative such curves that "pivot" around a target point for the analysis shown as 100 % of any assigned unit resistance and 100 % of the assigned movement for that resistance.

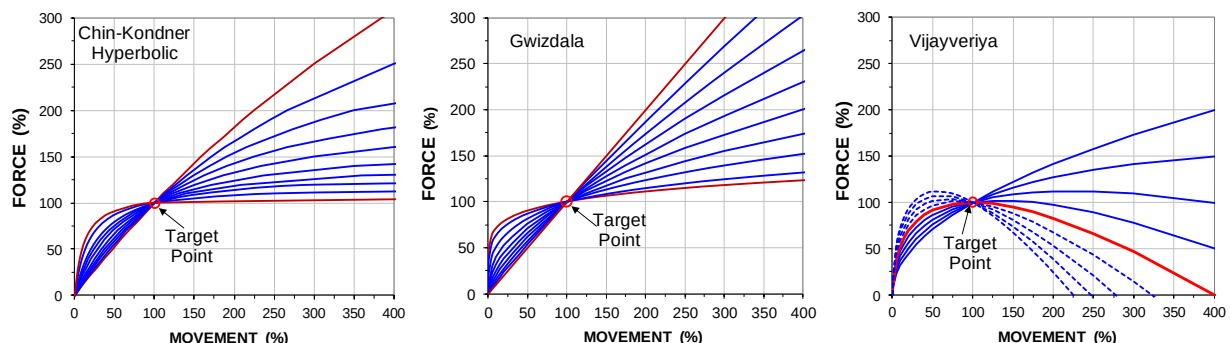


Fig. 1 Example of t-z and q-z curves for three typical functions applicable to force movement of piles

The project information provides soil densities and pore pressure distributions for initial and final conditions. Therefore, after adding-in the effect of the 2-m general surcharge, the initial and final effective stress distributions are known. The immediate difficulty lies in choosing for the soil layers a target unit shaft shear and unit toe stress applicable to the response of the subject 14-inch spun pile and, then, decide what type of t-z/q-z functions and function coefficients to apply to express how the response changes with the movement. Usually, shaft resistance answers to a hyperbolic function and toe resistance is best expressed in a Gwizdala function.

Any target movement appropriate for the shaft resistance target can be chosen, but a 5-mm value is a practical choice. This because, normally, a shaft element movement relative to the soils increases rapidly within the first 5 mm—stiff appearance of resistance. The continued development is less stiff even when strain-hardening, but can be plastic or strain-softening. In contrast, the toe resistance usually develops along a smooth curve per a Gwizdala function.

After some discussion and exchange of information with the Terlicei engineers and guided by experience from driven concrete piles in clay, the β -coefficient and toe stress target values for 5 mm target movements listed in Table 2 appeared were chosen.

Table 2

		Shaft β (--)	Toe N_t (kPa)
Layer 1	Outwash	0.22	(---)
Layer 2a	Clay	0.18	(---)
Layer 2b	Silty Clay	0.18	(---)
Layer 2c	Clayey Silt	0.10	(---)
Layer 3	Sand	0.25	(---)
Layer 4	Residuals	0.30	30

As to the t-z and q-z functions, after similar discussions and considerations, it was decided to use the same shaft Chin-Kondner hyperbolic function for all soil layers, but for Layer 2c, and a coefficient, C_1 , equal to 0.0090, representing a β -coefficient of $1/90\% = 1.11\%$ of the target β -coefficients at infinite movement, almost like assuming plastic response beyond the target movement. Information regarding Layer 2c indicated that its shear force versus movement had a slow rise, and would require much movement to reach the same resistance as Layers 2a and 2b; a target shear of $\beta = 0.20$ would require 20 mm movement. Assigning a C_1 -coefficient of 0.0065, the β -coefficient for a 5-mm movement comes out as 0.10 (which corresponds to $\beta = 0.60$ at infinite movement). The toe response was assigned a Gwizdala function with a coefficient, θ , equal to 0.60. The functions are shown in Figure 2.

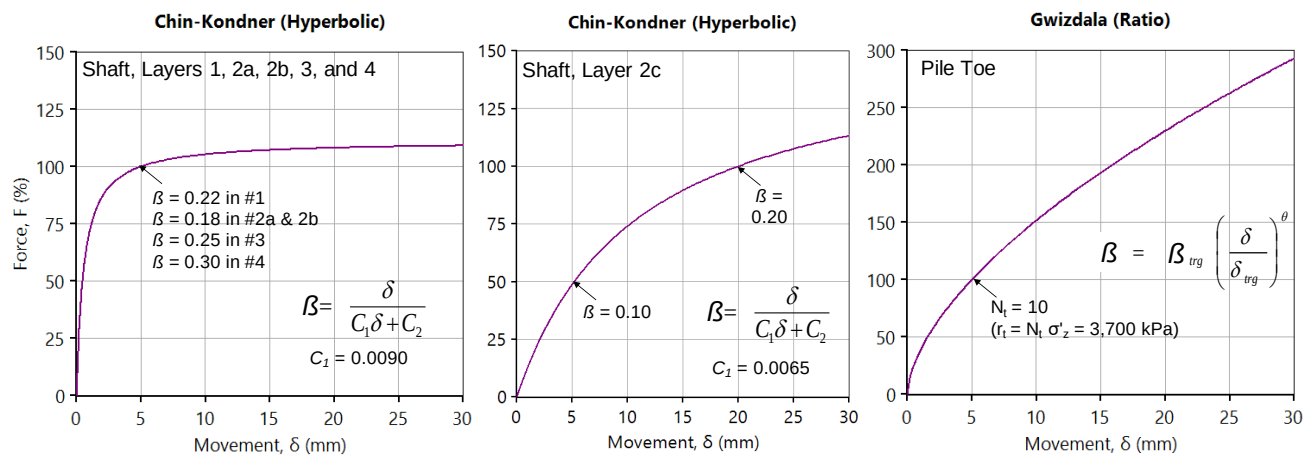


Fig. 2 The t-z and q-z curves selected for use in the analysis

The Table 2 values and the t-z/q-z functions of Figure 2 developed for the analysis are within typical ranges for the soil types of the project. Yet, choosing them required much input of time and thought.

Because of the relatively simple conditions of the example, the analysis of the response of the single pile to an applied load can be performed in an Excel spreadsheet. However, the analysis would only provide a few of the basic aspects needed for assessment and it would also require investing considerable time for the required very complex series of trial and error calculations. Yet, the results would only be approximately representing the pile response. The UniPile6 software requires minimal time for the input and the output provides all calculation results necessary for the assessing the suitability of the proposed piled foundation design. [Tables 1 and 2](#) show the soil input to use in the analysis along with the pile specifics. The t-z and q-z functions shown in [Figure 2](#) were chosen for the analysis.

The soil layer input for the clay layer, Layer 2a, is shown in [Figure 3](#) and the spun-pile input is shown in [Figure 4](#). The distributions of initial and final (long-term) effective stresses and pore pressures at the site and the soil settlement calculated by UniPile6 and plotted from the software output are shown in [Figure 5](#).

The screenshot displays the UniPile6 software interface for inputting soil layer data. The left sidebar contains a menu with options like INFORMATION, SETTINGS, SOIL PROFILES (1), PORE PRESSURES (2), LOADS (1), FILLS, EXCAVATIONS, CPT DATA, SPT DATA, PILE DATA (1), PILE GROUPS, MESH Z, t-z/q-z FUNCTIONS (4), and DESIGN NOTES. The main window is titled 'Terlicei: Layer #2a' and is divided into three sections: 'Profile Info...', 'Soil Layer', and 'Basic Properties'.

Profile Info...

#	Layer Name
1	Layer #1
2	Layer #2a
3	Layer #2b
4	Layer #2c
5	Layer #3
6	Layer #4

Soil Layer

Name	Layer #2a
Soil Classification	Clay
Display Color	Color [A=100, R=0, G=0, B=255]
Label	
Description	
Design Notes	

Basic Properties

Thickness (m)	11.00
Depth (m)	20.00
Void Ratio, e0	0.685
Total Density, pt (kg/m^3)	1,725.0

Shaft Resistance Parameters

Shaft Resistance Method	Beta-Coefficient, β
Beta-Coefficient, β	0.180
Cohesion Intercept, c' (kPa)	0.0
Unit Shaft Resistance Limit (kPa)	0.0
Shaft t-z Function	Shaft Chin-Kondner All layers but 2c

Toe Resistance Parameters

Toe Resistance Method	Unit Toe Resistance, rt
Unit Toe Resistance, rt (kPa)	0.0
Toe q-z Function	Select...

Preconsolidation

Preconsolidation	Prestress Margin, $\Delta\sigma'$
Prestress Margin, $\Delta\sigma'$ (kPa)	20.0

Consolidation

Method	Modulus Number, m
Stress Exponent, j	0.00
Modulus Number, m	20.0
Recompression Modulus Number, mr	200.0

Shaft Resistance Method

Parameter used to calculate the shaft resistance using the 'Static' method. Select Beta-coefficient or stress-independent unit shaft resistance.

Fig. 3 Input page for soil layer #2

General	
Name	Single Pile
Pile Type	Concrete
Description	Spun Pile
Coordinates, X/Y (m)	0.00 / 0.00
Pile Head Depth (m)	0.00
Pile Toe Depth (m)	41.00
Load Cell Depth (m)	41.00
Axial Load, Qd/Ql (kN)	1,200.0 / 100.0
Inclination (Degrees)	0.0
Installation Method	Driven
Display Color	Color [A=255, R...
Design Notes	

Shaft Geometry	
Shaft Shape	Round
Shaft Diameter, b (mm)	357.0
Modulus Override, E (GPa)	0.00
Reinforcement Ratio (%)	0.01

Toe Geometry	
Shape	Toe Ax = Shaft Ax

Reinforcement Ratio (%)
Steel reinforcement ratio used to adjust the modulus of elasticity of concrete. This value disregarded when the modulus is overruled.

Fig. 4 Input page for pile data

First insight is that the settlement in the rapidly consolidating coarse-grained soil layers (Layers 1, 3, and 4) will occur during construction and before any piled foundation settlement would be of concern for the foundation assessment. Therefore, the compressibility of those layers can be removed from the analysis. Moreover, the settlement calculated for the ground is of interest for the general area, connections of piles and entrances, etc., but not for the piled foundations. An important question is also that the calculated settlement will take a long time to develop. UniPile6 only delivers initial and long-term values. For analyzing the development over time, the coefficient of consolidation, c_v , of the clay needs to be known. If the settlement vs. time would need to be determined, the UniPile6 input data can be exported to UniSettle5. Then, with input of the clay c_v , UniSettle5 can calculate the settlement at different times.

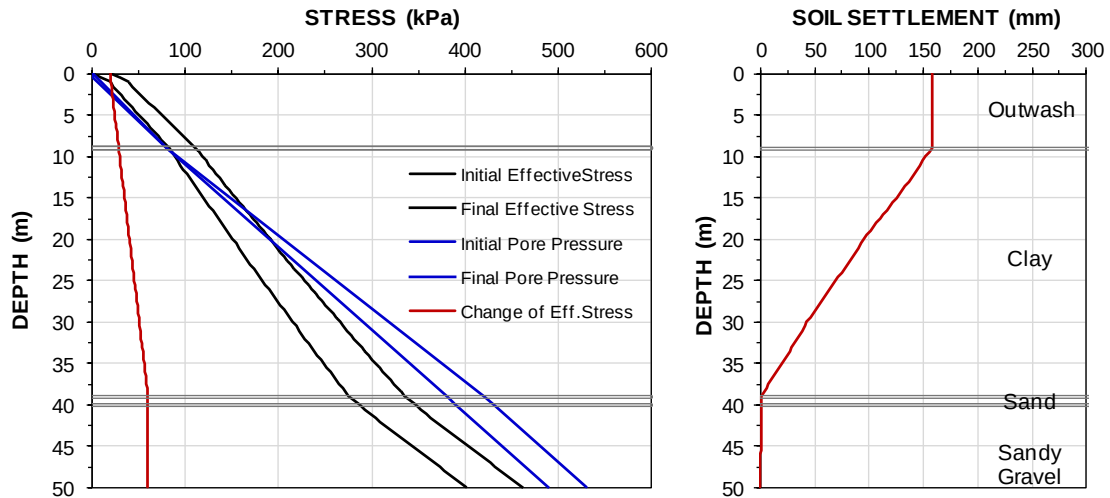


Fig. 5 Distributions of soil stresses and settlement calculated by UniPile6

Figure 6 shows the output for the short-term ("initial") distribution of axial force and "settlement" of the pile. The initial conditions are with the initial pore pressures and before the fill was placed, and the "settlement" comprises the pile-toe movement and pile compression as induced by the 1,200-kN sustained load. The first matter worth attention is that the pile compression plus small toe movement for the load placed on a pile during the construction of the structures is about 12 mm. Note, only the settlement developing after the construction is normally of concern for a foundation. A calculation of the long-term deformations and settlement includes this at-end-of-construction value and it needs to be subtracted from the calculated final settlement before assessing what settlement to expect for the structures. Note also that the at-end-of-construction shaft resistance is only mobilized beyond the 5-mm target movement in the upper half length of the pile and that the developed toe resistance is small.

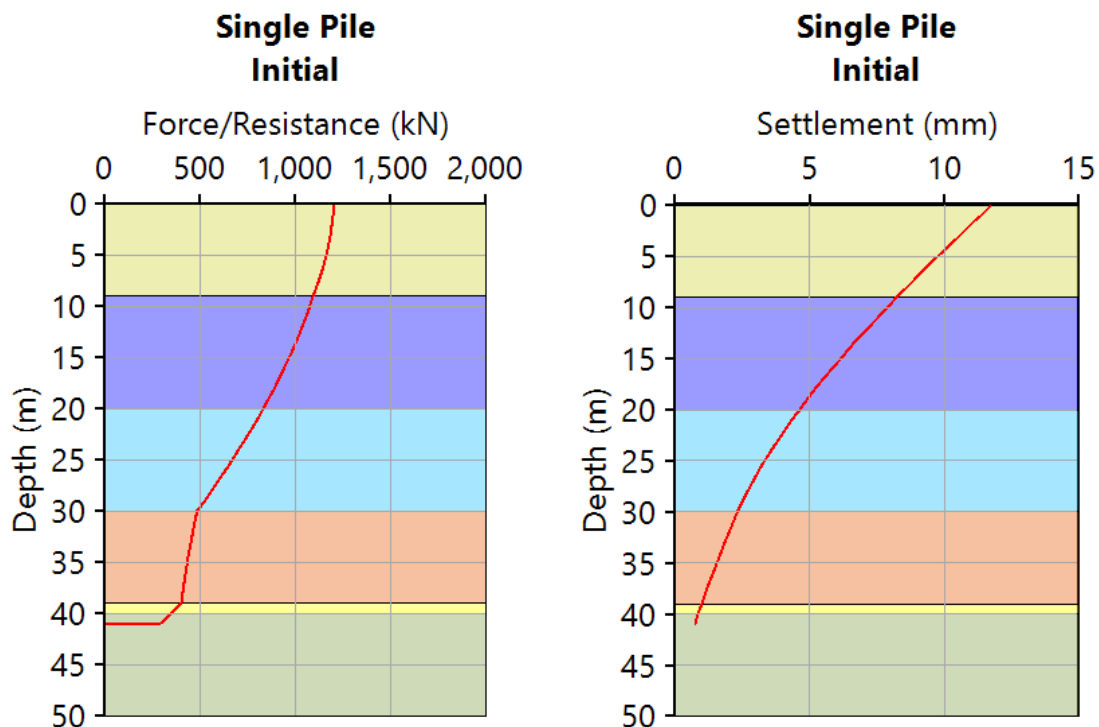


Fig. 6 Distributions of axial force and settlement immediately after construction

Conventionally, the simulation of the load-movement curves of a static loading test is the first-to-consider output of a piled foundation calculation. Figure 7 shows the UniPile6 output for the long-term ("final") conditions. Looking at a relation between the 1,300-kN total load (sustained plus live load) applied to the pile in relation to a "capacity" determined from the pile-head load-movement curve serves little purpose. The first matter worth attention is that the pile compression for the load is about 12 mm, i.e., half of the stated settlement limit, confirming the observation and conclusion made in regard to Figure 6.

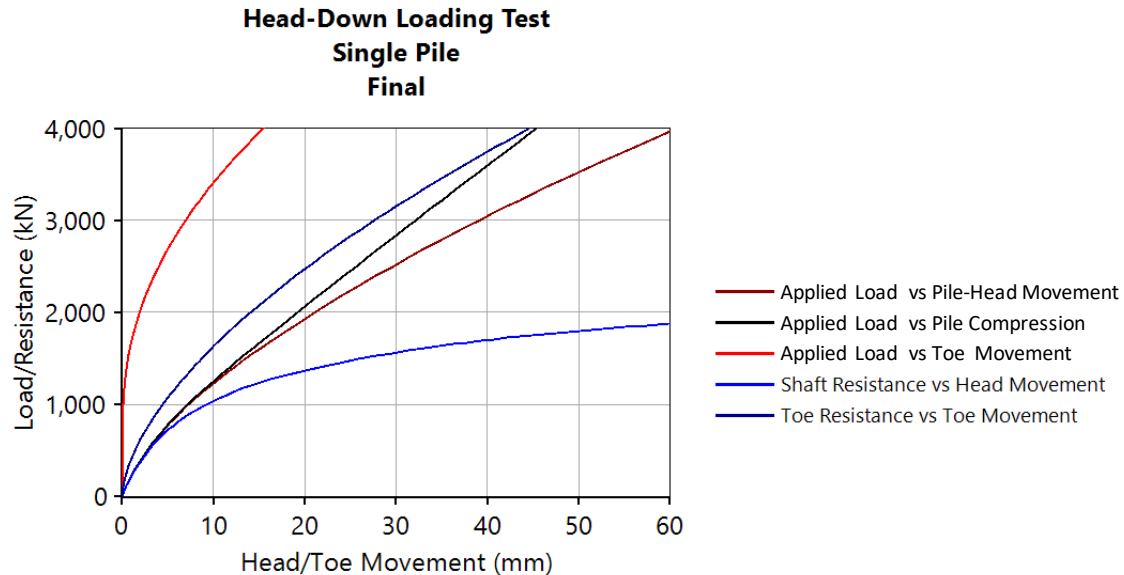


Fig. 7 Load-movement curves from simulation of long-term condition

The decisive output for assessing the pile is the settlement and pile force simulation according to the Unified Method shown in Figure 8 (the output also comprises tables). The simulation indicates that the total pile-head settlement is 40 mm. Subtracting the about 10 mm settlement that will have occurred as the structures were built, the analysis suggests that the long-term settlement of the piled foundation will be about 30 mm. The output also shows the Equilibrium Plane at 35 m depth, a maximum force in the pile of 2,700 kN, which is insignificant in relation to the pile structural strength, and a drag force of 1,500 kN, which is of no consequence for the design.

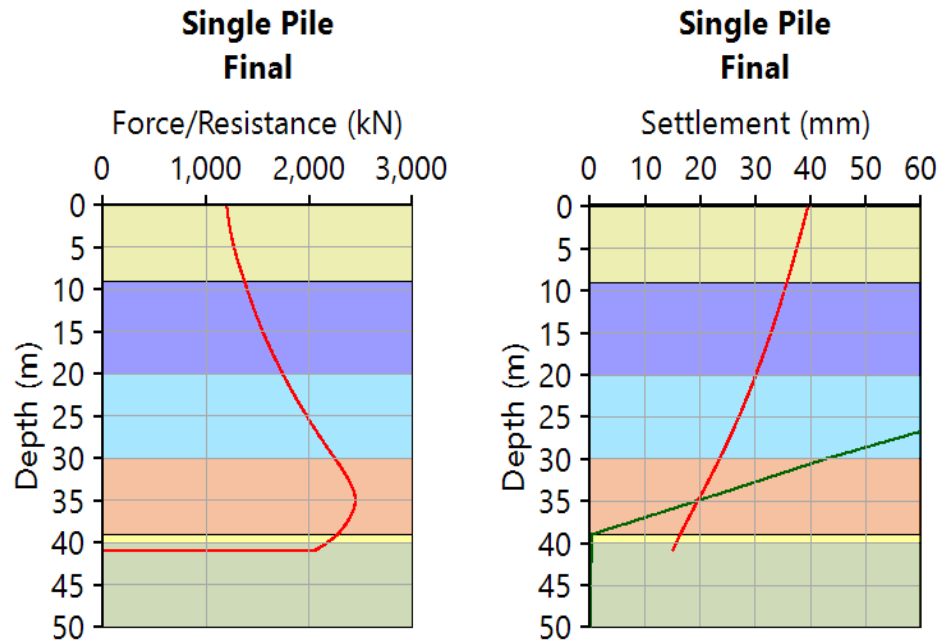


Fig. 8 UniPile6 output for Unified Method

The about 30-mm long-term settlement is a bit larger than the stated "one-inch" limit, 25 mm, but might be accepted as "close enough". However, all analysis input is with a margin and the output has a corresponding margin. The results imply that the design is OK, but they cannot be used to emphatically state that the proposed design is fully satisfactory. Nor can one state that the design will have to be changed. However the analysis results show clearly that the issue is not the reliability, representativeness, or precision of the settlement analysis of the soil, notably the clay, or the assumed β -coefficients, but that of the pile-toe response. For example, a renewed UniPile6 analysis with an N_t -coefficient of 50, i.e., assuming an larger target toe resistance, would lower the depth to the EP and reduce the calculated long-term settlement of the pile.

The best advice would be to look a bit more into the pile toe response. For example, doing dynamic tests (PDA and CAPWAP) on a few of the first construction piles driven as test piles to study the pile toe response, specifically, in terms of bearing and tendency to toe damage

In a real case, as opposed to an example for illustration of an approach, more information on the 30 m thick clay layer would have been desirable. A second look at the clay samples would provide information for revisiting the clay input data. Moreover, results of a CPTU- sounding would be very helpful. Input of such data for a renewed UniPile6 analysis to resolve the questions takes mere minutes.