A pile with the casing extended, then excavated

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Case Study

Residual Force and Downdrag. Impacts on Static Loading Tests and Design of CFA Piles

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Abstract: Construction of a three-storey school complex over compressible silt and silty clay followed by sand was affected by fill placement required to reach finished grade. Pile foundations were selected to mitigate excessive settlement caused by the building loads and downdrag resulting from the fill. Static loading tests on instrumented piles were performed to confirm the design. The back-analyses of the tests revealed presence of residual force and were adjusted for the final design by applying interactive response of force and movement. The analysis results showed that the initial design of the foundations was suitable. Mistakes in assessing live load and drag force, when applying the results of the static loading tests, and a misinterpretation of the building code led to the conclusion that the piles, as initially designed, would need to be substantially lengthened. Final assessment showed that lengthening was not necessary.

Keywords: piled foundations, static loading tests, strain-gage instrumentation, cptu, residual force, downdrag, drag force, settlement, building code

Introduction

The foundations of a three-storey school building constructed in Perth Amboy, New Jersey, at a site with subsiding ground required piled foundations. The most challenging and initially misunderstood aspects, including common misconceptions, were that, initially, sustained loads were combined with the drag force in the analysis of ultimate limit state bearing conditions. Moreover, a confusion between strength and limit state analysis, as it relates to downdrag, and misinterpretations of the building code caused portions of pile length to be excluded from contributing to the resistance.

Analysis Principles

A piled foundation is the alternative chosen when a geotechnical analysis shows that, if a structure is placed on a shallow foundation, excessive long-term settlement will or might develop. Settlement of a piled foundation beyond load-transfer movement can stem from the applied loads compressing the soil below the pile-toe level and/or from downdrag due to general subsidence, fill or other occurrences, such as a lowering of the groundwater table. Often, on having recognized that foundation settlement would be excessive, a paradox develops in that the continued design effort does not address settlement, but capacity. Moreover, it not only disregards that there still could be a settlement issue, but pursues an incorrect approach as to how the subsidence-induced drag force combines with the sustained and transient (live) loads.

Figure 1 shows the principles of interactive force and settlement design analysis ("unified method") applied to a typical single pile subjected to subsidence (Fellenius 1984; 1988; 2023). The pile is assumed to be a single 22 m long pile supporting a 700-kN permanent load (dead or sustained), Q_d, here, similar to the project piles. The conditions are for long-term, when the subsidence has ceased and pile and soil have reached an equilibrium condition. The left graph shows axial load distribution starting at the pile head with the applied permanent load, Q_d, and increasing with depth due to negative direction shear force, imposed by the subsidence, accumulating down to a drag force, Q_n, at a "Neutral Plane" (depth of the maximum force in the pile) and then decreasing due to positive direction shear forces to a toe force, R_t. The right graph shows the long-term subsidence at the site. At the neutral plane, the pile and the soil movement (settlement) are equal and the graph shows the pile settlement as a function of the pile toe movement and the pile 'elastic' force response (the slope of the "pile" curve). The neutral plane is also called the force equilibrium (left graph) or settlement equilibrium (right graph).

Note, that the resistances represented by the force distributions are long-term values developed as the soil adjusts to the movements and shear forces and toe stress. They are movement-dependent and, therefore, not necessarily, and not likely, ultimate values. The figure does not indicate a "capacity" because it and its ratio to the applied load are irrelevant to the function of the piled foundations in supporting the structure. The factor of importance is the long-term settlement of the pile head (i.e., of the piled foundation).

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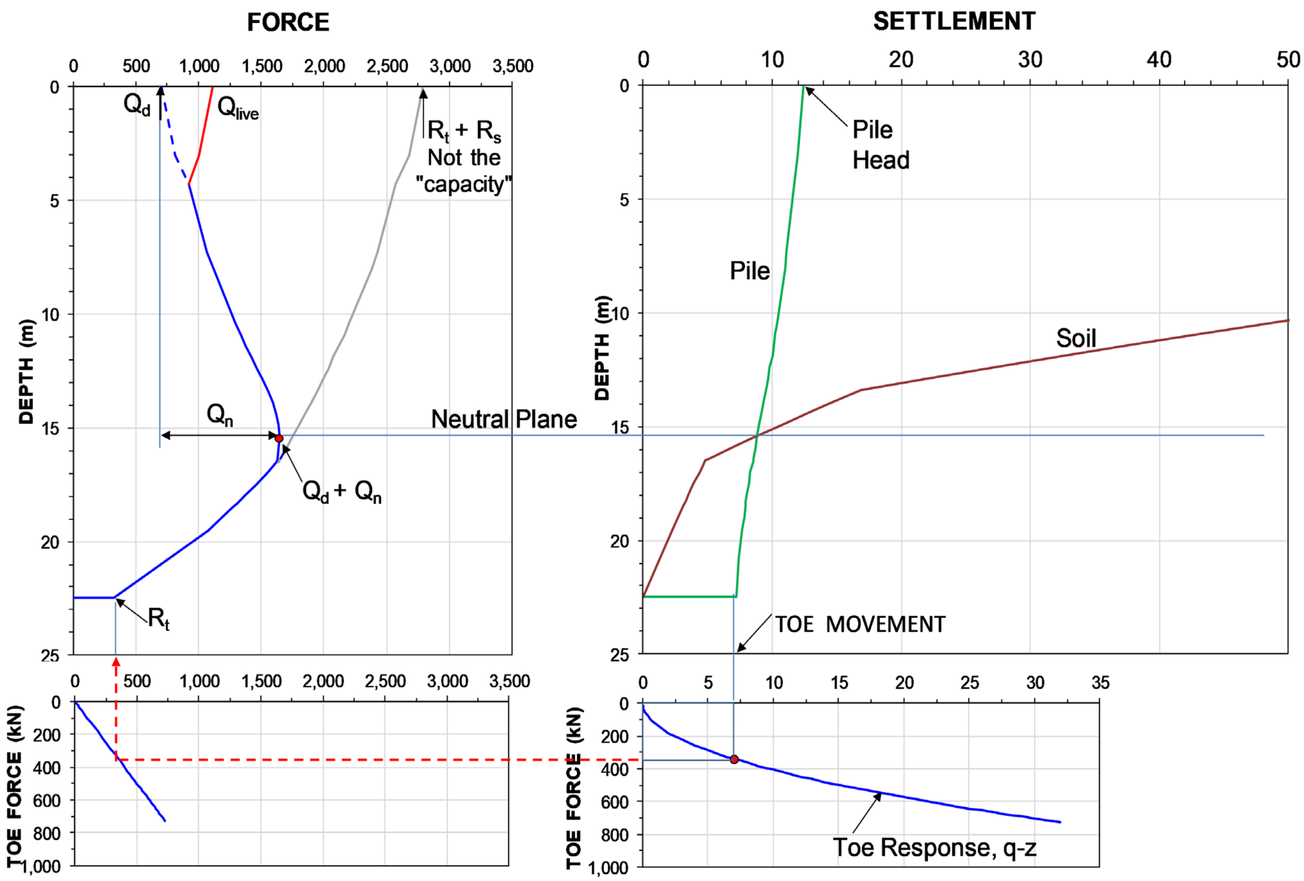

Figure 1. Principles of the "unified" design

The maximum axial pile force develops at the neutral plane location. The figure demonstrates that had the sustained load been larger, the neutral plane would have moved up and the new equilibrium would have resulted in larger toe resistance commensurate with larger toe movement and settlement, and *vice versa* for a smaller sustained load. That is, the piled foundation settlement is the result of interaction between forces and movements. Establishing these in a design requires analysis of shear-force movements and toe-force movement together with analysis of soil-settlement distribution.

The figure also shows the effect of applying a 400-kN temporary (live or transient) load, Q_{live}, to the pile (larger than usual building live loads for reasons of emphasis). This changes the direction of shear forces near the pile head from negative to positive until the so-mobilized resistance becomes equal to the live load. Below that depth, there is no change of axial force distribution and no change to the maximum load in the pile. Clearly, the live load will not add to the maximum force at the neutral plane. That is, live load and drag force must not be combined in determining the maximum force in the pile.

The analysis of resistance distribution, pile element movements, and soil settlement is interactive and greatly helped by having access to results from instrumented static pile loading tests where the pile toe has been appreciably moved.

Soil Profile

The site was vacant land prior to the 1950s when residential housing was constructed. Prior to 1950, the site was bisected by a small stream which was replaced by a storm drain also constructed in the 1950s. The housing was demolished in 2011, and to obtain a level final grade, about 3 to 5 m of new fill was placed across the site (before the piles were installed). The fill typically consisted of reddish brown silt, sand, or clay with varying amounts of gravel and included pieces of roots, wood, and concrete.

The site investigation included several CPTU soundings and borings. The New Jersey standard requires boreholes at c/c 15 m across a building footprint. Figure 2 shows four diagrams of CPTU results from the location of Test Pile TP-2C and a diagram of the N-indices from an adjacent borehole plotted superimposed over the cone stress, q_c-diagram. The thickness of the fill, and the pile-head and pile-toe levels of the test pile are indicated. Figure 3 combines similar N-index and CPT q_c-distributions from all three test piles. Notice that the piles have penetrated different lengths into the silt and sand layer.

Figure 4 shows the distribution of soil compressibility, expressed in Janbu modulus number (m), estimated from

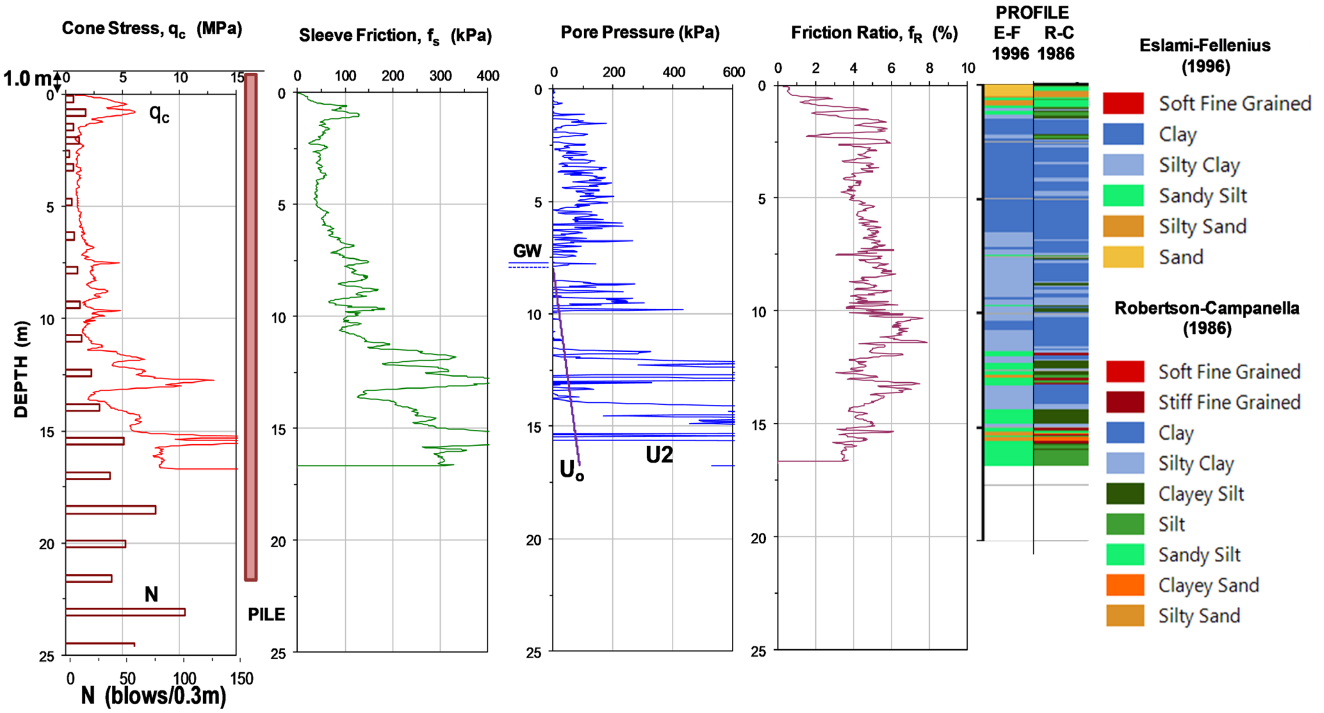

Figure 2. In-situ tests at TP-2C

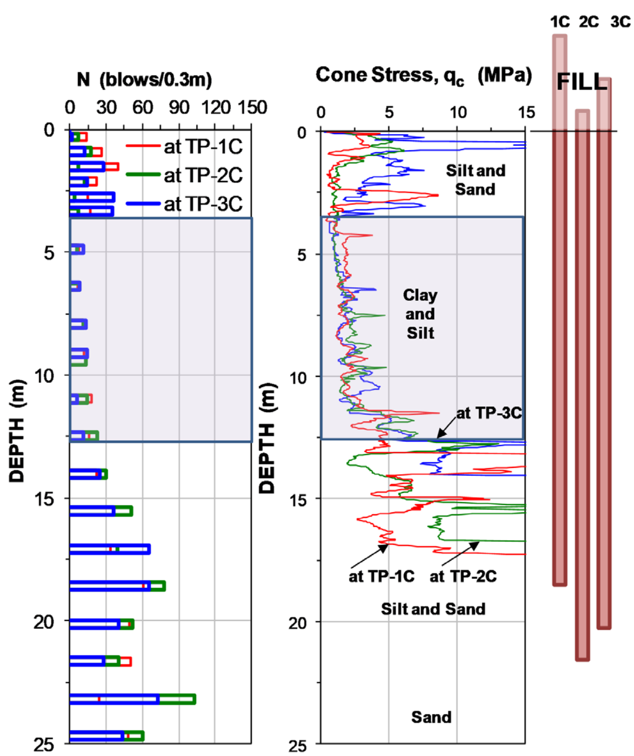

Figure 3. Combined N-index and q_c-diagrams at the test pile locations

the oedometer tests performed on samples from different depths at locations across the site (Janbu 1963, Fellenius 2023). The oedometer modulus numbers were used to calibrate the distribution calculated by the CPTU soundings (Massarsch 1994, Fellenius 2023) per the solid line shown. Applying this distribution of modulus numbers and fitting a settlement analysis to the stated 200-mm ground surface settlement gave a match to a 2 m average fill height and the distribution shown in the figure. Settlement analysis indicated that long-term settlement of the compressible soil after construction across the site would amount to about 100 to 200 mm at the ground surface.

Piles

The piled foundations comprised single piles and narrow pile groups containing 2 to 8 piles with a center-to-center pile spacing of 3 diameters (1.4 m). The piles were all 457 mm diameter continuous flight auger piles (CFA-piles) drilled in one continuous operation to 22.7 m embedment. The pile reinforcement areas for Piles TP-1C and TP-2C were 51.6 cm² and 25.8 cm² for Pile TP-3C. The assigned maximum sustained load was about 530 kN/pile. The layout of the test piles and piled foundations is indicated in Figure 5. The total building area was 25,000 m². The total number of foundation piles installed was 1,577. Thus, the Footprint Ratio, FR, of total pile area over the building area was 1.0 %.

Loading Tests

Head-down static loading tests (TP-1C, TP-2C, and TP-3C) were performed on three 22.7 m long piles 12, 16, and 21 days, respectively, after test pile construction. The maximum test loads ranged from 3,200 through 4,250 kN. The load increments were applied at varying intervals and an unloading-reloading event was included for two of the piles, as indicated in Figure 6. At maximum applied load,

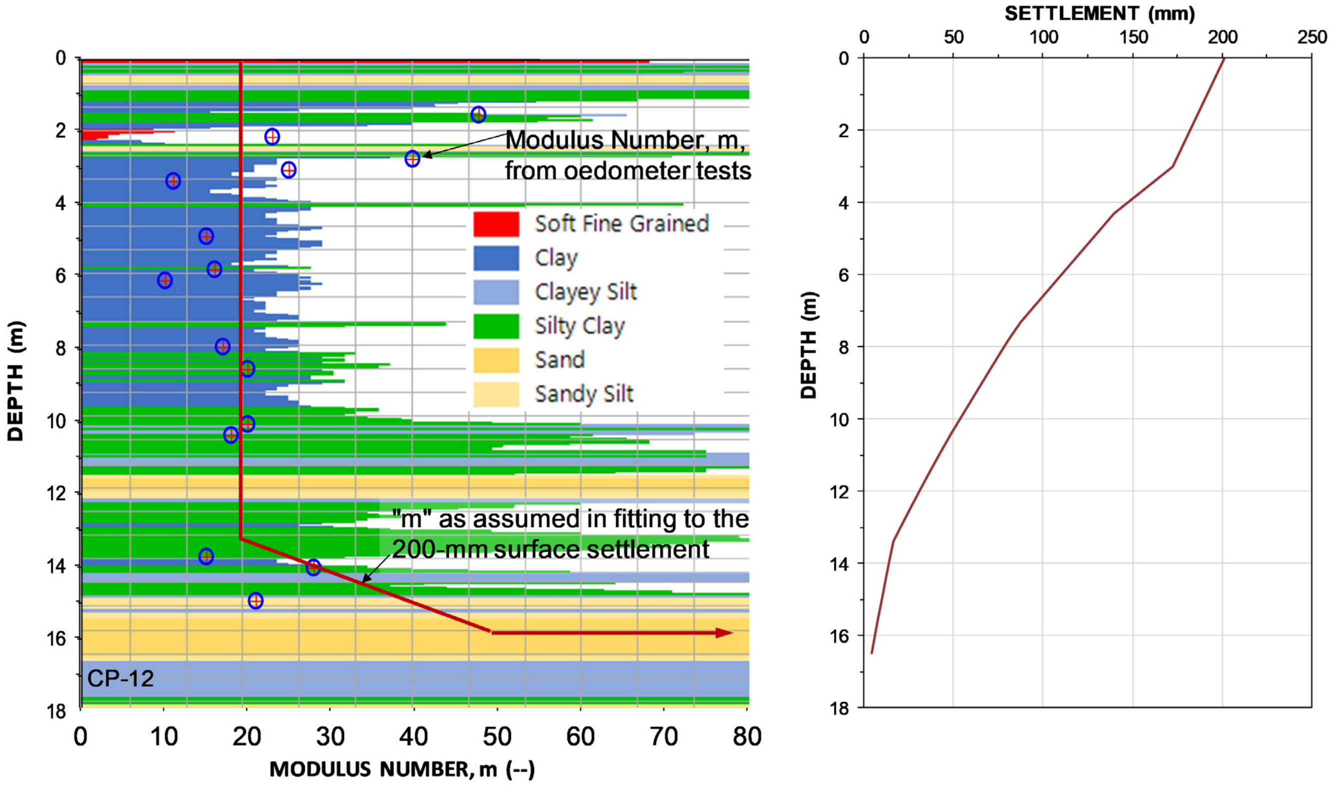

Figure 4. Distribution of Janbu modulus numbers and settlement

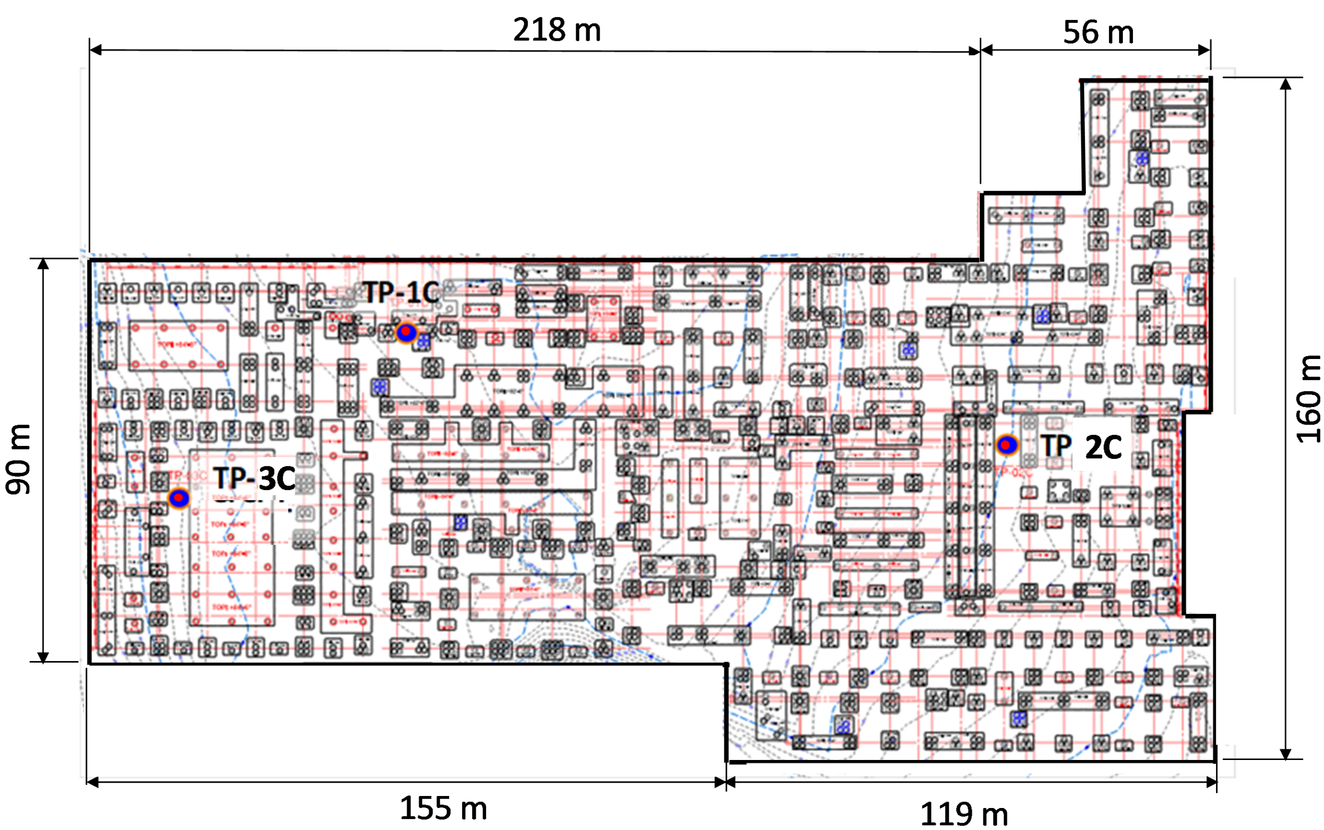

Figure 5. Layout of the test piles, piled foundations, and test piles

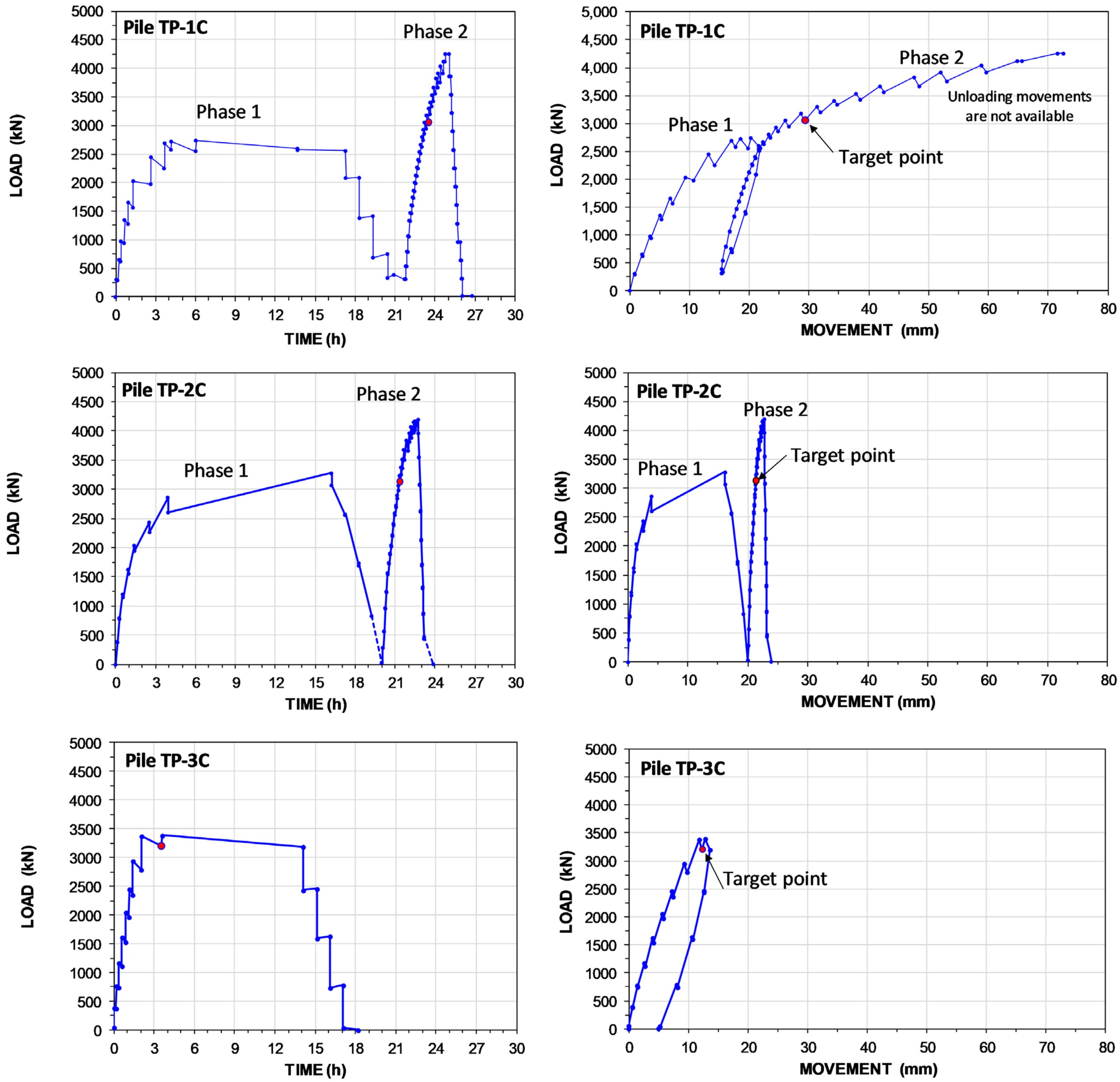

Figure 6. Load-time and load-movement records of the static loading tests

the measured pile-head movements ranged from 14 to 18 mm through 73 mm.

The test piles were instrumented with vibrating-wire (VW) gage pairs at several levels designated SG-1 through SG-8 (at depths 0.3, 4.3, 7.3, 10.4, 13.4, 16.5, 19.5, 22.5 m, respectively) for determining the distribution of the transfer of axial force resulting from each applied load. The uppermost gage, SG-1 was intended for calibrating strain to force. However, the 0.3-m distance to the pile head was far too short. Gage SG-8, which was only 0.2 m above the pile toe was similarly mislocated. Therefore, the strain plane at SG-1 and SG-8 were uneven and the records are unrepresentative. A gage pair should not be closer than two pile diameters to a boundary—pile head or pile toe. Moreover, the pile head was confined in an about 0.6 m long, 6.3-mm thick wall steel casing containing SG-1. The mislocation and the steel casing made the SG-1 strain records not representative for use with the records from the uncased gage levels further down the pile. The instrumentation did not include a toe-telltale for measuring pile-toe movement.

The purpose of a strain-gage instrumentation is to obtain the force distribution for the various loads applied to the pile head. Determining the axial force represented by a strain record requires knowing the EA-value in the basic relation of force and strain of the pile: $Q = EA\,\varepsilon$. A Young's modulus (E) value can be estimated from using the concrete strength adjusted for the amount of reinforcing steel. The average 28-day concrete strength, f'_c, for the test piles was 69 MPa. According to the American Concrete Institute, the E-modulus (GPa) as a function of concrete strength (MPa) is $4{,}700\sqrt{f'_c}$. The pile nominal area, A, is 0.164 m² (although the actual is likely somewhat larger). When adjusted for the reinforcement, the calculated EA-values are 7.2 and 6.8 GN, respectively. The ACI relation is just one of many similar equations and many other factors than strength affect the concrete E-modulus. Therefore, the so-calculated EA-value is only approximate and needs to be verified.

Figures 7 and 8 show the applied load (Q) vs. strain (ε) for Test TP-2C, Phases 1 and 2. For Phase 1 records, SG-3 through SG-5, once the shaft resistance was mobilized, the continued increase of load resulted in a more or less straight line with a slope Q/ε of about 5,000 kN/με or EA = 5 GN. The relation for converting force from strain is Q = EA ε, i.e., the slope is also the EA of the pile cross section. It is a good deal smaller than those calculated from the concrete strength and nominal area.

Alternatively, the EA-value can be determined from the records directly by means of the secant and tangent modulus method (Fellenius 1989; 2022). The secant method applies to the strain records from a gage level uninfluenced by any shaft resistance, i.e., the gage level nearest the pile head, SG-1. Figure 9 shows the secant slope distribution, the load divided by the strain as a function of strain, Q/ε, with records from the uppermost gage level, SG-1, in Test TP-2C. Had the gage been placed at proper distance, the E_sA-values, but for the first value, could have been expected to line up horizontally. However, the first several values are affected by strain-plane shift, only become more consistent at larger loads when the differences due to the plane shift in relation to the strain become relatively smaller. The effect could also be due to the gage zero-reading not being true, as it would have a similar effect. The indicated EA-value is about 4 GN. However, because of the presence of the steel casing At SG-1, this EA-value is not relevant to the values at the strain-gage levels down the pile.

Figure 10 shows the tangent slope distribution, which is the increment of load divided by the increment of strain (ΔQ/Δε), plotted vs. strain (ε) with records from Test TP-2C, Phases 1 and 2. The tangent stiffness is independent of any error in zero value. However, as it is a differentiation method, it depends very much on the accuracy of the records and the veracity is adversely affected by unloading-reloading events and uneven load-holding durations—unfortunately included with the test schedule. Moreover, when applied to strain-readings from where the shaft resistance has developed between the load application point (the jack) and the gage level, unless the shaft response is plastic, the E_tA points will not plot along a horizontal line, but be sloping, whether

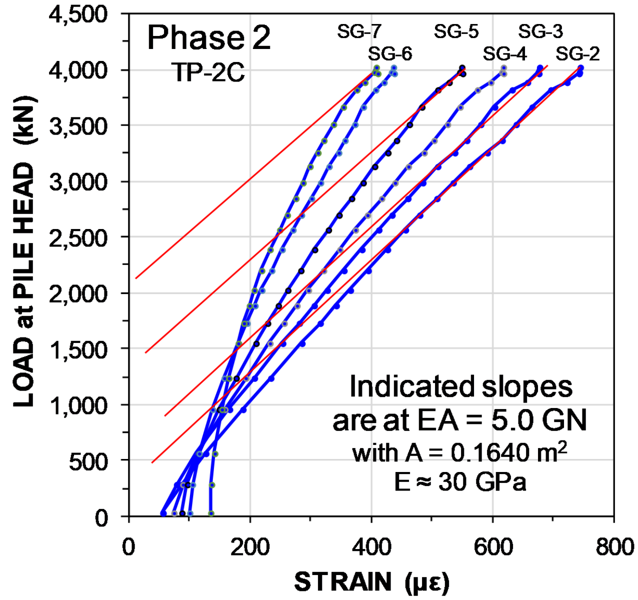

Figure 8. TP-2C Load vs. strain, Phase 2

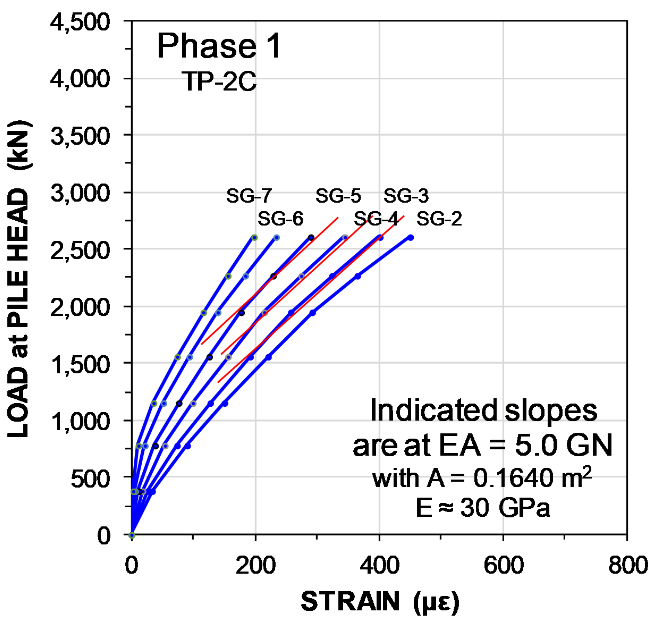

Figure 7. TP-2C Load vs. strain, Phase 1

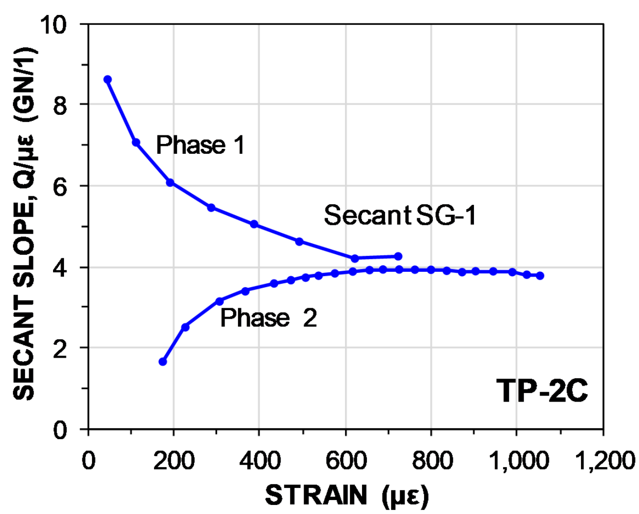

Figure 9. SG-1 Secant slope

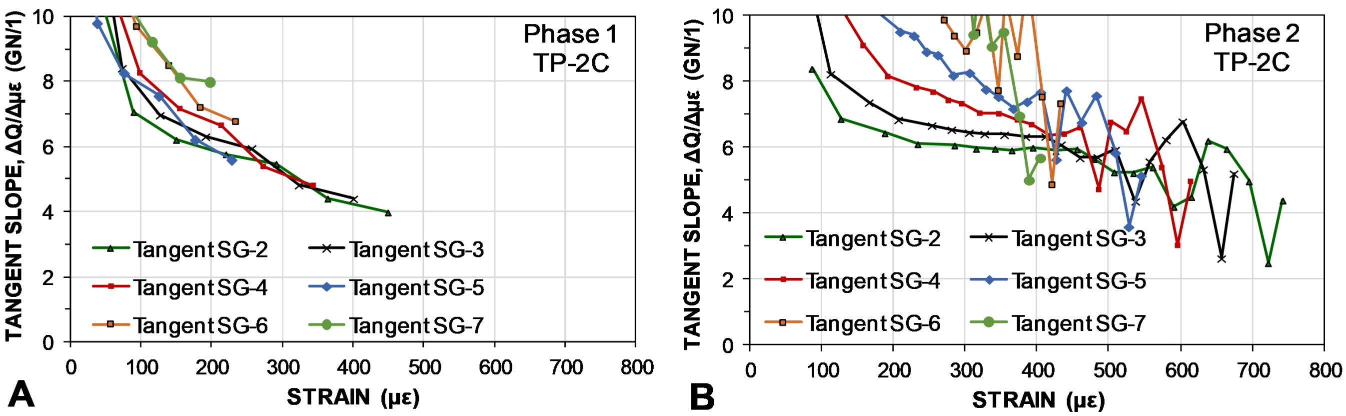

Figure 10. Tangent slopes for SG-2 through SG-7

the slope would be with a rising or lowering trend would depend on whether or not the shaft shear response is from strain-hardening or strain-softening. No clear tangent slope appears in the graph. The average tangent slope beyond 300 με is about 5.0 to 6.0 GN and the similar assessment for Piles TP-1C and TP-3C produced EA-values ranging from 4.5 to 5.5 GN.

The range of uncertainty of the EA-values can be narrowed down by calculating the force distribution for a specific "target" load-movement record for which the strain values are converted to force using trial EA-values. The so-obtained force distributions are then compared to the applied "target load" and the reasonableness of shaft response in relation to the soil layers. The target load-movement pair (the "target point") is best chosen to one that resulted in a pile movement of about 5 mm for an average pile element at the end of the load-holding duration. After some trial and error, the effort resulted in determining the EA-from the force distribution obtained from the strain records induced by choosing the 3,000 kN applied load as target load and adjusting the EA-value until the distributions appeared reasonable. This gave EA-values of 5.5 GN for Pile TP-02C and 5.0 GN for Piles TP-1C and TP-3C.

Figure 11 shows the force distribution for the applied load determined in the three tests determined applying the mentioned average EA-values applied to the strain values measured at the last reading for all applied loads. Tests TP-1C and TP-2C included an unloading-reloading event, but no unloading-reloading event was included in Test TP-3C. The highlighted lines at about 3,000 kN load signify the distribution for the target load, used for determining the EA-value in the back analysis. There is a significant difference between the force distribution of Piles TP-01C and TP-03C compared to Piles TP-02C inasmuch that after unloading Phase 1, Pile TP-02C was left with a toe force much larger than that in the other two test piles.

The approximately linear shape of the force distributions indicate that the unit shaft resistance did not increase with depth along with the increasing effective overburden stress despite the fact that the soil resistances expressed by both q_c-resistances and N-indices increased with depth, as illustrated in Figure 12. This disparity is due to the pile being affected by presence of residual force. The residual force is considered due to the ongoing settlement caused by the recent placement of fill across the site having affected the test piles between construction and testing. The dashed line in the figure starting from zero force shows a potential distribution of the residual force. This distribution added to the distribution for the maximum load (the dashed line starting from the maximum load) indicates the potential true force distribution.

The force distributions of the other two tests indicated a similar presence of residual force. The back-calculated force distributions of all three tests show the same true distribution (same beta-coefficients at target load distribution) for all three tests, but with different toe responses and different distributions of residual force. Figure 13 shows the calculated residual force and the true force distributions with the target distributions for the three tests. The shaft resistance at the target load is the same for all three test piles.

With the adjustment to residual force, the back-calculated plastic shaft resistances corresponding to the target force distribution correlated to beta-coefficients of 0.40 in the upper silt and sand, 0.25 in the compressible clay and silt layer, 0.30 in the underlying silt, and 0.60 in the dense silty sand near the pile toe level. The toe response varied between the test piles as a function of the induced pile toe movement. The toe force followed a Gwizdala *q-z* function with a function coefficient of 0.50, e.g., a toe movement of 25 mm required a toe-stress of about 10 MPa and a toe force of about 1,700 kN (Gwizdala 1996, Fellenius 2023).

The Original and Final Designs

The original design applied the 2018 International Building Code, New Jersey edition. "Capacity" was estimated from the pile-head load-movement curve and shaft resistance was back-calculated from the instrumentation records to determine the drag force without consideration of residual force.

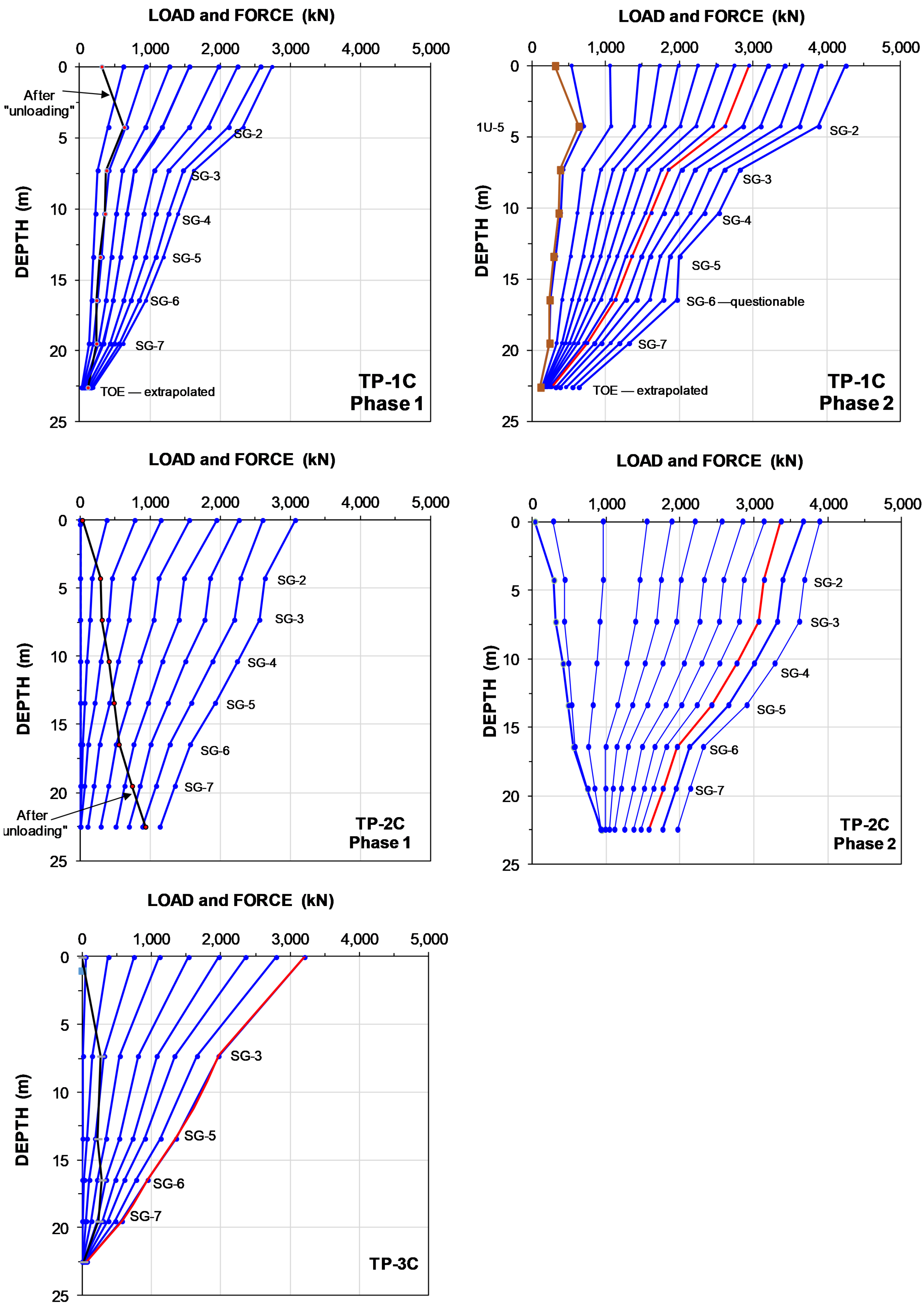

Figure 11. Load-time and load-movement records of the static loading tests

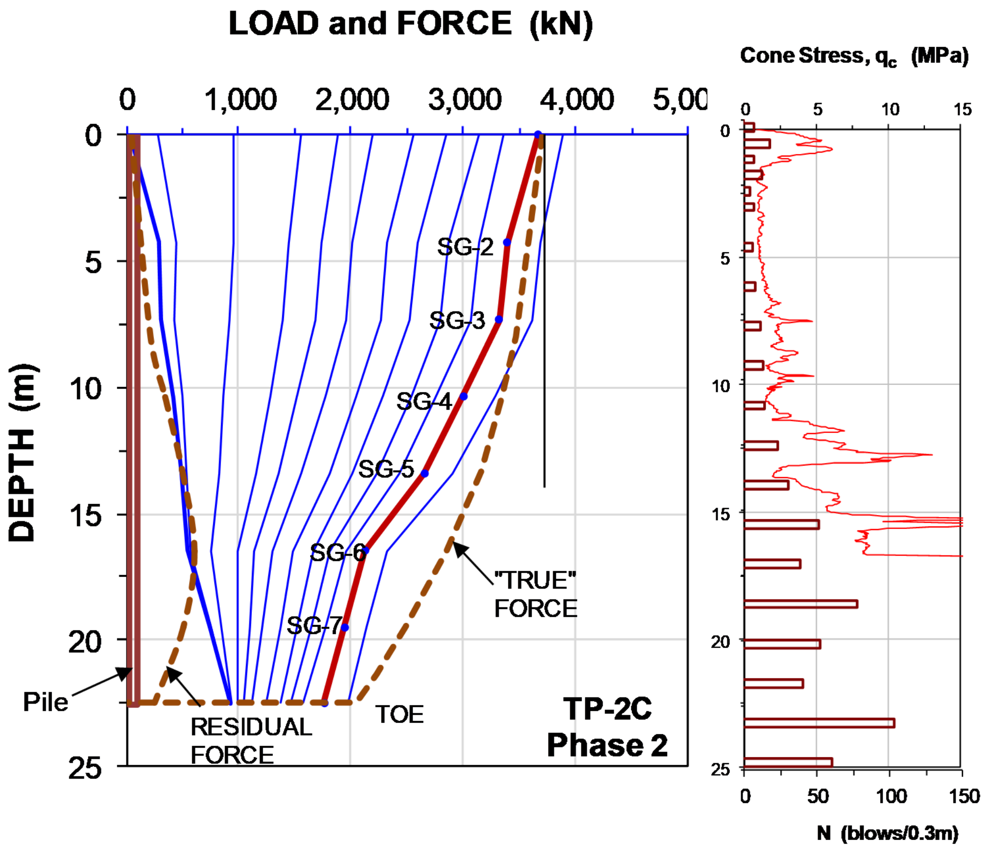

Figure 12. Force distributions in Test TP-2C and distributions of q_c-resistance and N-index

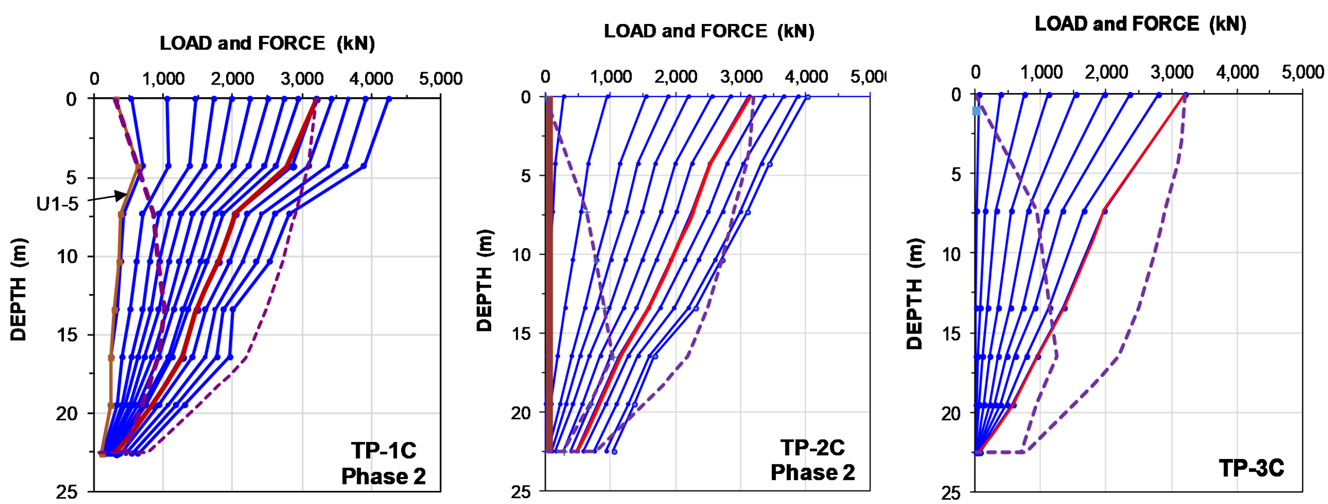

Figure 13. Measured and residual force distributions in all three test piles (the red lines are target force distributions used for back-analysis and for determining the residual force distribution)

Omitting the effect of residual force will always result in an overestimation of shaft resistance and drag force.

To make matters worse, the original design only considered the "capacity" to be obtained from below the neutral plane. This was caused by a misinterpretation of the building code provision stating "*Deep foundation elements shall develop ultimate load capacities of not less than twice the design working loads in the designated load-bearing layers*". The original design placed the neutral plane at what the code refers to as the "load-bearing layer" and required a resistance equal to twice the design load plus the drag force to be obtained from below this depth. This not only mistakenly

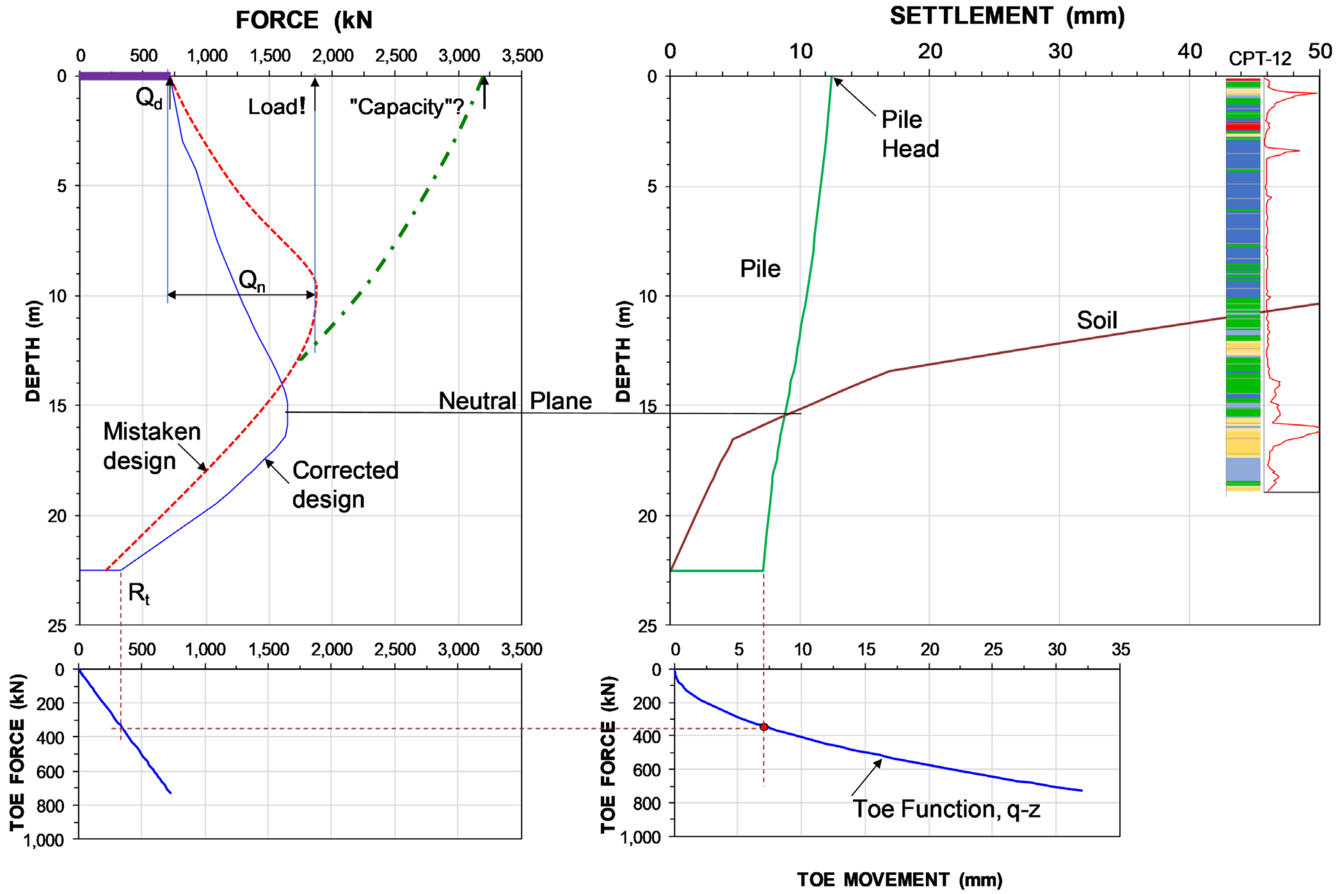

Figure 14. Original and final design approaches

included the drag force with what the code referred to as the "working load", but it also confused the limit state analysis with the strength state analysis.

The erroneously overestimated drag force combined with the code misinterpretations generated a "working load" comprising sustained load, transient load, and drag force, requiring a lengthening of the piles (approximately 35 %), which would have considerably increased the costs of the foundations.

The original and final design approaches typical for the project piles at the original pile length are indicated in Figure 14 together with the force distributions for the sustained load (here Pile TP-1C). The "mistaken design" and the "capacity" are approximately what was estimated in the original design from the then back-calculated force distribution for the test piles. The estimated pile head (foundation) settlement is well smaller than the 35-mm value stated acceptable for the structure. Thus, the displayed conditions for the final design were satisfactory for the structure.

Lumping live load and drag force together was such an obvious mistake that it was easy to correct. It was harder to correct the error made by assuming that the drag force could act at the same time as the "capacity" and even more so, the confusion caused by the misinterpretations of the building code. However, the analysis displaying the presence of residual force showed that the shaft resistance in the uppermost layers was smaller than first deduced. Thus, the drag force was shown to be correspondingly smaller than first determined from the original back-analysis resulting in the "capacity" being sufficient for the load. The back-analysis of the test results confirmed that the originally assumed pile length was acceptable—and substantial costs and construction time were saved.

The corrected design also removed an, as it were, non-consequential error. Settlement calculations for the location of the neutral plane, mistakenly high up in the soil, would have resulted in significant downdrag. However, no settlement analysis was included for the foundations so this consequence was not discovered. Analysis applying the deeper and correct location of the neutral plane showed that downdrag would not be of concern for the piled foundations.

Closure

The three full-scale tests showed that the piled-foundation design gave satisfactory results in that the long-term settlement was well below the stated limiting value of about 35 mm. The shaft resistance values adjusted for residual force were similar to values usually observed for soils of the subject types. Careful analysis of the test results in regard to downdrag and residual force combined with proper application of the building code enabled correction of mistakes in the original design and avoided extensive increase of construction costs.

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