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ASSESSING AND BACK-ANALYZING A BIDIRECTIONAL TEST ON A CFA PILE

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ABSTRACT

A bidirectional static loading test was performed on a 17.4 m (57 ft) long, strain-gage instrumented Continuous Flight Auger (CFA) pile in a soil profile consisting of a recent fill over natural soils composed of mixtures of sand and fines. The nominal pile diameter was 864 mm (34 in). The strain-gage measurements were evaluated to establish an appropriate EA-parameter for conversion of strain to axial force. The resulting axial force distributions and measured force-movement relationships were then fitted in UniPile6 back-analysis to obtain t-z/q-z functions representing the pile response. The fitting established the shear-force vs. shear-movement of the pile elements for the range of applied load and enabled simulating the test results. The shaft resistance response was significantly strain-hardening. The functions were used to convert the bidirectional test to an equivalent head-down test including simulating the distribution of axial force and the load-movement response of the head-down test. Differences in resistance mobilization and potential residual-force distribution between bidirectional and head-down loading were explained. The analysis indicated that no residual force was present in the test pile.

Keywords: bidirectional testing, continuous flight auger pile, EA-parameter

INTRODUCTION

Piled foundation projects often include a static loading test carried out either before construction commencement to allow for changes in the design, if found warranted, or later to confirm that the foundation performance would be consistent with the design assumptions. For either purpose, it is desirable to know the distribution of resistance along the test pile. However, a routine head-down test says very little about this, which is why instrumented test piles are becoming more frequent.

It is of course important to have reliable instrumentation to measure the imposed strain, but converting the strain to force requires a reliable conversion factor, the EA-parameter. The parameter cannot reliably be determined from assumed pile area, A, and E-modulus derived from a concrete strength relation, but must be obtained from analysis of the test records, which requires tests employing equal size increments and equal load-holding durations, excluding unloading-reloading events. Moreover, it is not always possible to reliably determine if the force in the pile, as determined from the strain-gage records, is affected by presence of residual force in the pile. Particularly so for records obtained from a head-down test. A bidirectional test will make it easier to assess the extent of potential presence of residual force in a test pile, because the load applied at the bidirectional (BD) level is a true load that is unaffected by residual force. This allows the back-analysis of the test records to determine presence and distribution of potential residual force in the pile and develop a reliable interpretation of the true pile response. This would, for example, lead to more representative test results and facilitate implementing changes to a design before construction.

The following case history from a recent project in Ohio illustrates the process of analyzing the results of a strain-gage-instrumented bidirectional test pile, including determination of the EA-parameter, calibration of t-z/q-z functions, and evaluation of potential residual force.

PILE DETAILS AND SOIL PROFILE

The project foundations consisted of rows of piles connected by grade beams. The overriding design demand was an uplift load (unfactored) of about 1,220 kN (275 kips). The test pile was a 17.4 m (57 ft) long, nominally 864 mm (34 inch) diameter, strain-gage instrumented CFA pile, constructed on

October 30, 2025. A bidirectional (BD) static loading test was carried out on November 5, six days later.

Seven boreholes were drilled in October 2024 near the test pile. The soil profile comprised about 3 m of recent fill over a natural soil composed of a mixture of fines and sand. The fill was placed about a month or two before constructing the test pile. Figure 1 shows the soil profile with water content and N-index diagrams, compiled from seven boreholes drilled within about 5 m radius from the test pile, and depths of BD cell and strain-gage pairs. In contrast, the water content profile indicated uniform soil conditions. The water contents corresponded to bulk densities of about 2,000 kg/m^3 in the fill, 1,900 kg/m^3 to 10 m depth and 2,000 kg/m^3 below. The water levels encountered in the boreholes varied, but the groundwater table was stated to be at 3 m depth. The pore pressure distribution was assumed to be hydrostatic. The SPT N-indices indicated a rather scattered distribution of relative density: on average, compact in the fill and above 10 m depth, underlain by dense material that, on average, became denser with depth.

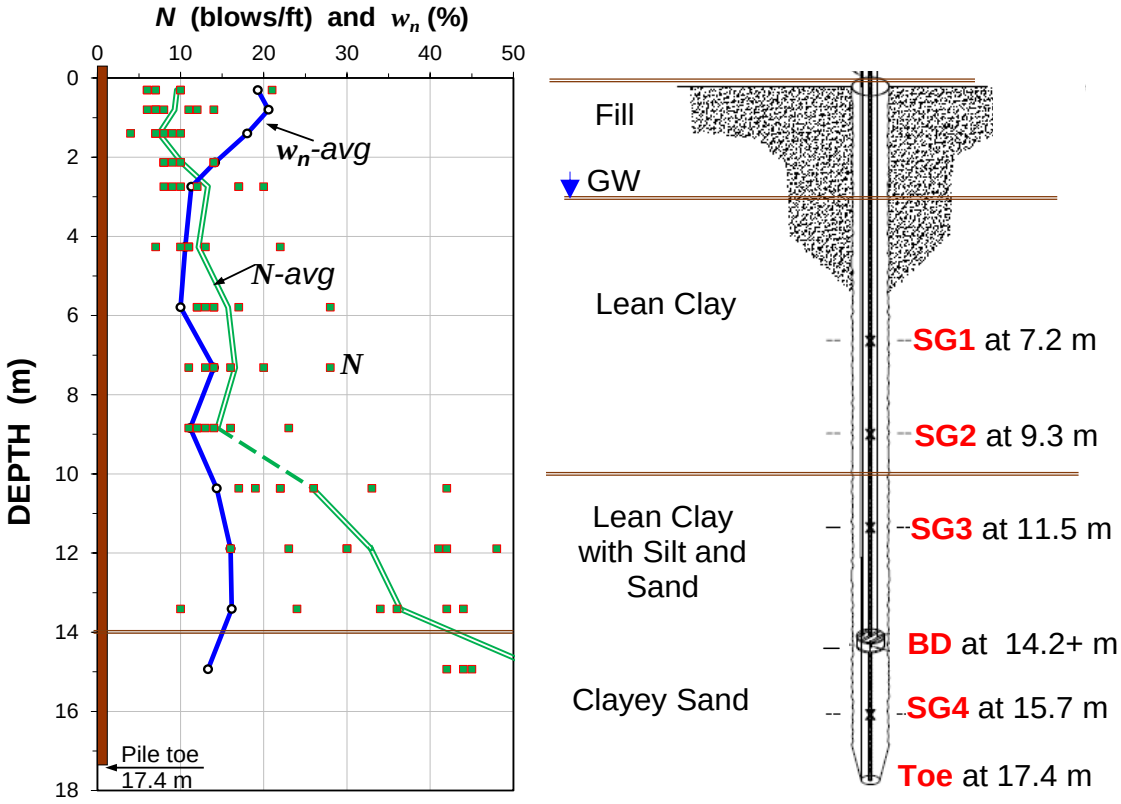

Fig. 1. Soil profile and pile schematics

The recently placed fill will cause the natural soil to settle over time considering the dominantly fine-grained soil profile. Because the pile was installed while settlement was actively developing, negative skin friction and associated drag force, that is, residual force in the pile, was anticipated. However, the compressibility was low: Janbu modulus number, m (determined from the oedometer tests contained in the soil report) was larger than 100. Therefore, downdrag would be insignificant, as would also any associated residual force.

BIDIRECTIONAL TEST

Test procedure and measured movements

The bidirectional test was carried out in 21 equal increments of 150 kN applied every 10 minutes. The reported maximum weld-breaking force was 370 kN (83 kip), exceeding the magnitude of the

scheduled second load increment. Therefore, instead of unloading and starting the test from zero load, the test proceeded directly to load increment L3, 474 kN (106 kips), as the first test load. The full sequence of the initial weld-breaking records was not included in the test report.

Figure 2 shows the applied BD load versus measured strain, with the recorded upward applied load reduced by the 170-kN (38 kip) pile buoyancy. The upward movement of the pile head and the downward movement of the pile toe are also shown. The compression of the pile was small relative to the measured movements.

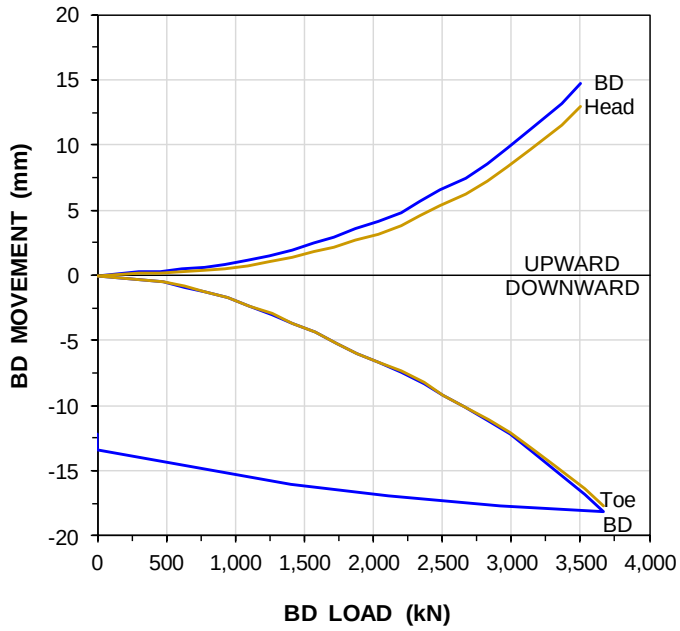

Fig. 2. Upward and downward BD load-movement curves with head and toe curves

For the subject test, the BD expansion, pile compression, and pile-head movement were direct measurements. Upward and downward BD-movement and pile-toe movement were derived quantities: Upward BD movement was calculated by adding measured upward segment compression from measured pile-head movement. (The latter was not the conventional measurement relative to a reference beam, but measured via a precision surveying instrument). The downward BD movement was the upward BD-movement minus the BD-expansion. Pile-toe movement was the downward BD-movement minus the measured downward compression obtained from a telltale extended to the pile toe.

Figure 3 compares the telltale-measured compression to that calculated from accumulated average of measured strain over distance between gage levels. The measured compression over both upward and downward segments would be expected to exhibit a more or less linear relation. Instead, the graph shows an initial somewhat stiff linear response shifting to a softer also linear response, which could be due to friction between the telltale rod and the guide-pipe. On that basis, the reported BD movements may be about 0.5 mm smaller than actual values at the maximum applied load. The 0.5 mm Rr is small in relation to the reported BD movements, which were on average about 10 mm. The figure is supplemented with the load-strain curves determined from t-z/q-z curve fitted to the test records in UniPile6 (marked “unp”), addressed later.

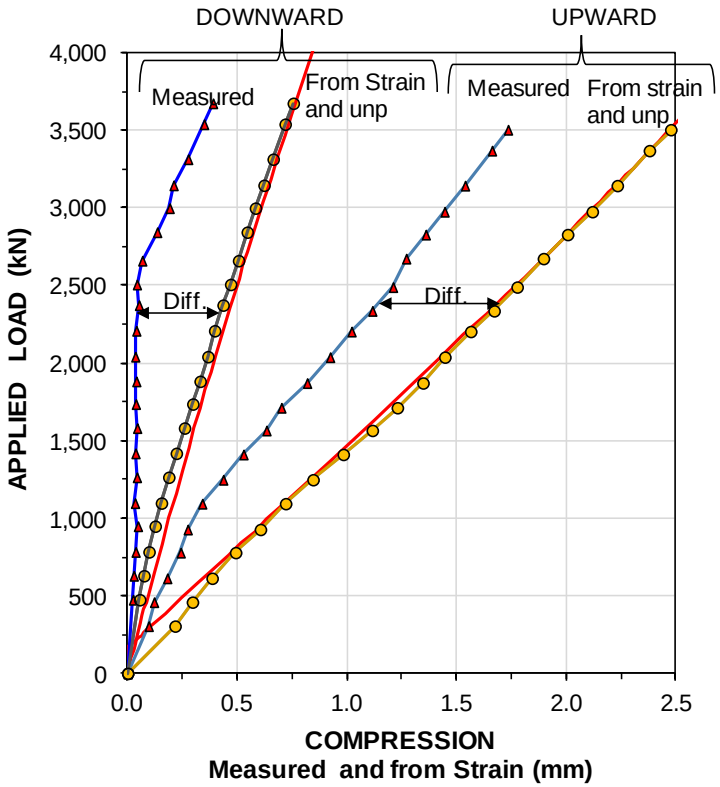

Fig. 3. Applied load versus measured and calculated compression (The "unp" stands for "calculation by means of UniPile")

The EA-parameter

Determining the force at the gage locations requires knowing the Young's modulus, E, of the pile and the pile cross sectional area, A; the combination is the EA-parameter. The force at a gage level is equal to EA times the measured strain. Thermal Integrity Profiling (TIP) was performed on the test pile and the results indicated a consistent diameter distribution. The nominal diameter was 864 mm (34 in), but the TIP results indicated that the actual diameter was approximately 1,020 mm (40 in) at the pile head (ground surface) reducing linearly to about 940 mm (37 in) at the pile toe. The diameter at SG1 through SG3 was about 940 mm (37 in), corresponding to a cross-sectional area approximately 18% greater than the nominal value.

The EA derived from the records of applied load and resulting strain recorded at the gage levels may be affected by shaft shear. If the soil layer between the considered gage level and the pile head would be strain-hardening or strain-softening, rather than plastic, the EA-parameter will be overestimated or underestimated, respectively. Therefore, the gage level suitable for determining the EA-parameter is the gage level close to the BD cell where no or minimal shaft resistance is mobilized between the gage and the cell. Moreover, a gage located one or more gage levels further away from the load source will also be affected by the fact that the force reaching the gage will have been reduced by the intermediate resistance. Presence of residual force may further affect the calculated EA-parameter.

Figure 4 plots the applied BD load (upward and downward) versus strain measured at SG1 through SG4. The red line represents an approximate EA-parameter slope. The steepening slopes of the SG1 and SG2 curves, beginning at about 110 με, might indicate a shaft-resistance strain-hardening response. The graph suggests an approximate EA-parameter of 12 GN for all four gage records by the slope method.

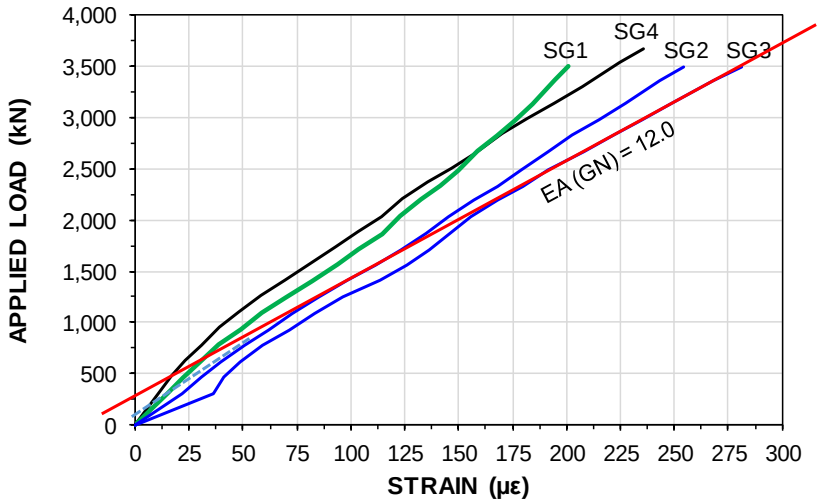

Fig. 4. Load-strain curves and EA-parameter line

The tangent EA-method is a more precise method for determining the EA-parameter than the slope method. The tangent method assesses increment of load divided by increment of strain plotted versus strain. It is based on assuming that, once the shaft resistance between the load application point (here, the BD cell) and the gage level has become fully mobilized (i.e., reached a plastic state), the EA-plot would show a constant EA-value for additional increase of strain. Because the method is based on differentiation, small imprecision of the numbers inflates differences in the EA-values and, therefore, even small differences can result in significant scatter and deviation from the correct value. By calculating EA as a running average of a few records, the display becomes smoother, enabling a better decision on the EA-parameter than one based on a plot of single values.

Figure 5 shows the calculated EA-parameters determined as a running geometric mean of three records. The dashed blue curve is plotted from the not smoothed SG2 records, demonstrating the need for smoothing the data. The apparent increase in EA at gages farther from the BD cell does not indicate a change in pile stiffness, but reflects that the additional shaft resistance between the BD cell and the gage due to strain-hardening.

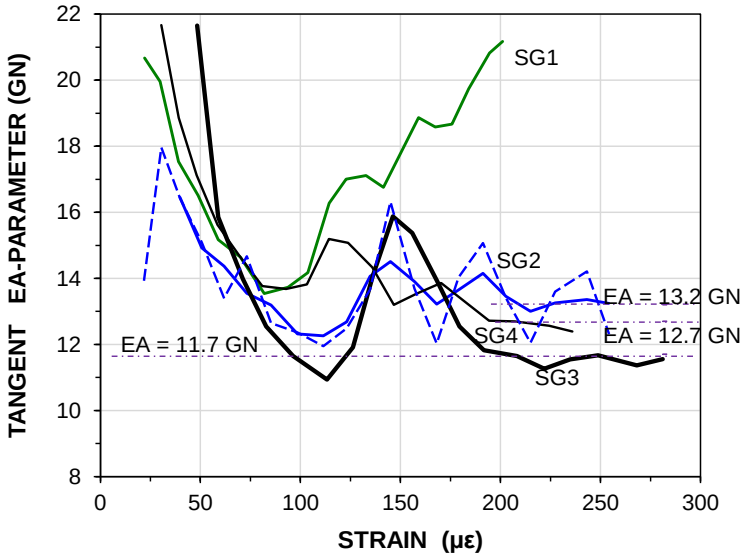

Fig. 5. Tangent EA-parameter versus strain

The data show a change of trend in the EA-values for SG3 (closest to the BD) between strain records 125 - 200 με: increasing and then decreasing. The reason is not known. Unless the soil between the BD and SG3 is particularly strain-hardening, the SG3 EA-parameter of 11.7 GN, rounded to 12 GN, can be considered representative of the pile EA at SG3. The EA-parameters indicated for SG2 and SG4 are larger. The reason for this could be that the soil shaft resistance along the pile is strain-hardening. The uniformity of the pile justifies applying the 12-GN value also to the records of SG1 and SG2.

An EA-parameter of 12 GN for a 940 mm diameter pile corresponds to an E-modulus of 17 GPa. Calculated for the nominal diameter of 864 mm, the E-modulus becomes 20 GPa, which is still rather low. The low values are likely due to the young age (six days) of the concrete. CFA piles 30 days after casting would usually be expected to have an E-modulus at least in the range of 25 through 35 GPa.

Strain-derived force response

Figure 6 shows BD load and forces (determined as measured strain times EA = 12 GN) versus BD-movement, upward and downward, respectively. The records appear consistent. The maximum pile movement relative to the surrounding soil exceeded 10 mm at all gage levels. The maximum measured compression (accumulated strain-gage data) was about 2 mm.

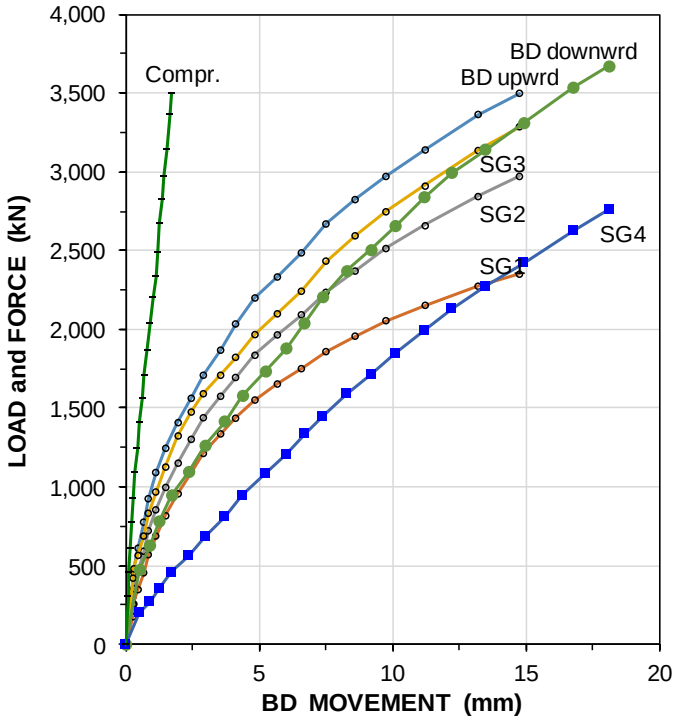

Fig. 6. Applied BD load and measured force versus BD-movement

Figure 7 shows the axial force distribution for the applied loads for every second load applied, L3 through L23. The small horizontal offset at the BD level is a result of adjusting for shaft buoyancy in the upward BD load. The indicated two pile-toe forces are extrapolated with a small allowance for shaft resistance increasing with depth.

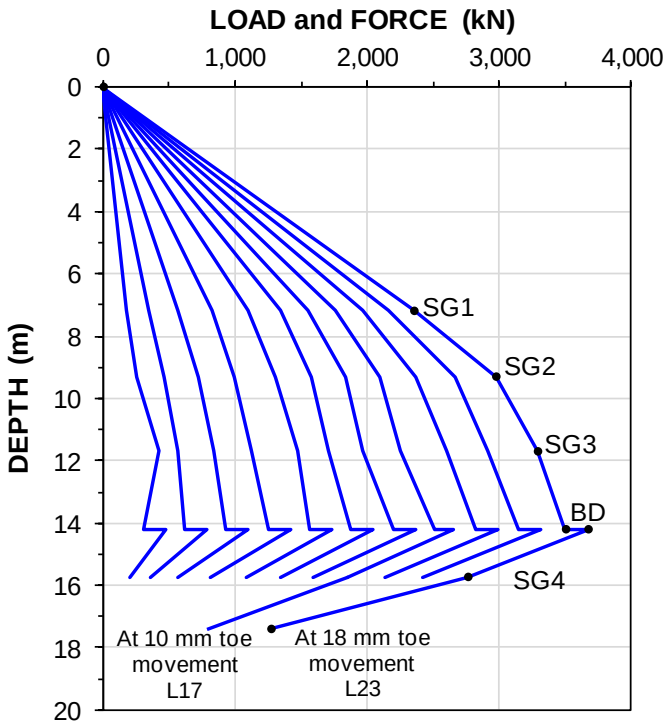

Fig. 7. Load and force distributions

BACK-ANALYSIS OF THE TEST RECORDS

The back-analysis of the test records starts by choosing a target ß-coefficient ($ß_{trg}$), target movement (δ_{trg}) and target *t-z* function for each distinct soil layer or pile length between gage levels, i.e., pile element. Each pair of target ß-coefficient and target movement is a point on the *t-z*/*q-z* curve for the pile element. This, along with the pile and soil particulars, is all input necessary for an analysis simulating the load-movement of the pile element. The process is iterative, keeping the target movement while adjusting the target ß-coefficient, *t-z* function, and function coefficient until the simulated force-movement curves match the measured response.

The calculated fitted gage force-movement records above and below the BD level are independent. For the above BD fit, the upward response, the pile element between the pile head and SG1 was addressed first. Then, the fit to the SG2, SG3, and BD curves were addressed in turn. For the below BD fit, the downward response, the back-analysis started with simulating the downward SG4-curve, which also addressed the pile toe response.

Figure 8 shows the final fit of the simulated to the measured load- and force-movement curves of the subject test pile performed using the UniPile6 software (unisoftGS.com). The curve marked "TOE" includes the two extrapolated toe force values plotted versus toe movement.

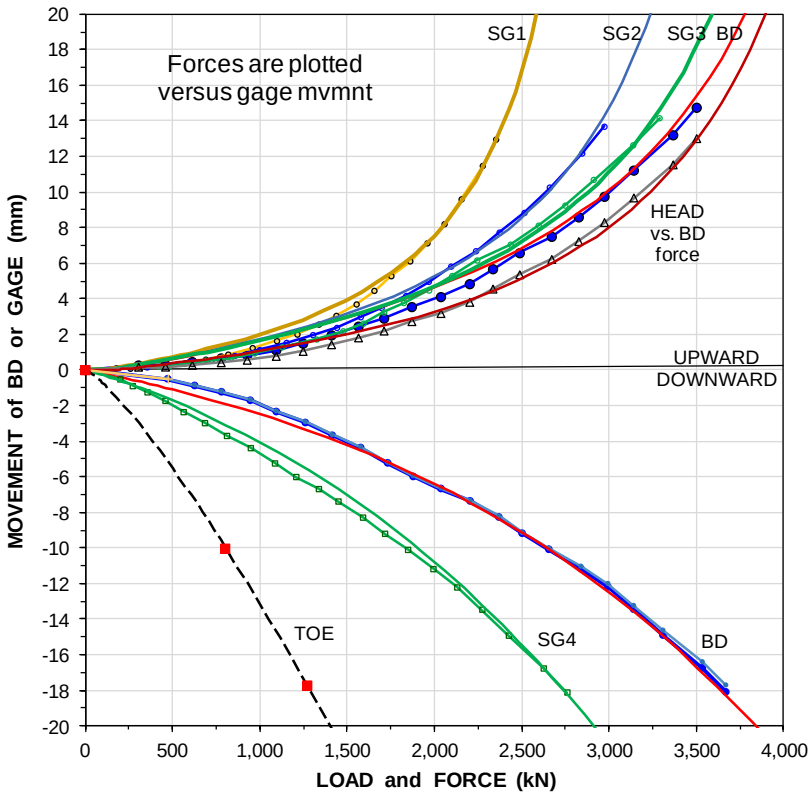

Fig. 8. Measured and simulated (fitted) load and force movements

Table 1 shows final target ß- and δ-input and C_1-coefficients (hyperbolic t-z function coefficient) for the shaft resistances used to achieve the fitting in UniPile6. The numbers (1) through (6) represent the soil layers defined by the SG depths, starting with #1 for the fill. If the groundwater table would be assumed at, say, 7 m depth, the evaluated ß-coefficients reduce. In Lauer #1, to about 0.8, a more realistic value, but still large. The Layer #4 ß-coefficient of 0.05 at 5-mm movement is small for a clay with a water content of 10 to 15 %. All *t-z* function coefficients indicate strain-hardening shaft shear.

Table 1. Shaft input

Layer function #)	Depth Interval (m)	Target, δ_{trq} (mm)	Target, $ß_{trg}$	Function Coeff (--).	t-z/q-z (name)
1	0.0 - 7.2 head - SG1	5	1.40	0.0054	Chin-Kondner
2	7.2 - 9.3 SG1 - SG2	5	0.45	0.0030	Chin-Kondner
3	9.3 -11.5 SG2 - SG3	5	0.20	0.0030	Chin-Kondner
4	11.5 -14.2 SG3 - BD	5	0.05	0.0040	Chin-Kondner
5	14.2 -15.2 BD - SG4	5	0.70	0.0045	Chin-Kondner
6	15.2 -17.4 SG4 - Toe	5	0.77	0.0030	Chin-Kondner
6	17.4 Toe	5	r_t = 600 kPa	0.80	Gwizdala

Figure 9a shows the target ß and δ and associated simulated and fitted *t-z* function. All ß and δ target pairs represent points on the respective *t-z* function. The function coefficient determines the shape. All functions represent significant strain-hardening with increasing movement. For comparison, the figure includes a *t-z* function (curve (0)) with a more common coefficient, C_1, equal to 0.90, representing an almost plastic response beyond the target movement. Figure 9b shows the simulated and fitted toe response, which was almost linear. The target *q-z* input was for a target movement, δ_{trg}, of 5 mm and a Gwizdala toe function with a coefficient, θ, of 0.80.

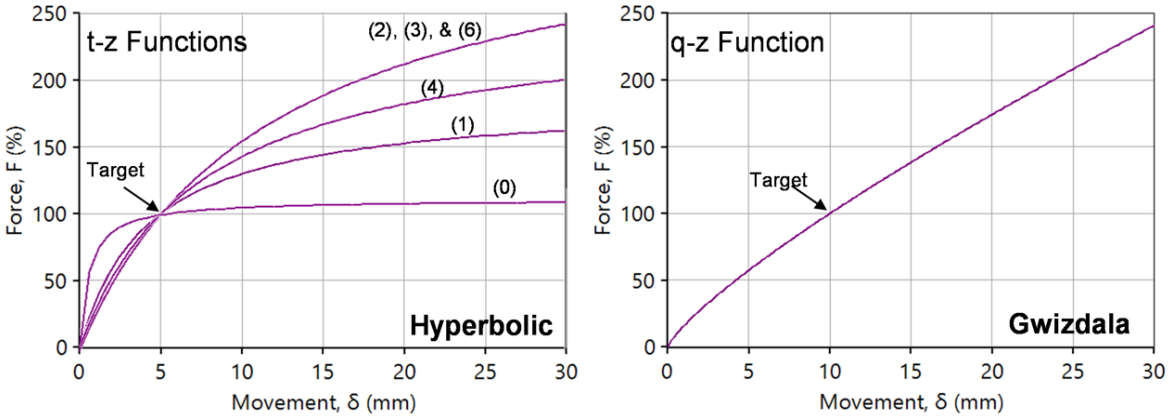

Fig. 9. t-z and q-z functions used in the final fitting

EQUIVALENT HEAD-DOWN RESPONSE

For comparing the results of the bidirectional test to those of a conventional head-down test, the bidirectional test data can be converted to show the equivalent head-down response of the test pile. Figure 10 shows the UniPile6 simulation results for the equivalent head-down curve for the pile employing the parameters obtained in the fitting process. The shaft-resistance curve indicates that the design uplift load of approximately 1,200 kN would produce only minimal upward movement.

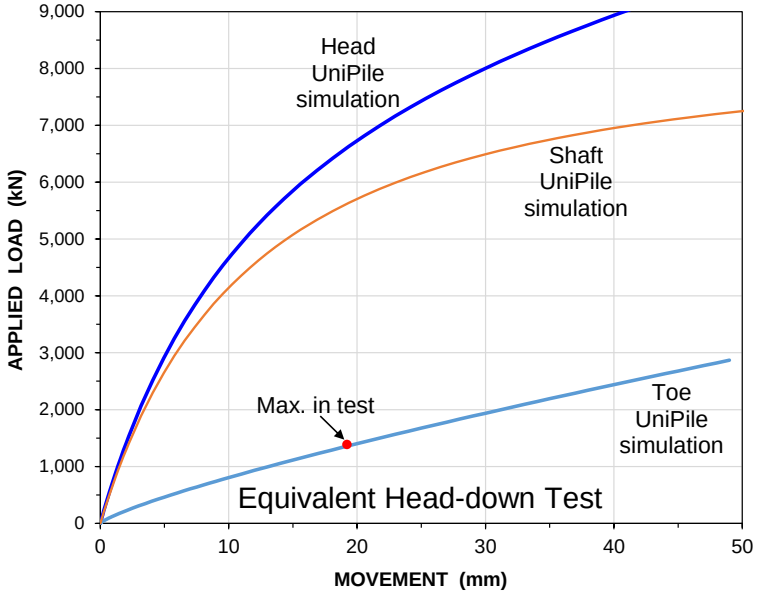

Fig. 10. Equivalent head-down test

POTENTIAL RESIDUAL FORCE

Once all parameters were determined in the fitting process, UniPile6 was used to determine the equivalent force distribution for any load applied to the pile head, as in a head-down test. Figure 11 shows the measured force distribution (1) measured for load increment L17 and the corresponding force distribution for an equivalent head-down test load that generates a force at the BD equal to the load and downward movement of the L17 load level (2,657 kN and 10 mm). At this load, the BD upward pile-head movement was 5 mm. The equivalent head-down test required an applied load of 5,875 kN, which is 580 kN larger than twice the BD-test force due to the fact that larger movement was required to overcome the shaft resistance above the BD level before reaching the BD load. The downward force distribution curve (2) is, therefore, not the curve "flipped over", i.e., mirrored, from

the BD curve, but is calculated from the test records by means of UniPile6 with input of the derived parameters.

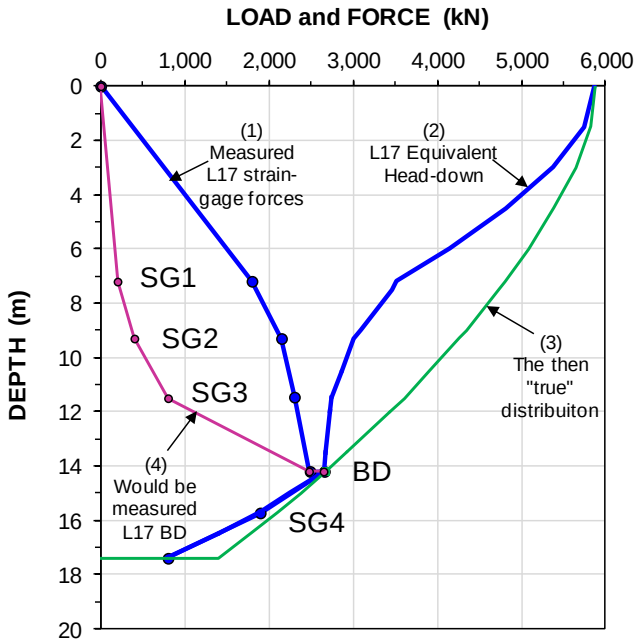

Fig. 11. BD-force distribution for L17 and its equivalent head-down distribution and distributions corrected for hypothetical residual force

The equivalent head-down distribution curve (2) appears to signal that the test pile is subjected to a considerable residual force: large shaft resistance in the upper layer diminishing to very small to insignificant at deeper depth. For both a bidirectional test and a head-down test of an instrumented pile, the measured force distribution includes residual force, if present. (Again, the measured applied BD load is a "true" load unaffected by presence of residual force). Were residual force present in the pile, then, the "true" distribution of the head-down test would be expressed by Curve (3), showing the distribution that would occur after the applied load had overcome the downward-directed shear forces associated with negative skin friction and the curve would show the subsequent effect of mobilizing positive shaft resistance. Curve (4) would then show how in the BD test only the distribution of shaft shear mobilized in addition to the already present negative skin friction.

So, if Curve (3) represents the "true" force distribution in the pile, then, the bidirectional test would not have measured the force distribution shown by Curve (1), but one shown by Curve (4). As Curves (1) and (4) are very far apart, the conclusion must be that there is no residual force present in the test pile and the measured force distribution (1) is representative for the "true" static resistance of the test pile. Thus, somewhat unexpectedly considering the distribution of N-indices shown in Fig. 1, the test results showed the resistance distributions (Fig. 12) to indicate large shaft resistance in the upper soil layers (above 8 m depth).

The stiffer response in the upper portion of the pile is not what one would expect from the subsurface information (Fig. 1). A larger pile diameter in the upper layers could explain the stiffer response; however, there is no independent evidence confirming that condition and it conflicts with the results of the Thermal Integrity Profiling. The difference is also not a consequence of the selected strain-hardening *t-z* functions. Analyses applying a *ß*-coefficient distribution fitted to each applied load, foregoing a *t-z* analysis, produces a similar indication of stiffer response within the upper 8 m of the profile.

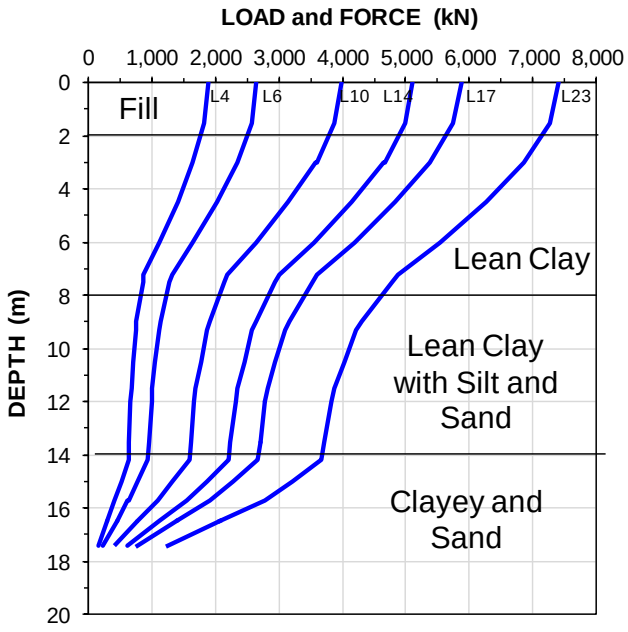

Fig. 12. Force distributions for the equivalent head-down test

CONCLUSIONS

Based on the bidirectional (BD) loading test, interpretation of the strain-gage records, and UniPile6 back-analysis of the instrumented CFA test pile, the following conclusions were drawn:

1. Determination of a representative EA-parameter was a necessary first step in converting measured strain to axial force. In the present case, the young age of the concrete and the strain-hardening shaft response made it difficult to identify a unique EA-value from the gage records. An EA-value of 11.7 GN was judged to provide the most reasonable basis for the subsequent analysis.
2. Iterative fitting of measured load-movement and gage-force responses using t-z and q-z functions in UniPile6 provided a practical means of fitting a theoretical axial-force distribution to what was measured, thus "calibrating" the analytical model.
3. This case history demonstrates the importance of evaluating potential residual force, even when the analysis ultimately shows that none is present.
4. The stiffer response in the about 8 m thick uppermost soil layer of the pile was inconsistent with the trend indicated by the SPT-indices.
5. Overall, the analysis demonstrated that back-analysis of the test result based on fitting the data to *t-z/q-z* functions enabled confident re-assessment of small changes to the design. Such analysis can significantly improve interpretation of loading tests on instrumented piles and lead to more economical foundation design.

ACKNOWLEDGMENT

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