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Subject: Static loading tests in Mobile, AL
Straight and tapered piles (TSFP)
Final Report

This letter reports the results of the April 2025 full-scale static loading tests in Mobile, Alabama, comparing the response of three taper steel pipe piles (TSFP) to that of two piles with uniform cross section. The information and data herein are taken from the report by Scientific Applied Concepts Limited (SACL), Ontario "Pile Loading Test Program, Conventional Static, Bidirectional, and PDA, AL-DOT Site, Mobile, Alabama", dated May 29, 2025.

All piles, TP1 - TP5, were concrete-filled, closed-toe, pipe piles with 3/8-inch (9.5 mm) wall, a steel area, A_{steel}, of 169 cm^2 (26 in^2) driven to a 57-ft (17.4 m) predetermined depth. Piles TP1 and TP2 had 18-inch (457 mm) diameter and Piles TP3 - TP5 were TSFP piles, i.e., same 18-inch size pipe down to 25 ft (7.6 m) above the pile toe where the section tapered to 8-inch (203 mm) toe diameter.

SUMMARY

The piles were strain-gage instrumented and driven on March 26 (TP1 and TP3) and 27 (TP2, TP4, and TP5), restruck on March 28(RST1), and April 29 (RSTR2), 2025. SACL performed dynamic monitoring (PDA) followed by CAPWAP analysis of blow records from both initial driving and restrikes. The piles were driven by Jordan Pile Driving Inc., Alabama, who also concreted the piles and set up the kentledge-type static loading tests. The piles were strain-gage instrumented by SACL, who also carried out the static loading tests.

The site is a paved parking lot close to the Mobile River at the corner of Dunlap Dr. and Austal Way, Mobile, AL. But for an about 2-m thick zone of loose gravelly sand with 30 % fines content between 15 and 21 ft (4.6 and 6.4 m) depth, the soil consists to 90 % of sand size grains, and the density is 115 pcf (1,850 kg/m^3). The density over and below this zone is 128 pcf (2,050 kg/m^3) and the consistency is compact to about 21 ft (6.4 m) depth, loose to about 54 ft (16.4 m), and, then, dense. When the static loading tests were carried out, the groundwater table (GW) was at 16 ft (5 m) depth. Figure 1 shows results of a CPT sounding performed at the site. The q_t-graph is supplemented with the distribution of SPT N-indices.

Figure 2 shows the resulting load-movement curves (records are from the end of each load increment). Labels TP1 and TP2 denote straight piles and labels TP3 - TP5 denote taper piles (lower 25 ft linearly reduced from 18-in to 8-in diameter). The test on Pile TP3, was disrupted and prematurely terminated when the kentledge weight showed to be insufficient (platform lifted off when adding the next increment to the 2,550 kN load at 13 mm movement; the records after that occurred are left out in the graph. For full records, see Figure 6B). The curves show that, for the same movement, the movement being larger than about 5 mm, the taper piles carried close to 20% more load than the straight piles.

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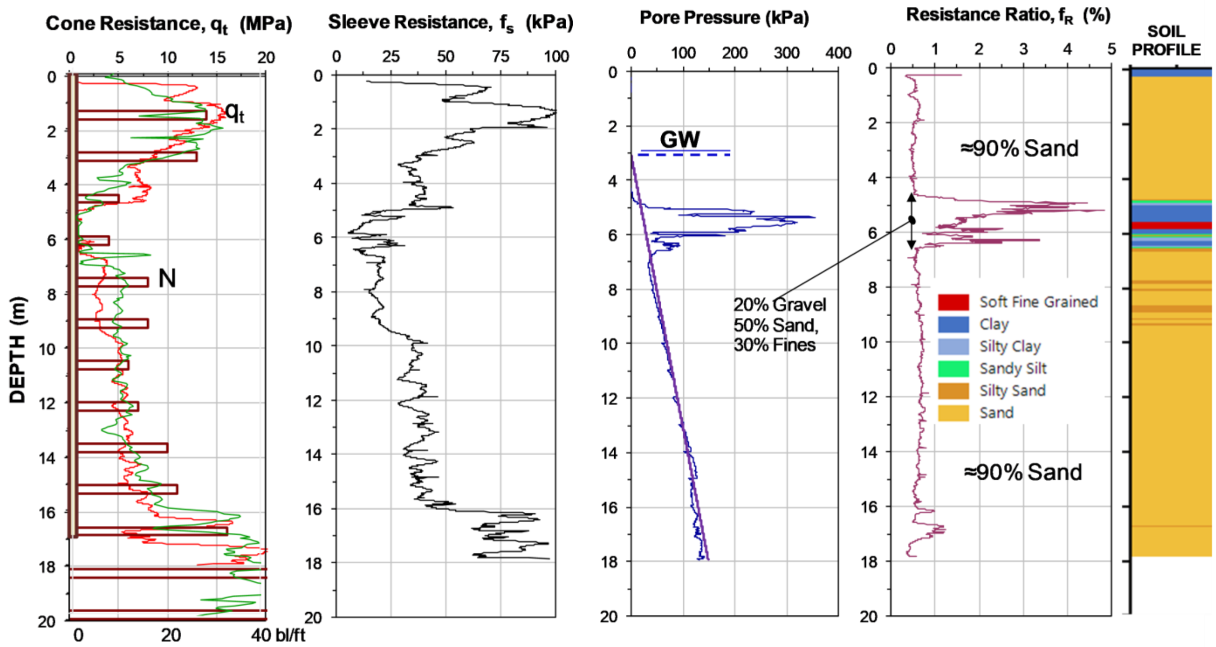


Fig. 1 Soil profile by CPT and SPT records

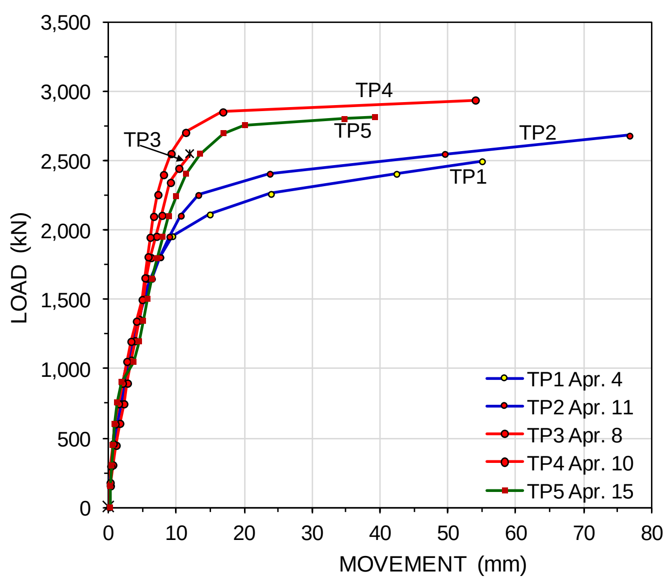


Fig. 2 Load-movements for the five test piles

INSTRUMENTATION

All piles were instrumented by means of vibrating-wire strain-gages Type Geovan Model GV-2410. The gages were placed on a rebar cage comprising four #5 bars (15.9 mm) held together with about 200 mm diameter rings spaced 2.0 m apart and equipped with spacers (bracings) to center the cage in the pile. The gages were placed as one or two diametrically opposed pairs at five levels in Piles TP1 -TP4 and six levels in Pile TP5 as indicated in Table 1 for depths below the ground surface. (The gage depths shown for Pile TP5 were amended after field adjustment to placement difficulties). The two most important gage levels are the uppermost level (which were to serve as calibration gages for determining the EA-parameter of the pile cross section) and the pile-toe gage, which determines the pile-toe force (relying on the EA-calibration).

Table 1. Gage depths and pair numbers

	TP1 - TP4		TP5	
Gage Level	Depth (m)	No. of Pairs	Depth (m)	No of (Pairs)
SG6			0.52	2
SG5	0.42	2	6.42	1
SG4	6.42	1	8.27	2
SG3	9.41	2	9.82	2
SG2	14.42	1	14.07	1
SG1	16.89	2	14.62	2

Pile TP5 was scheduled to include a bidirectional cell (hydraulic jack) placed just above at the transition between the straight and the tapered sections. The purpose of the bidirectional test on TP5 was to test for potential locked-in residual force. However, as described further on, the concreting operation adversely affected both the bidirectional cell and strain-gage instrumentation in TP5 and the bidirectional test; the latter to the point that it could not be made.

Each instrumentation cage included a pair of a machined telltale rods (6 mm diameter with internal couplings) to enable measuring the full-length pile compression, installed in guide pipes (25 mm steel pipe; 6 mm wall and with external couplings). Pile TP5 had also a telltale pair to the bidirectional cell upper plate and two extensometers to measure the cell opening. The telltale rods were inserted in the guide pipe before placing dial gages and connections to the data collector. As is practice, there was no oil in the guide pipes to reduce friction between pipe wall and rod.

PILE DRIVING, DYNAMIC MONITORING, and PILE PREPARATION

The piles were driven using an APE D30-32 open-end diesel hammer, with a rated energy of 69.9 kip-ft (94.8 kJ), to a predetermined depth of 57 ft (17.4 m) below grade. As mentioned, Piles TP1 and TP3 were driven on March 26 and Piles TP2, TP4, and TP5 on March 27, 2025. The hammer transferred energy was about equal for the piles; the average transferred energy was about 16 kJ. The hammer fuel setting was not noted to the records. Figure 3 compiles the distribution of the measured maximum impact force and the driving log diagram of the five test piles, showing the penetration resistance (blows/0.3 m. The hammer force was consistent, slightly larger for the taper piles as opposed to the uniform piles and the two taper piles required 50 % larger number of blows to reach the prescribed depth. The observations of impact force and number of blows are indicative of a larger soil resistance to the pile penetration.

The pile driving was monitored using Pile Driving Analyzer (PDA) during initial driving and at restrike on March 28, one to two days later. A second restrike was performed on all test piles; on April 29; for TP1 and TP3, 33 days after end of driving, and, for TP2, TP4, and TP5, 34 days, respectively.

PILE PREPARATION

SACL Engineers opted to terminate the concrete pour below the upper end of the steel pipe (pile head) to enable installing the telltale and arranging for monitoring the pile compression from inside the piles. As discussed later in this report, placing the jack to load on the pile rim resulted in delamination of steel and concrete, i.e., gradual loss of adhesion between the steel pipe and the concrete core as the test progressed, causing the strain measurements to become compromised.

SACL had called for the piles to be filled with fluid grout pumped through a grout hose discharging at the bottom of the piles after installation. The instrumentation cage was then to be lowered into the grouted pile. However, instead of grout and grouting equipment, ordinary concrete was delivered to the site. As it was necessary to stay with the project schedule, the delivered concrete was used and poured down the pipe from the pile head. As the instrumentation cage could not be pushed into this stiff consistency concrete, the cages were placed in the empty pipe before pouring the concrete. For unknown reason, no slump test or cylinders were prepared from the concrete and, therefore, the concrete strength is unknown.

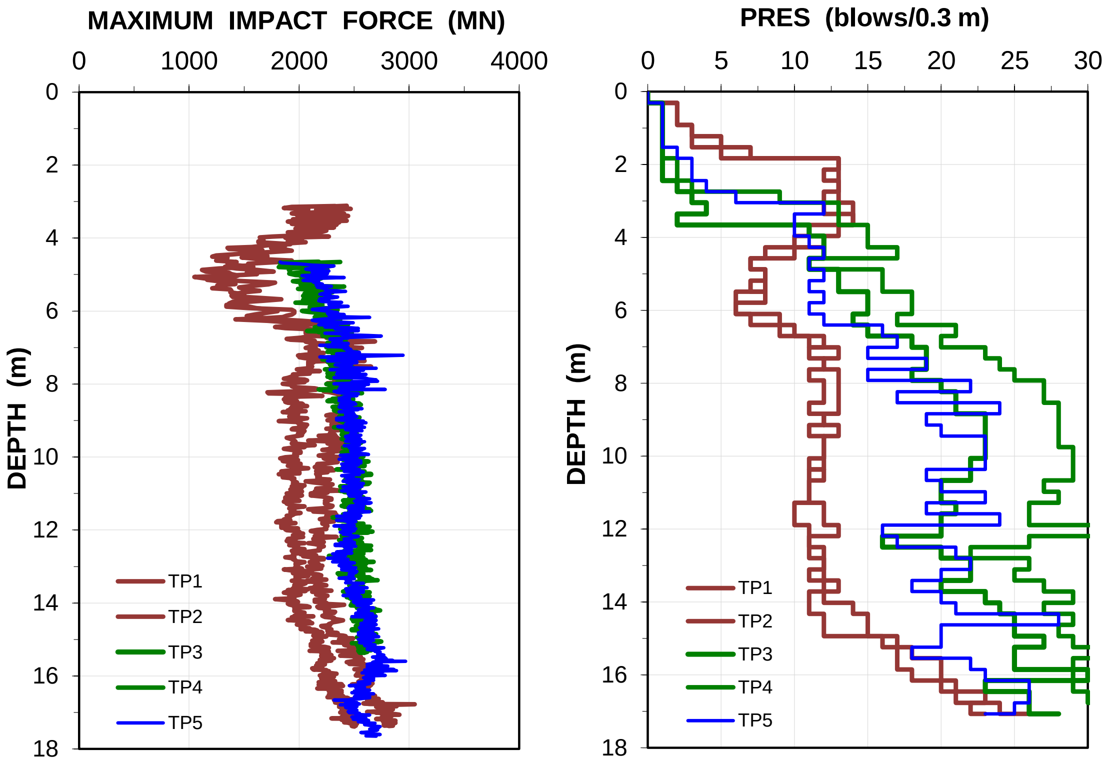


Fig. 3 Maximum driving force (FMX) and pile penetration resistance (PRES)

Regarding Pile TP5, because of the flexible (telescopic) connection of the steel pipe at the location of the bidirectional cell allowed water to enter the pipe, Pile TP5 filled with water after the driving. The concrete could therefore not be dumped into the pipe as it would then segregate, and the concrete had to be placed using a tremie pipe. As the tremie pipe could not be inserted with the instrumentation cage in place, the instrumentation cage with the bidirectional cell had to be installed after the concrete had been poured. To make the concrete sufficiently fluid for enabling insertion of the cage in the pile, water was added to the mix. However, the only tremie pipe the contractor had available was 3 m (10 ft) short and, therefore, the concrete in the lower 3 m length was discharged into the water. This seemingly caused segregation of the concrete at the bottom of the pipe and it became too dense for the cage to penetrate to the pile toe. The cage, therefore, had to be cut and the gages relocated (Table 1). Unfortunately, when inserting the amended cage, the concrete had set-up sufficiently to prevent the cage to reach the new desired depth and the bidirectional cell became located above the shift from straight to tapered section, resulting in that the bidirectional test could not be performed. A head-down test was instead carried out. However, the strain-gage data became severely messed up and, but for one gage level, the data were not useful.

LOADING ARRANGEMENT and TEST SCHEDULE

The static loading tests was performed on April 4 through April 15, 2025, comprising 150-kN (34 kips) load increments with no intermediate unloading-reloading. The set-up time between initial driving and loading test were 7, 14, 12, 13, and 18 days for Pile TP1 through TP5, respectively. The load increments were applied every 16 minutes (the operator wanted to make sure on the prescribed 15-minute load holding). A separate load cell was used to monitor the applied loads. The reaction support was a loaded platform placed on two 16 ft by 5 ft timber mats (See Figure 4). Three free space between mat and pile was 4.3 ft (1.3 m). The test load was from a loaded platform (concrete-block kentledge system) and the assigned kentledge weight was 3,000 kN. At the end of the second test (Pile TP3), the kentledge started to lift off when the applied load was about 2,600 kN. For the following tests, additional concrete weights were placed on the platform.

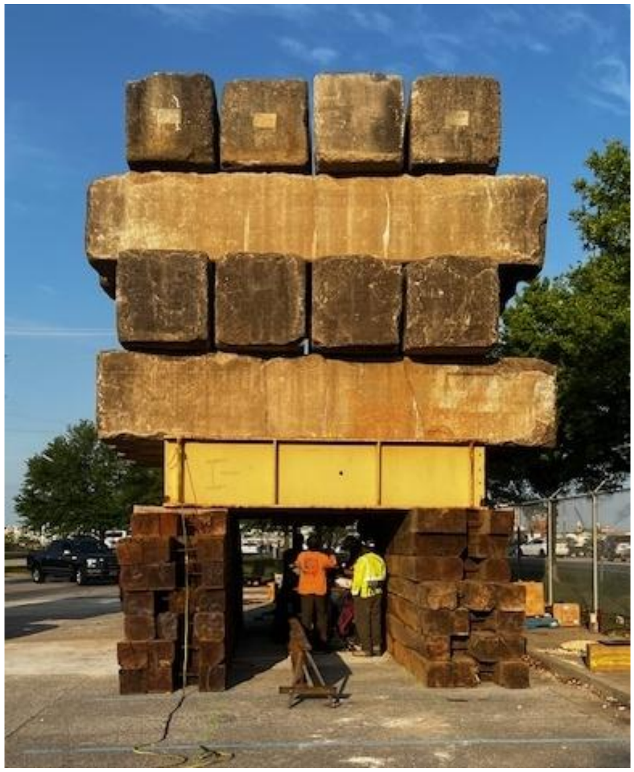


Fig. 4 Kentledge arrangement

Figure 5 shows the load-time records of Pile TP1. The same schedule was applied to all five head-down tests. Data were collected every 30 s and stored on a data collector unit for later processing. For all piles, on reaching the maximum load, maintaining the jack pressure and applied load, required frequent activation of the pump. It is possible that the shaft resistance developed a strain-softening mode, while the pile was pushed deeper. (This could have been established by not activating the jack and letting the movement cease for a stabilizing smaller than the maximum load—with then activating the pump to the assigned maximum load a few times).

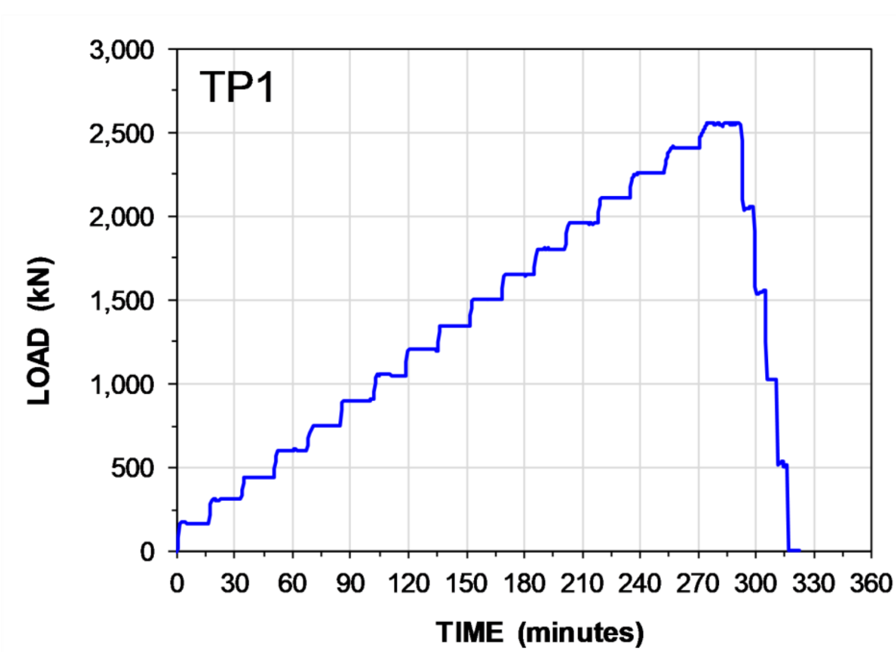


Fig. 5 Load-time records (Pile TP1)

STATIC TESTS

The pile-head load-movement curves of the five tests are shown in Figures 6A - 6C, comprising the records at the end of each 16-minute load-holding. The test record tables are attached in Appendix I. The curves appear straight-forward, but for the compression records of Pile TP5 maximum load records (and, therefore, also the records of the toe movement). The reason for the deviation is not known.

As shown in Figure 1, compiling the tests, the taper piles, Piles TP3 - TP5 carried close to 20% more load than the straight piles.

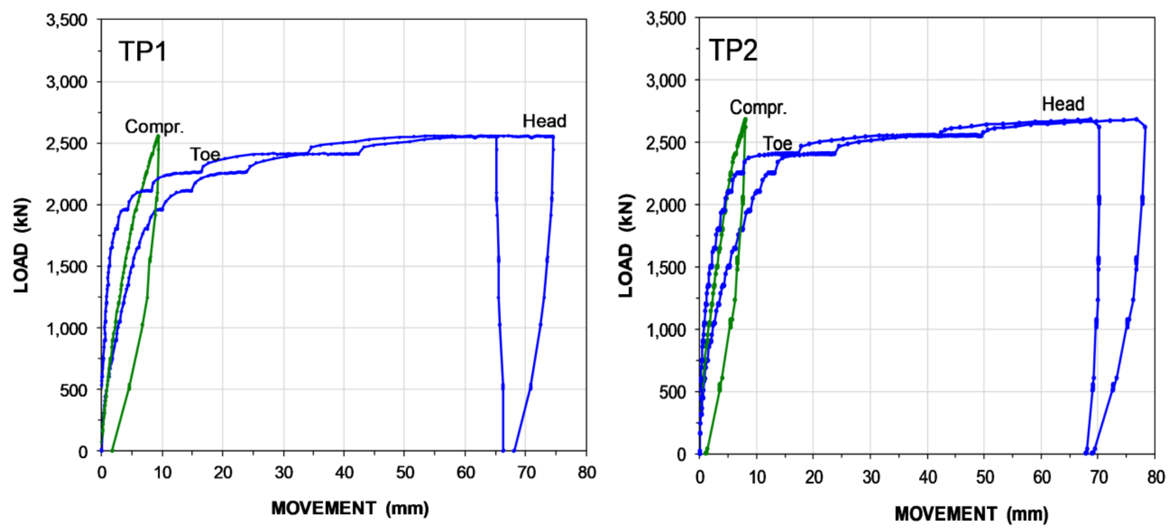


Fig. 6A Piles TP1 and TP2 load-movement curves

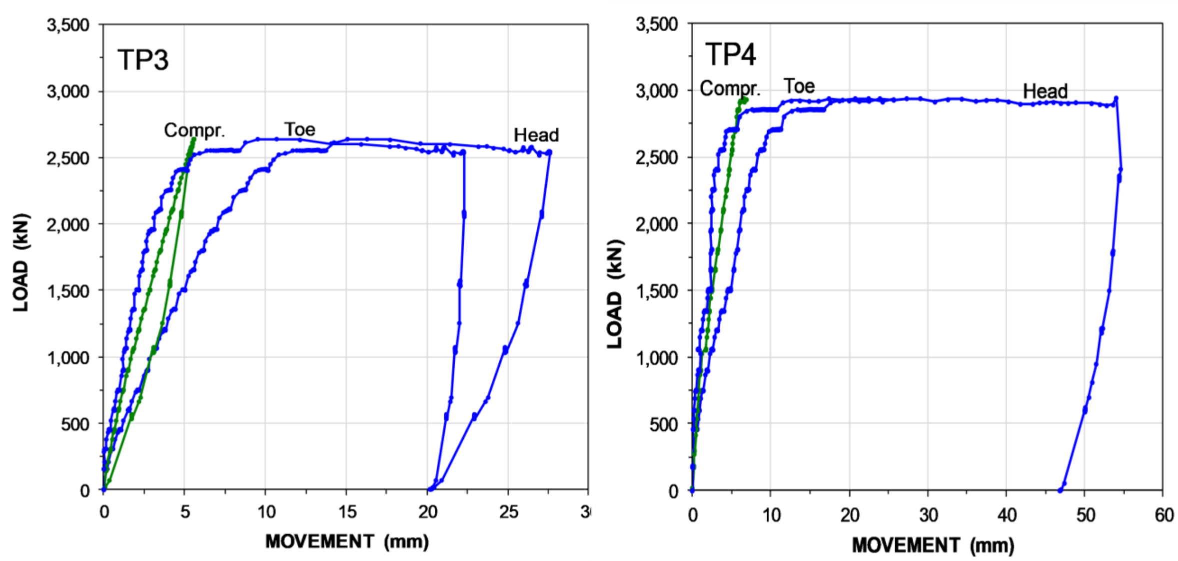


Fig. 6B Piles TP3 and TP4 load-movement curves

The strain-gages were placed in diametrically opposed pairs to eliminate influence of bending. At three of the five gage levels, two pairs were used for extra precaution against loss of a gage (a loss of a gage means the loss of the use of also the surviving gage, i.e., the full gage-pair). The labels SG1 through SG5 indicate the gage levels numbered from the level nearest the pile toe and upward. The label addition "-1" through "-4" designates the individual gage at the gage level, odd and even numbers represent pairs).

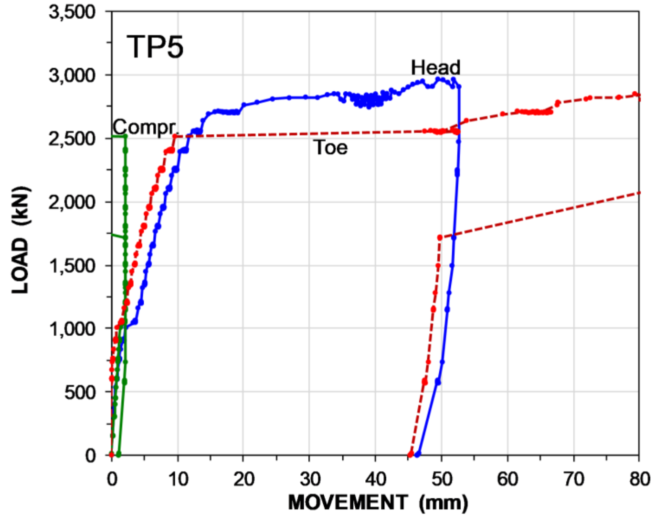


Fig. 6C Pile TP5 load-movement curves

All gages survived the pouring of the concrete and delivered consistent data representative for the strain in the concrete at the gage location. Figure 7A shows an example, typical for all gage pairs, of the response of four individual gages at Level SG3 and of the average of all four. Gages SG3-1 and SG3-3 comprise one pair and Gages SG3-2 and SG3-4 the second pair. The graph to the right shows that the two pairs gave practically the same average. The slight difference is inconsequential. The pouring of the concrete down the pipe were feared to have damaged the gages, but the gage attachment proved to be sturdy enough to take the abuse.

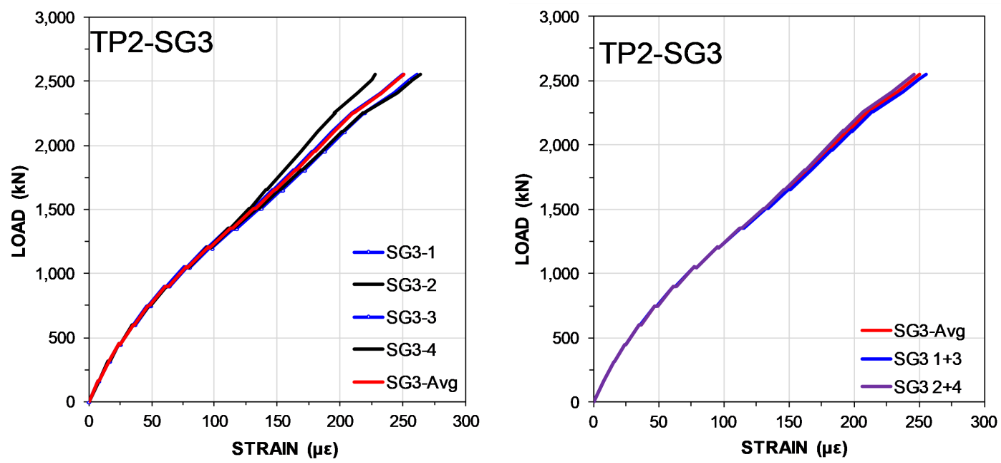


Fig. 7A Strain-gage readings at SG3 level in Pile TP3. Left graph: all records and average of all.
Right graph: average of the two pairs and average of all.

However, the unfortunate placing the jack on the rim of the steel pipe instead of on the concrete, significantly impaired the strain-gage measurements. In loading a pipe rim, the steel pipe laterally expands and this expansion, albeit minute, will cause a loss of adhesion (delamination) between the steel and the concrete gradually progressing down the pile as the load increases. While the strain-gages would then give values of the strain in the concrete, the ever so accurate records of the concrete strain will not reliably reflect the average strain in the pile and provide a correct value of the force at the gage level. The delineation caused the uppermost gage level, SG5, to became useless at first load increment. This gage was intended to serve as "calibration gage", that is, to give the conversion from strain to force via the EA-parameter. The loss caused conversion of the strain records to force to become imprecise. Moreover, because most of the force in the pile load will be in the steel pipe, the pile will compress more than would a pile with full interaction between steel and concrete would do.

Figure 7B shows examples of strain records in Pile TP1. While Gages SG1 (0.5 m above the pile toe) and SG2 (3.5 m above the pile toe) appear to have provided consistent strain records, Gages SG3 and SG4 show load-strain data that appear inconsistent to loads beyond about 1,800 kN and 1,200 kN, respectively, indicating loss of adhesion between steel and concrete. Gage SG4 even indicates some reducing strain — unloading—beyond about 2,000 kN, which indicates a full loss of adhesion between steel and concrete.

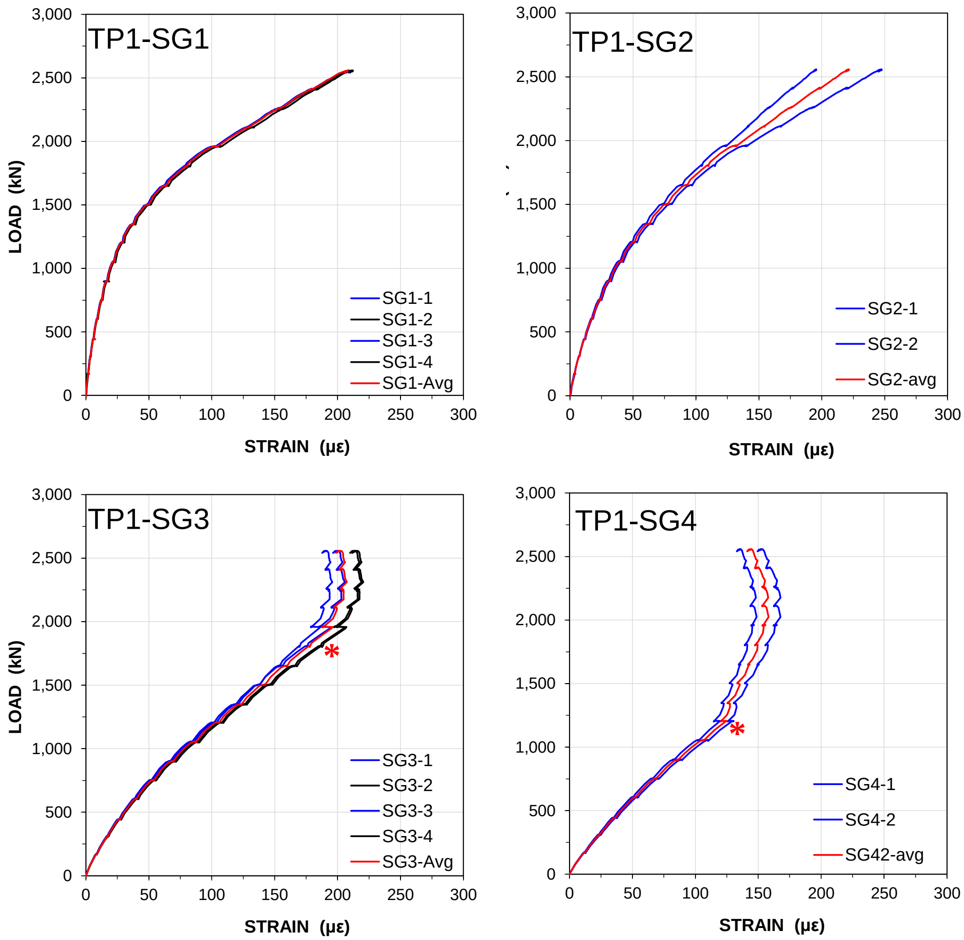


Fig. 7B Load-strain records for Pile TP1 strain-gage levels

Figure 7C shows strain-gage records SG2 and SG3 in Pile TP3 (taper pile). In contrast to the Pile TP2-SG2 records, the Pile TP3-SG2 records were inconsistent starting from about 2,000 kN applied load, while TP3-SG3 showed consistent values throughout.

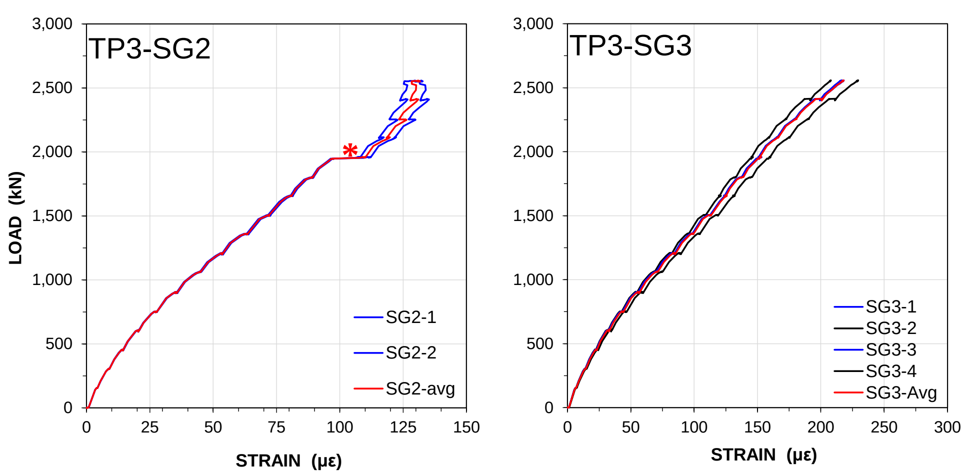


Fig. 7C Load-strain records for Pile TP3 strain-gage levels SG2 and SG3

To determine the relation between strain and force, the strain records can be plotted as change of load over change of strain (i.e., the Tangent EA-Parameter) versus strain; the plotted values will usually converge toward a more or less constant slope. However, there are two important conditions for this to be true: first, the soil response must be fully mobilized and truly plastic. If the soil resistance is strain-hardening or strain-softening, the EA-Parameter indicated by the slope of the plotted curve will be proportionally larger and smaller, respectively. Second, the pile modulus, E, must be approximately constant, which is usually the case, however, although, depending on the mineral of the ballast material, some concrete mixes will show reducing E-modulus for increasing stress. This is why a calibration gage level, such as Gage SG5, independent of the soil, is necessary.

Figures 8A - 8E combine graphs showing applied load vs. strain and graphs showing EA-parameter vs. strain. In Figure 8A, the records of SG1 and SG3 of Pile TP1 appear trustworthy, whereas those of SG3 and SG4 are consistent only before reaching a certain applied load, as mentioned in connection with Figure 7B. For the former, the slope of the curves is about 7.5 GN. Figure 8B shows the similar graphs for Pile TP2 with the same 7.5 GN slope of the load-strain curves. For both, the EA vs. strain plot suggests a slight deviation from the 7.5-GN value. It also indicates where SG3 ceased to provide reliable numbers. The EA of TP2-SG2 slopes downward, which could be taken to indicate a strain-softening shaft response, as could that of TP2-SG1. However, the latter gage level is very close to the pile toe and represents the toe resistance, which is likely not strain-softening, but, instead, strain-hardening. Indeed, the tangent method for determining the EA-parameter is far from exact.

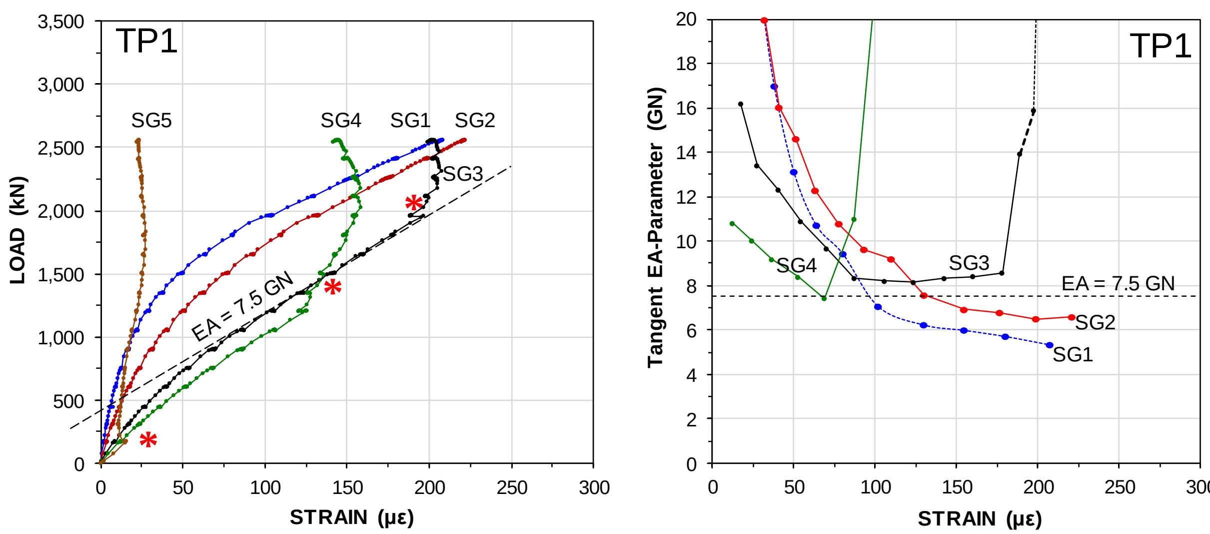
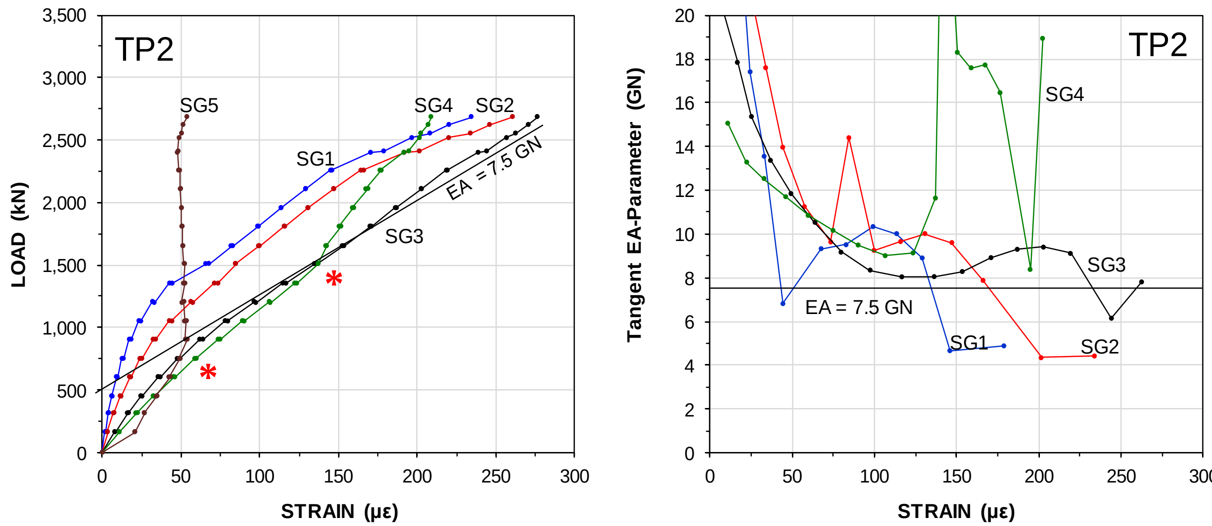


Fig. 8A Load vs. strain and EA-Parameter vs. strain for Pile TP1.

An EA-parameter of 7.5 GN correlates to a concrete E-modulus of 31 GPa, which is entirely commensurable for the confined concrete in the piles. It would imply a modulus of about 28 GPa for the unconfined concrete considering usual range of Poisson ratio for concrete of about 0.20. The maximum credible value would be about 35 GPa, which would correlate to an EA-parameter of 8.0 GN. However, the TP3 load-strain curves in Figure 8C appear to suggest an EA-Parameter of 10 GN (correlating to E = 47 GPa). This is misleading and due to the fact that the full shaft resistance for SG3 and SG4 was not mobilized even at the applied maximum load combined with the lifting of the kentledge platform mentioned earlier. That is, the pile response is still in a strain-hardening state.



Fig, 8B Load vs. strain and EA-Parameter vs. strain for Pile TP2

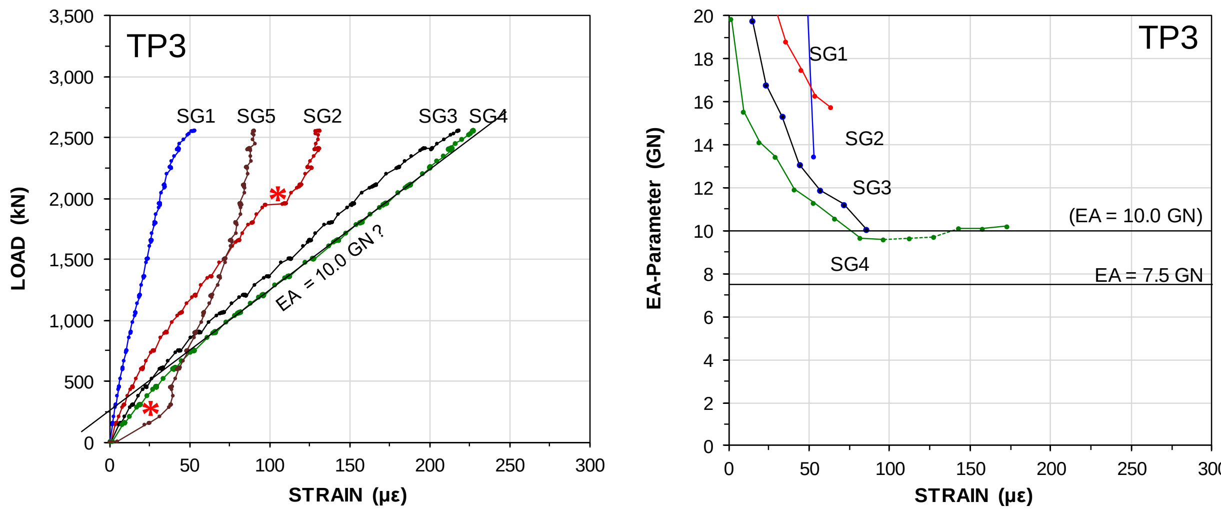


Fig. 8C Load vs. strain and EA-Parameter vs. strain for Pile TP3

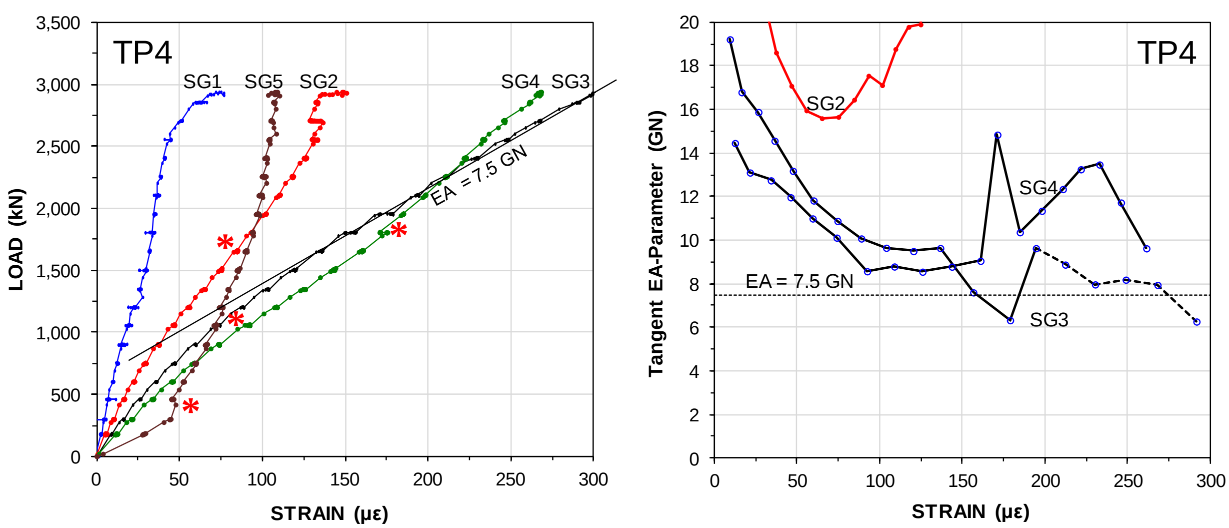


Fig. 8D Load vs. strain and EA-Parameter vs. strain for Pile TP4

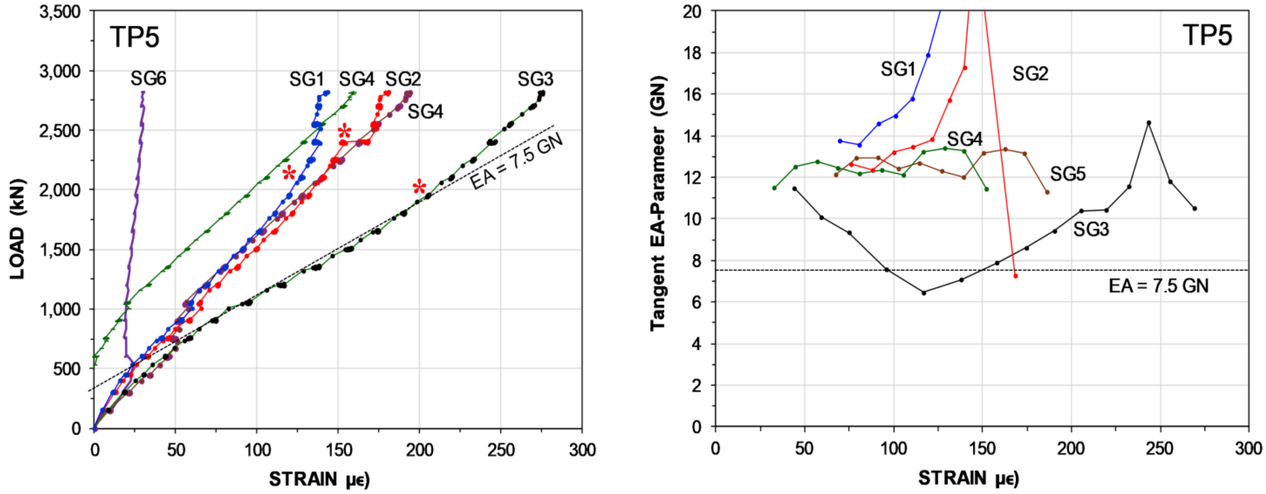


Fig. 8E Load vs. strain and EA-Parameter vs. strain for Pile TP5

As the SG5 calibration was lost in all test piles, the EA-Parameter had to be estimated from the EA-correlations. The best estimate was 7.5 GPa. This value, which, as mentioned, correlates to an E-modulus of 31 GPa for the concrete, was used to convert the measured strains to force for the strain gages in the uniform section. In the tapered section (TP3 and TP4), steel and concrete area-proportioned EA-parameters were used: 4.0 GN and 2.0 GN at SG1 and SG2, respectively. The so-determined force distributions are plotted in Figures 9A - 9E. Force values found unrepresentative due to the loss of adhesion (delamination) between the steel and the concrete were excluded.

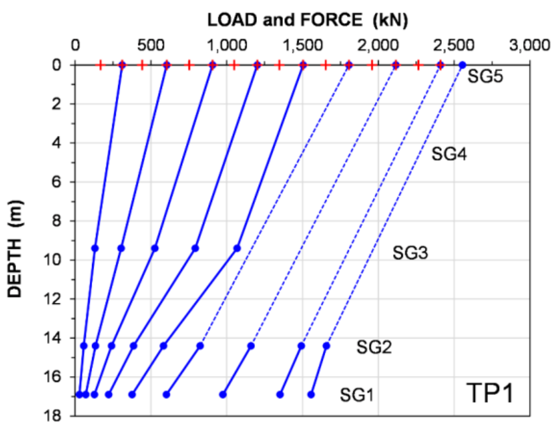


Fig. 9A Force distribution Pile TP1

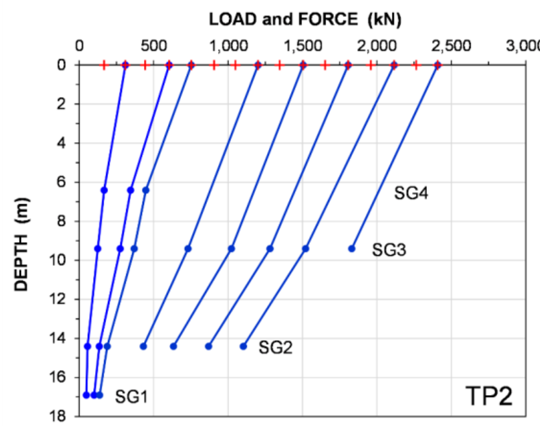


Fig. 9B Force distribution Pile TP2.

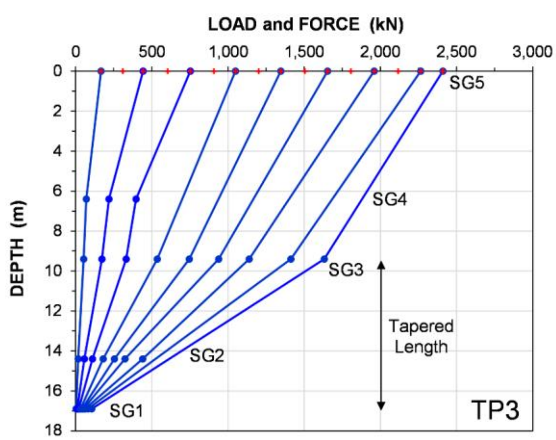


Fig. 9C Force distribution Pile TP3

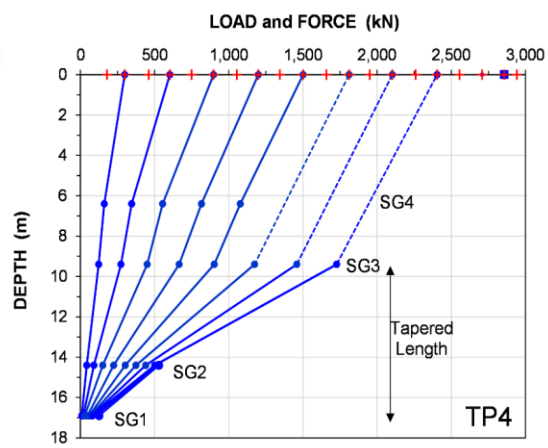


Fig. 9D Force distribution Pile TP4

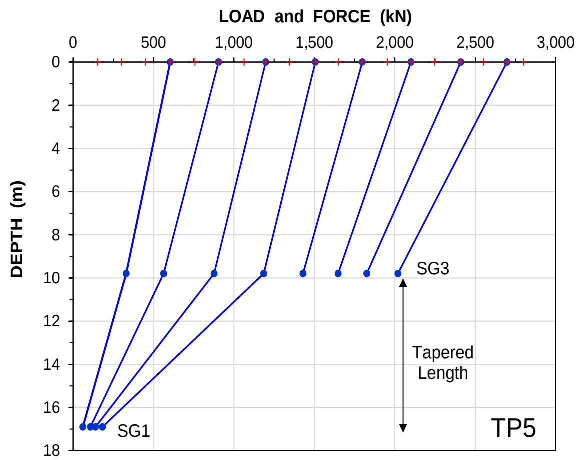


Fig. 9E Force distribution Pile TP5

It is notable that the differences in force between gage levels SG2 and SG1 in Piles TP1 and TP2 are rather small. At his depth, the shaft resistance (depicted by the slope of the force curve and the difference between the forces should instead be large compared to the force distribution higher up in the pile. That it is not so, is an indication of presence of residual force

DYNAMIC TESTS

The CAPWAP determined load-movement curve are compiled in Figures 10A - 10C for Piles TP1 through TP5, respectively, together with the load-movement curves measured in the static loading tests carried out 7, 14, 12, 13, and 18 days after the pile were installed. RSTR2 driving tests were carried out one month after the piles were installed. The CAPWAP determined load-movement curves plot consistently below those measured in the static loading test.

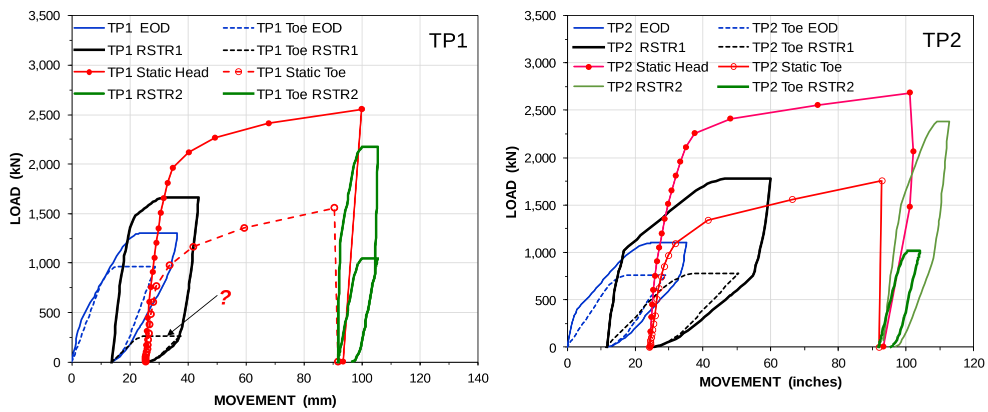


Fig. 10A Piles TP1 and TP2. Load-movement curves of static test and CAPWAP

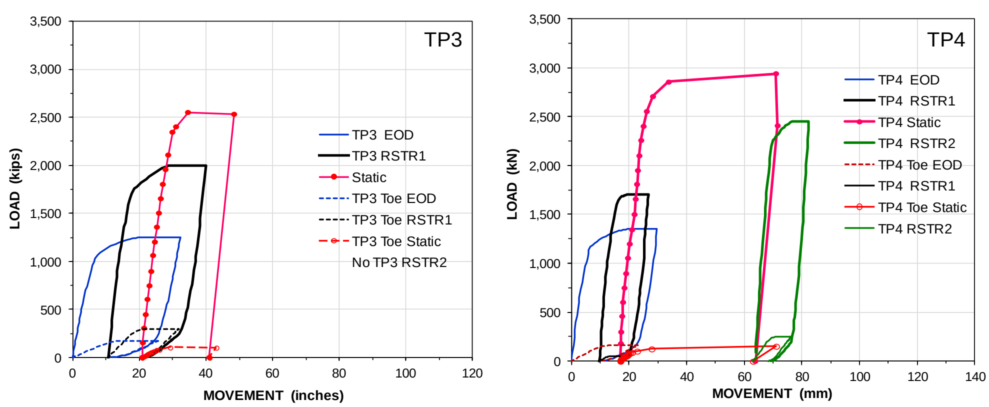


Fig. 10B Piles TP3 and TP4. Load-movement curves of static test and CAPWAP

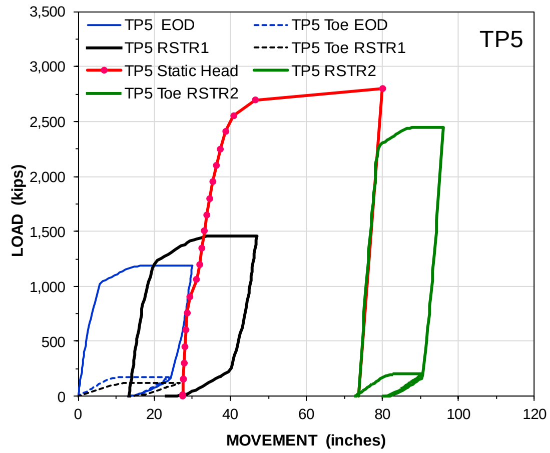


Fig. 10C Pile TP5. Load-movement curves of static test and CAPWAP

The CAPWAPs indicate that there was a slight set-up between the End-of-Driving (EOD) and the one-day restrike (RSTR1) events. The CAPWAP results from the 30-day restrike (RSTR2) imply that the set-up continued during the full month additional wait time. However, considering that the pile mass (due to the concrete) had increased by about four times, the RSTR2 CAPWAP results are not fully comparable to the RSTR1 CAPWAP results.

The notable finding is that for both the static and dynamic tests, the pile-toe resistance for the uniform piles (Piles TP1 and TP2) were significantly larger than that for the taper piles (Piles TP3 - TP5). As the difference is much larger than the difference between total resistance, both the static and the CAPWAP results indicate that the shaft resistance of the taper piles was larger than that of the uniform piles despite the larger surface area of the latter piles.

Figure 11 compiles results of CAPWAP-determined load-movement curves and those of static loading tests carried out on a prestressed concrete pile driven close to the subject test site. (The case was used as a reference test in planning for the subject test). The 13-day restrike implies a minor increased set-up compared to the one-day restrike. However, the increase can well be due to the preceding static loading test leaving the pile with a larger residual force than present after the one-day restrike.

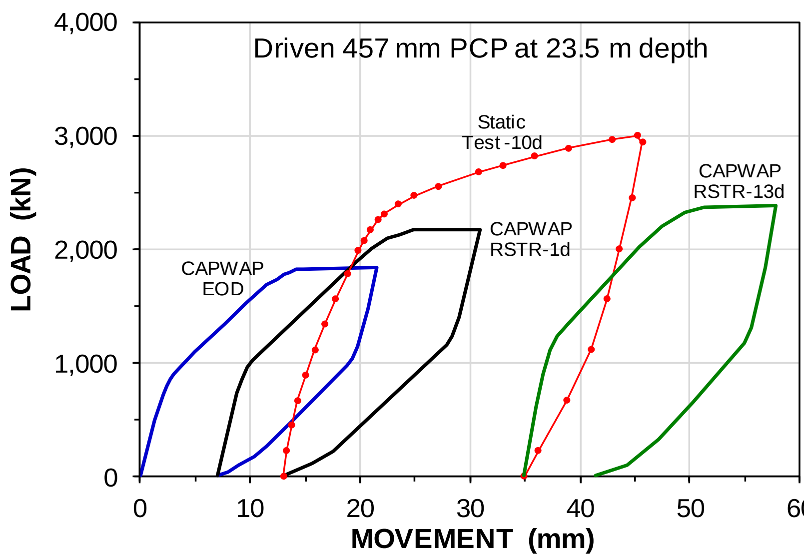


Fig. 11 Results from the reference test. (Data from Mobile River Load Test Program TP-10B-1-2 JDBss AFT Project 118008_I-10.pdf–2018)

Figure 12 indicates a compilation of the static outcome of the EOD dynamic tests and CAPWAP analyses, While the toe resistance of the uniform and the taper piles is compatible to the toe area of the taper pile being 20 % of that of the uniform pile, despite the reduced shaft area of the taper section (average area of the taper section being about 75 % of the full size section), the indicates shaft resistance along the taper length is several times larger for the taper pile as opposed the uniform pile.

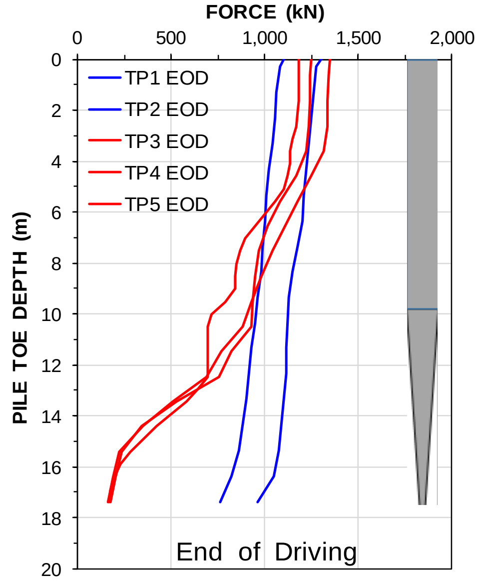


Fig. 12 CAPWAP-determined force distribution for Piles TP1 -TP5 at EOD

RESULTS ANALYSIS

Shaft and toe resistances are governed by two main aspects. First, by the effective stress acting against the shear surface, taken as proportional to the effective overburden stress, second, by the ratio between the shear force and the effective stress, called "beta-coefficient (ß)", and third, by the movement between the pile surface and the soil, usually referenced to the movement of the pile with the soil assumed not moving due to the pile being pushed down. (In reality, there is no movement between the pile surface and the soil, but a shear deformation zone. The movement reference lies away from the pile). If the soil is preconsolidated is almost always unknown, as it is also for the subject site, but it is considered limited to the initial part of the load-movement curve (Fellenius 2025[1]).

Determining the effective overburden stress can be quite complex as it has to consider the depth to the groundwater level, potential pore pressure gradient in the soil layers, potential changes of degree of consolidation, and preconsolidation stress. However, for the subject case, the depth to the groundwater table is known and the gradient is unity in the sand. Additional important factors affecting the effective overburden stress are potential fills, loaded areas, excavations, neither of which, however, are involved in this test project. Therefore, the distribution of distribution of effective stress along the piles is known for the subject case.

The unknown factors are the ß-coefficients in the soil layers and their dependency on movement as well as mineralogy, rotation of principal stresses, and taper (as for the current taper piles. The movement relations are expressed in so-called t-z functions for shaft resistance and q-z functions for toe resistance. The increase of ß (resistance) rises initially almost linearly, and, then, either shows a transition to a gradual increase (strain-hardening), gradual decrease (strain-softening), or becomes constant (plastic).

[1] Fellenius, B.H., 2025. Basics of foundation design—a textbook. Electronic Edition, www.Fellenius.net, 572 p.

The back-calculation analysis comprises fitting the strain-gage calculated force-movements to those measured at the gage levels, starts with the pile-toe records at the lowest gage, in this case SG1, the toe-force gage. Each fit involves selecting a specific point, i.e., a "Target Force" and "Target Movement" and, then, applying effective stress conditions to fit a calculated force to the actual response. The second step is to choose a t-z or q-z function for the pile element and adjust it in a series of calculations until calculated force movement and measured agree. An adjustment of the target force may be found necessary as the fitting progresses.

After achieving a satisfactory fit for the toe-gage level force-movement, the action is repeated for the next gage level, keeping the input that gave the fit to the pile toe and lower pile elements, etc., until, finally, the measured and calculated pile-head load-movement agree. Then, the so-obtained various target forces (ß-coefficients) and movements (t-z and q-z functions) constitute the theoretical analysis parameters for the pile response. Most common objective is to obtain the parameters that determine the movement (settlement) of a pile or a group of pile supporting a specific sustained load , or piles of lengths and sizes that differ from the test piles in some way or either.

Theoretically, the analysis is quite direct as no heavy algorithms are included. However, the analysis is too complex for a hand calculation, even with spreadsheet assistance other than for very simple case. It is, therefore, necessary to employ a suitable software. Interacting with the UniPile6 software (www.unisoftGS.com) will make the process simple and fast.

Figure 12A shows the final TP1-fit of measured and calculated applied load to the pile-head, pile-toe force, shaft resistance, and pile compression versus movement. The movements of SG2 and SG3 are estimated from the telltale records. The measured curves are plotted in blue and the UniPile-calculated curves are in red. Note that the calculated compression line, applying EA = 7.5 GPa, is steeper than the telltale measured. This is because the measured compression is affected by the steel-pipe and concrete delamination and primarily governed by the steel compression and only partially affected by the compression of the concrete. This has caused the measured compression to be larger than for an intact pile. As indicated in Figures 8A-8E, once the bond between the steel and concrete is affected, the strain-gage records, although they are true measurement of strain in the concrete, the strain coupled with the pile (steel and concrete) EA-parameter represents a smaller than true value of the force at the gage location. Similarly, the measured load-movement for the pile head is true, but the measured movement mostly reflected the compression of the steel pile and only partially the compression of the concrete. That is, as the delamination progressed, the forces calculated from the strain measured in the concrete were smaller than true.

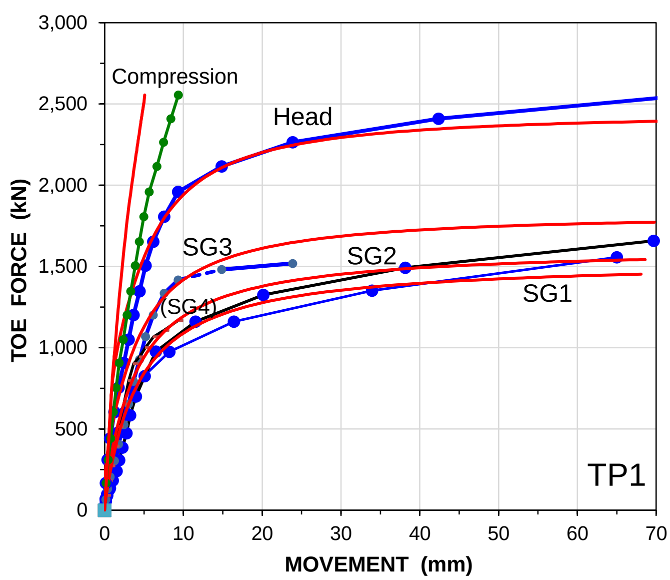


Fig. 12A Measured and fitted load-movement curves for Pile TP1

The fitting of the simulation to the records assumed that the force-movements start at zero force, whereas the pile in reality already has a significant force of unknown magnitude present ("residual" force). The residual force could be modeled and a fit be obtained by employing two separate force-movement calculations (t-z/t-q functions); one for, say, the first about 10-mm movement and one for the remaining records. However, as the response beyond 10 mm is more important for the assessment of the pile, only the second input was used.

The figure shows a difference between the measured and calculated pile compression. As mentioned, the reason for the difference lies in the fact that the delamination resulted in the concrete compressing less than the steel, the latter being the telltale-measured compression. The input to the analysis for the fitting assumed a concrete E-modulus of 31 GPa (EA = 7.5 GPa). The difference between the assumed EA-parameter of the pile and the actual will have affected the initial portion of the fit to the force-movement of the gages less than the latter portion. An EA-parameter for the case that would include, first full, and, later, partial influence of the adhesion steel to concrete and changing as the load increased was not attempted.

The fits were achieved by target ß-coefficients and unit toe resistance at a 5-mm pile-element target movement and function coefficients indicated in Figure 12B. Normally, the toe response would follow a Gwizdala q-z relation which would be a flatter curve than that shown in the figure by the hyperbolic q-z function that gave the best fit to the records. The strain-softening function (Rahman) shown in the figure was applied to all shaft elements. However, the ß-coefficients that gave the final fit differed with depth. That the back-calculated target ß-coefficient for the lower half of the piles is very small (ß = 0.16) is commensurable with presence of residual force.

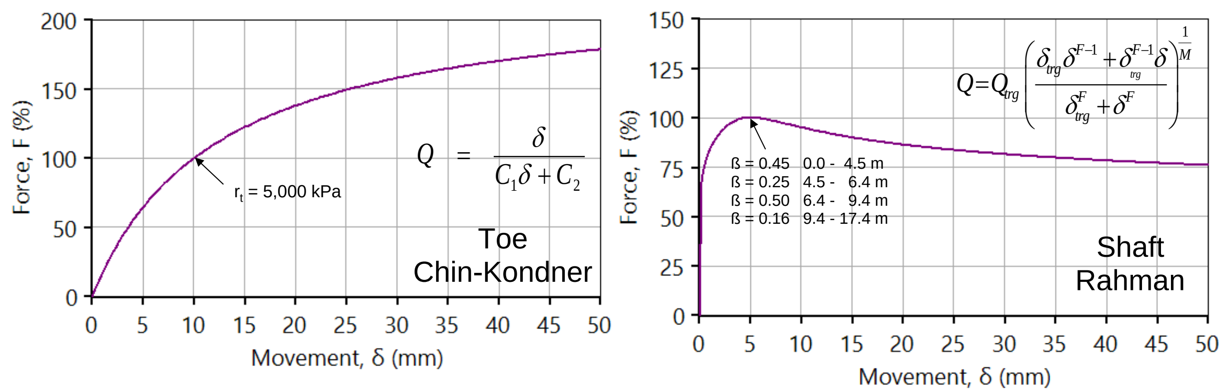


Fig. 12B Pile TP1 q-z and t-z functions

Figure 13A shows the measured and simulated force-movement curves for Pile TP2. The UniPile simulation was made with the same target toe stress as for Pile TP1 and with slightly smaller target ß-coefficients above 9.4 m depth. The shaft t-z function was also a Rahman function, but one with slightly more pronounced softening. The loss of gage values (see Figure 9B) made more detailed fitting not meaningful.

The analysis of the tapered pile is more challenging. The toe area is much smaller, only 20 % of the full width, although the pile-toe response is the same in principle. The shaft resistance acts also in shear between pile and soil, however, the taper accentuates the rotation of principal stresses. That is, it should be expected that the taper would generate large shear force. In addition, the effect of the taper is that the pile-to-soil relative movement induces a compression in the soil much like the toe force does to the soil below the pile toe. The first effect can be addressed by the ß-coefficient coupled with a t-z function and the second by assuming a concurrent toe-bearing type of response expressed in a q-z function applied to the projected donut-shaped area difference between the top and bottom of the pile element. The UniPile6 software incorporates this method of separating and sharing the shear and compression effect developed by a tapered pile element.

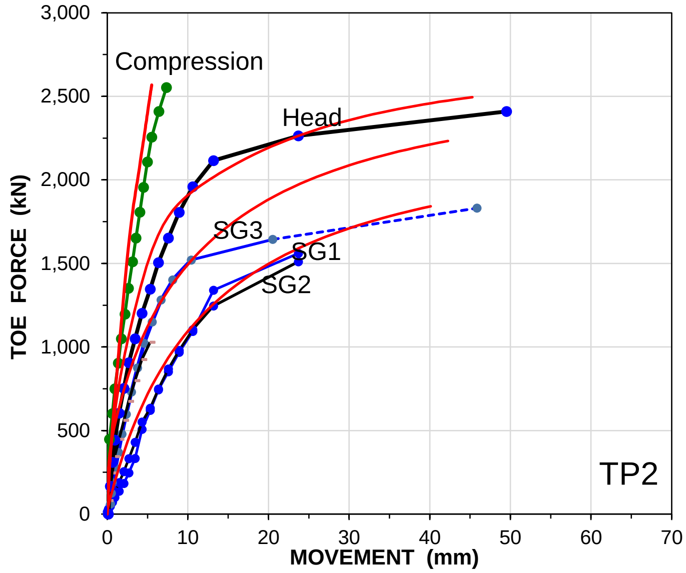


Fig. 13A Measured and fitted load-movement curves for Pile TP2

Similar to the response indicated in the CAPWAP analysis, the pile toe response of the taper piles is much smaller than for the uniform piles. However, the forces measured at the SG3 gage levels located at the interchange between the straight and tapered section is about similar to that of the uniform piles, Piles TP1 and TP2, indicate a larger shaft resistance along the tapered section as opposed to the straight section even considering the gradually reducing pile circumference of the tapered section.

Figures 13B and 13C show the measured and fitted force-movement curves for the taper piles, Piles TP3 - TP5. The fit was achieved by the same t-z and q-z functions as used for the uniform piles (c.f. Figure 12B) employing a slightly larger target ß-coefficient and similar target toe stress. Along the taper section, the "donut" shaped projected area of each pile element was assigned a the same q-z function as used for the pile toe, but about twice as large a target toe stress.

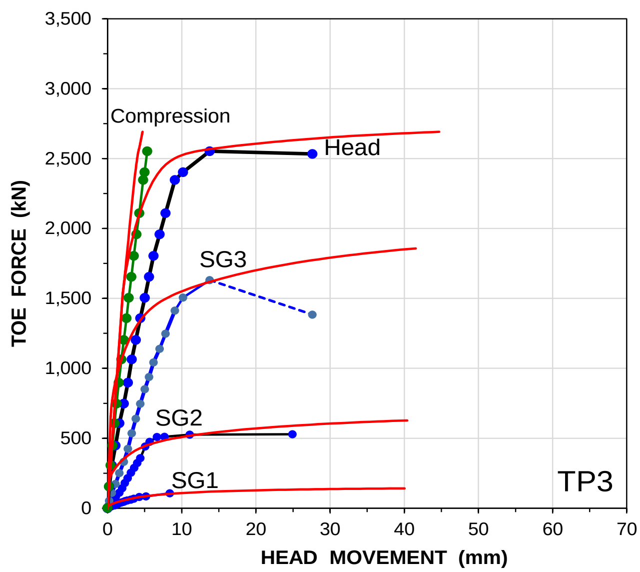


Fig. 13B Measured and fitted load-movement curves for Pile TP3

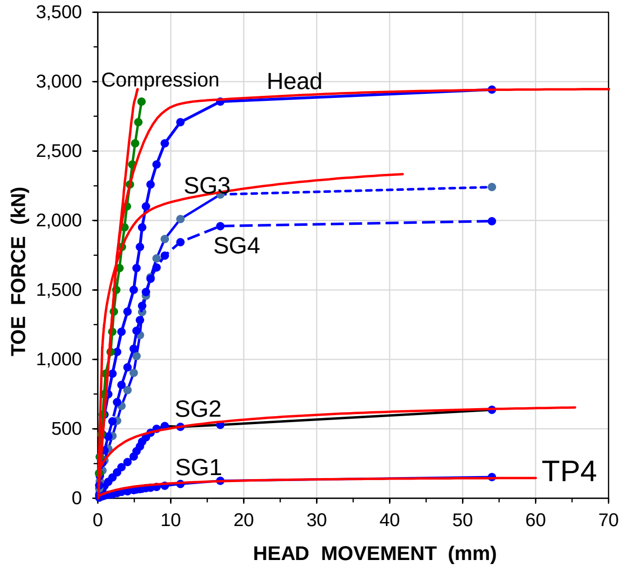


Fig. 13C Measured and fitted load-movement curves for Pile TP4

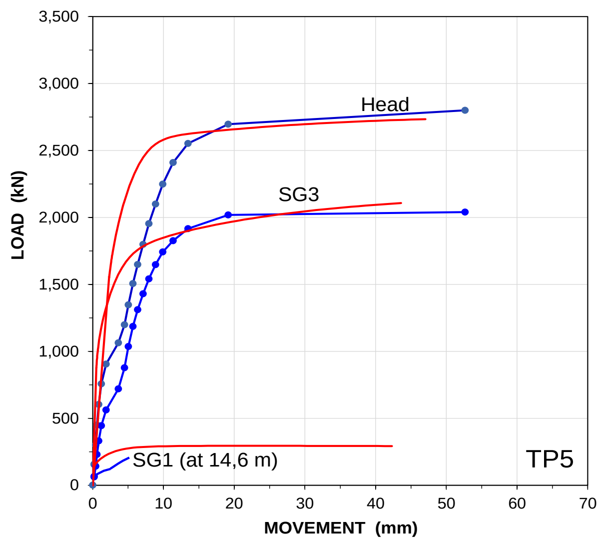


Fig. 13D Measured and fitted load-movement curves for Pile TP5

Figure 14A compiles force distributions at the 2,100 kN applied load for all five test piles, as back-calculated from the static loading tests, using UniPile6 and the fitted parameters, and Figure 14B compiles the distribution of the back-calculated shaft resistance vs. movement for the piles. Although not measured, all test piles will have presence of residual force. It is likely that this force is larger in the taper piles than in the uniform piles. This, because the soil, in rebounding from the last hammer impact and, then, pushing against the taper, will have encountered larger resistance, resulting in larger residual force.

The fit of the uniform piles to the back-calculated strain-gage force-movement response applied the same shaft resistance input for both piles. In contrast, to achieve the fit to the response of the taper piles the shaft resistance input had to differ between the piles, which is likely due to differing magnitude of residual force for the piles. Along the uniform length of the piles (above 9.8 m depth), the fitted force distributions for Piles TP3 - TP5 (the tapered piles) indicated a slightly larger shaft resistance as opposed to Piles TP1 and TP2 (the uniform piles). This is likely because of the pile having a larger residual force along the taper length will generate a larger residual force also along the uniform length above.

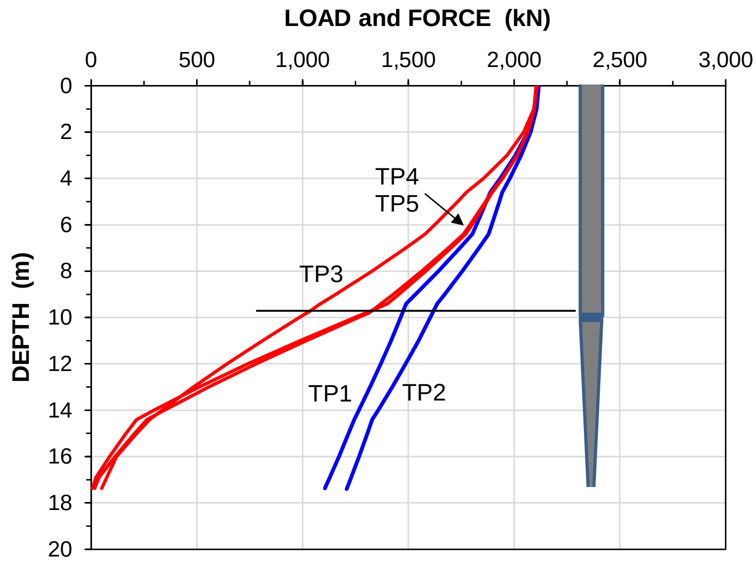


Fig. 14A Calculated resistance (force) distributions for Piles TP1 - TP5

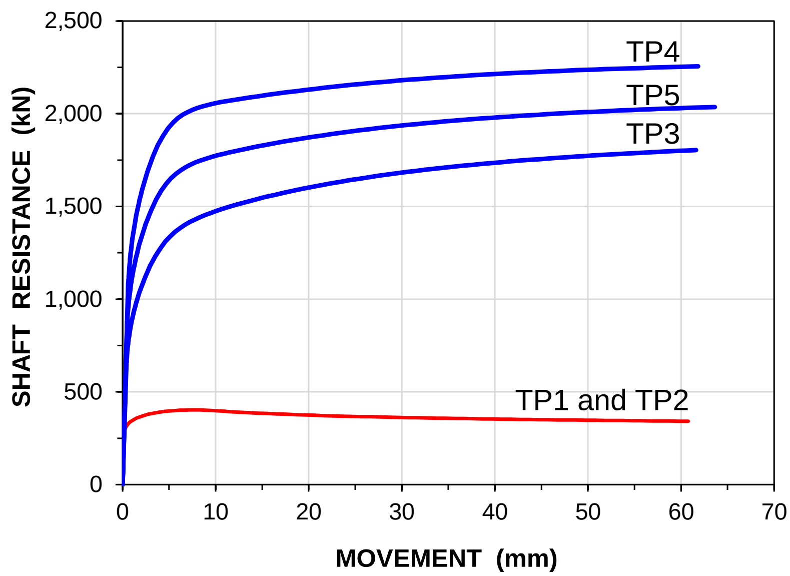


Fig. 14B Calculated shaft resistance vs. movement for Piles TP1 - TP5

The comparison of the force distribution indicates that the taper piles developed a shaft resistance along the tapered length, which, at a movement of about 5 mm, mobilized more than three times the shaft resistance mobilized by the uniform section piles in the same soil layer. The difference in shaft resistance could be due to the fact that the tapered pile in being installed displaces the soil laterally, densifying it so that the shaft resistance becomes larger as opposed to for the uniform cross section piles. An additional reason could be the fact that the principal stresses have rotated more than the for uniform pile, which would add resistance (according to Nordlund, 1963[2]). Also, and it may be the main reason, as the test load increases and the pile elements move down, the soil is displaced and compressed, which results in increased resistance.

[2]Nordlund, R.L., 1963. Bearing capacity of piles in cohesionless soils. ASCE Journal of Soil Mechanics and Foundation Engineering, 89(SM3) 1-35.

Modeling the shaft response is complex. A simple approach is to model the response as the combined effect of friction and compression (Fellenius 2025). The friction component is modeled by a target ß-coefficient and a t-z function and the compression component is modeled by a target stress and a q-z function applied to the "donut-shaped" projection of the taper for each pile element. This method is provided as an analysis option in UniPile6.

Published information on results of laboratory tests have verified that shaft resistance of a tapered pile is larger for a uniform pile. Ghazavi and Ahmadi[3] published results comparing the response of uniform and tapered piles in full scale tests on two square cross section, drive precast concrete piles. One was a uniform 400-mm (15.7 inch) diameter, 12.5 m (41 ft) long pile and the other was a 570-mm (22.4 inch) diameter, 12.5 m (41 ft) long pile tapered along the full length to a 200 mm (7.9 inch) toe diameter. The total surface area of the two piles were equal. The test results are shown in Figure 15A along with load-movement curves obtained by fitting the conditions for the uniform pile and the tapered piles in a UniPile6 analysis employing the mentioned approach for modeling the effect of the taper by assuming the shaft shear from effective stress ß-analysis combined compression effect acting on the "donut-shaped" projection of the taper. The same ß-coefficient, t-z function, and q-z function was used for both piles. The target stress for the "donut projections" of the taper pile elements was 50 % larger than that assumed for the pile toe, but the same q-z function (Gwizdala) as that applied to the pile-toe element was applied also to the tapered length pile elements.

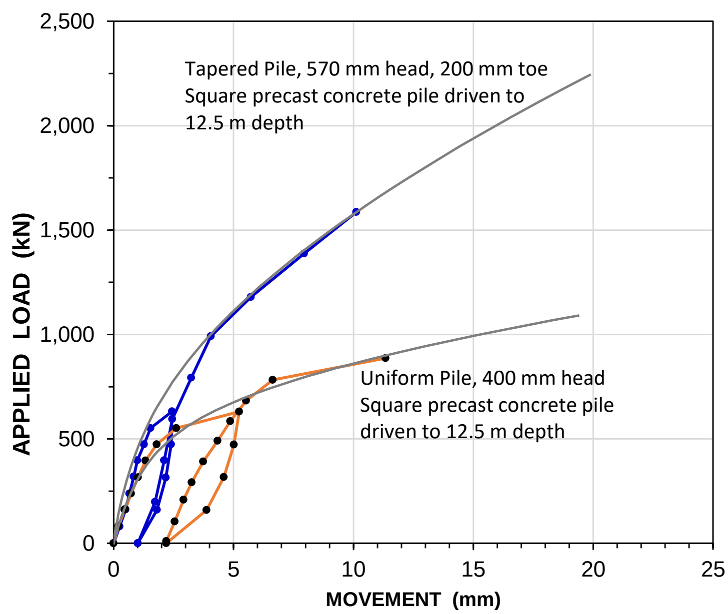


Fig. 15A Comparison of load-movement curves from tests on uniform and tapered precast piles

Figure 15B compares the back-calculated shaft resistance vs. movement for the two PC test piles indicating that the shaft resistance along the tapered pile was four times larger than along the uniform pile with the same surface area.

[3] After Ghazavi, M. and Ahmadi, H.A., 2008. Time-dependent bearing capacity increase of Sixth International Conference on Case Histories in Geotechnical Engineering. Univ. of Missouri Rolla. 6 p.

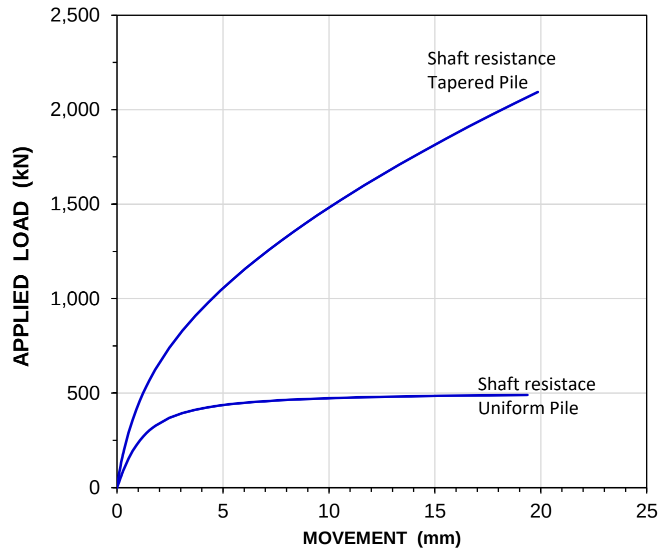


Fig. 15B Calculated shaft resistance vs. movement for the piles

CLOSURE

The main observation is the considerably large shaft resistance developed for the taper length as opposed to the uniform length.

Both the uniform and tapered test piles were affected by presence of residual force and this to an unknown degree. However, even with a "correction" for presence of residual force, the difference in shaft resistance along the depths of the tapered pile length is still considerable.

The taper provides a significant increase of the TSFP of bearing more than well offsetting the reduced area of pile shaft and pile toe. I believe this is due to the taper angle adding to the effect of rotation of principal stress coupled with resistance from soil compression because of the lateral displacement when the pile moves downward.

Sincerely,

Bengt H. Fellenius