Draft September 13, 2025

Bearing of uniform pile versus tapered pile—full-scale study

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Abstract The axial response of uniform and tapered piles were investigated. Anecdotal observations and model-scale tests have suggested that tapered piles provide higher bearing than uniform piles of equivalent dimensions, but reliable full-scale comparative data have been scarce. Published records of full-scale tests comparing the response to load of equivalent uniform and tapered piles in Iran and Italy were analyzed. Instrumented full-scale test piles were installed in Mobile, Alabama and subjected to both static and dynamic loading tests. The results of all tests showed that tapered piles developed substantially larger shaft resistance, at least doubling that of the uniform piles along the tapered length and, despite the smaller toe size, the bearing of the tapered piles exceeded that of uniform piles by up to 20% and more at comparable settlements. Back-analysis using UniPile6 software confirmed that the increased shaft due to the taper could be represented as a "donut" effect.

INTRODUCTION

Foundations placed on soft soil, usually clay or silt, have been supported on piles for more than two thousand years. For the longest time, only tapered piles—timber piles—were available. Uniform cross section steel pipes or steel beams used as piles came about two hundred years ago. About a hundred years ago, the driven precast pile appeared. Anecdotally, it was "known" that the timber pile had larger bearing than the same size (pile butt) and length of the alternative pile. The reason, it was stated, was that, unlike steel and concrete, the wood drained the soil, enabling it to consolidate and gain strength. Until recently, the taper itself was considered to have negligible effect on the pile bearing. However, since the mid-1900s, and mostly for piles driven in sand or coarse silt, it has been stated, that tapered piles provide larger bearing than equal size or area uniform piles. Piled foundations comprising tapered piles are now regularly used in many areas due to this higher bearing and also to reduced material costs. For example, tapered steel pipe piles are common in Eastern USA and tapered prestressed spun piles are frequently used in Switzerland and Northern Italy, when associated driving vibrations can be accepted (G. Togliani 2025, personal communication).

The larger bearing of the tapered pile compared to an equivalent size uniform piles is anecdotal, because there are few full-scale correlations in directly comparative tests on uniform piles; two are quoted in this paper. To meet the need for case records, where uniform and tapered piles are directly compared, Browning Enterprise, Inc. sponsored a test series in Mobile Al, comprising full-scale, side-by-side, static loading tests comprising three tapered steel pipe piles (TSFP) and two uniform piles (Fellenius 2025, SACL 2025).

THE ANALYSIS OF PILE RESPONSE TO APPLIED LOAD

Shaft and toe resistances depend on two aspects. First, they are proportional to effective stress and they depend on relative movement between the pile and the soil. Unit shaft resistance is expressed by the ratio between the shear force and the effective stress, called "beta-coefficient (β)", and unit toe resistance, similarly by the toe-coefficient, called N_t. The β- and N_t-coefficients depend also on mineralogy, rotation of principal stresses, shear- and E-moduli, and pile taper. Second, the coefficients are also proportional to the mobilized movement between the pile surface and the soil, usually assumed to occur right next to the pile surface (i.e., the soil is assumed to not move due to the pile imposed movement. In reality, movement does not occur directly at the interface surface, but within a shear deformation zone around the pile. That is, the movement reference lies away from the pile). Whether or not the soil is preconsolidated or preloaded plays a part, but how is almost always unknown. Its effect is considered limited to the initial part of the load-movement curve (Fellenius 2025).

Determining the effective overburden stress relation to use can be quite complex as the effective stress analysis has to consider the variation of density, depth to the groundwater level, potential pore pressure gradient in the soil layers, change of degree of consolidation, and preconsolidation stress. Additional important factors affecting the effective overburden stress are potential fills, loaded areas, and excavations.

It is common to represent shaft and toe resistance by a single value, an ultimate resistance that is independent of movement. Because shaft resistance normally develops an approximately plastic response after an initial movement, as a characterization, an ultimate shaft resistance has informative value. However, toe resistance rarely develops plastic condition and, as a characterization, ultimate toe resistance is meaningless. Both shaft and toe resistances are best are characterized in concert with their specific movement relation.

The shaft and toe resistances movement relations are called t-z and q-z functions describing resistance versus movement. As to the shaft resistance, initially the *ß*-coefficient vs. movement curve, rises steeply, almost linearly, and, then, it either becomes constant (plastic—ultimate shaft resistance), or shows a transition to a gradual increase (strain-hardening) or decreases gradually (strain-softening). As to the toe resistance, the N_t-coefficient versus movement usually shows a gentle rise from zero to infinite movement, always strain-hardening—there is normally no ultimate toe resistance unless a value at a specific movement is so denoted.

For a uniform pile, the shaft resistance expressed by the ß-coefficient is governed by the mobilized shear force with minimal change to the soil. However, for a tapered pile, the movement of the pile introduces a lateral component and a compression of the soil that adds soil resistance. The effect of the compression due to the taper can be accounted for by increasing the ß-coefficient or be an add-on separate value correlated to the taper by different ways. For example, Nordlund (1963) proposed calculation based on rotation of principal stress associated with the taper angle. Kodikara and Moore (1993) proposed a simplified similar analysis. Hatah and Shafaghat (2015) performed numerical analyses. Fellenius (2002; 2025) suggested to address the effect to the taper by a "donut" approach. That is, the projected size difference—the "donut" area— between the top and bottom of any unit length of the pile can be treated as a pile toe that adds resistance. This "donut" contribution can be calculated by means of applying an N_t-coefficient to the "donut" area, as suitable for the soil layer.

The *ß*- and N_t-coefficients depend on the soil characteristics, which can vary immensely from site to site, and on the construction procedures (driven, bored, material, etc.). Therefore , no analysis of response of a pile to an applied load is possible without correlation to experience, and lacking that, back-analysis of actual tests.

Most common objective of the back-analysis of test results is to obtain the parameters that determine the movement (settlement) of a pile or a group of pile supporting a specific sustained load, or of piles of lengths and sizes that differ from the test piles in some way or either.

The back-calculation analysis of the load response of a pile element comprises fitting the strain-gage calculated force and movement to those measured at the gage level. Each fit involves selecting first a specific point, i.e., a "Target Force" and associated "Target Movement". Then, effective stress conditions are applied to fit a calculated force-movement to the target force-movement points (values). The next step is to choose a t-z or q z function for the pile element and adjust it in a series of trial calculations until calculated force-movement curve and measured agree. An adjustment of the target force may be found necessary as the fitting progresses.

The analysis of loading test results starts with fitting calculations to the measured pile-toe element. After achieving a satisfactory fit for the toe-gage 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. The so-obtained various target forces (N_t and, then, β-coefficients) and movements (t-z and q-z functions) constitute the theoretical analysis parameters expressing the pile response.

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

RESULTS OF LOADING TESTS ON TAPERED PILES

Several model tests and numerical analyses have been published that compare the response of tapered and uniform piles. For example, Robinsky and Morrison (1964) reported tests performed in a sand box that showed taper piles mobilizing considerable greater shaft resistance than straight-sided piles. El Naggar and coworkers performed static loading tests on 1.2 m long model tapered and uniform piles in sand in a chamber equipped with an air bladder that could produce horizontal stress on the pile shaft. They found that that the shaft resistance of the tapered piles was greater than that of the uniform piles (Wei and El Naggar 1998, El Naggar and Wei 1999, El Naggar and Sakr 2000, and Khan, El Naggar, and Elkasabgy 2008). Paik, Lee, and Kim (2010) also performed model tests and found similar results including that that the taper also enhanced unit toe resistance. Ibsen and Barari (2025) found similar results in tests on 2 m long piles installed by jacking into sand. Gupta and Rajagopal (2015) and Shafaghat and Khabbaz (2020) presented brief reviews of the mentioned references and others.

Fellenius and Ataee (1999) criticized the relevance of the various model tests, stressing the fact that while they produced results of qualitative values, such as indicating that taper enhances shaft resistance, the quantitative results, such as ratio of enhancement, resistance distribution, and evaluated soil parameters, are not applicable to response of full-scale piles.

If presence of the taper had no effect, one would expect that a uniform pile and a tapered pile, if having the same surface area, would show the same total shaft resistance. However, the toe response of the tapered piles would be smaller due to its smaller toe area.

The anecdotal contention of greater bearing for tapered piles is supported by the results of several full-scale static loading tests on tapered piles, showing good bearing performance, e.g., Dougherty (2017). Fellenius et al. (2000), Horvath and Trochalides (2004), and Shafaghat and Khabbaz (2020). However, the references do not include comparison to uniform piles.

Indeed, all the mentioned papers, and many others not mentioned, share a weakness, the absence of full-scale tests for use in direct comparisons and in support of theoretical calculation for equal conditions. Case histories that provide direct comparison of load-movement response of tapered pile versus uniform pile are rare. I only know of three. The first is from Massarsch et al. 1997, comparing static loading tests on 9 m long, 200 mm open-toe steel pipe and a 500 to 150 mm conical concrete pile driven in sand in Sweden (Massarsch et al., 1997). The taper was 19 mm/m; 1.11°. The tapered pile produced significantly larger bearing. However, the piles are somewhat too dissimilar to warrant fine-tuning the response difference.

Tests in Iran

The second full-scale case, presented by Ghazavi and Ahmadi (2008), compared the response of uniform and tapered piles in full-scale tests in Iran on two, 12.5 m long, square cross section, driven, precast concrete piles. The soil description was described a "cohesive". The paper included no mention of the depth to groundwater table. One pile was uniform 400-mm diameter and the other was a 570-mm diameter, 12.5 m long pile tapered along the full length to a 200 mm toe diameter, i.e., a taper of 7.5 mm/m; 0.5°. Thus, the piles had essentially equal shaft area. The reported load-movement curves, shown in Figure 1, indicate that the tapered pile developed considerably larger bearing than the uniform pile. The piles were tested twice, first 35 days after driving and then again 254 days later.

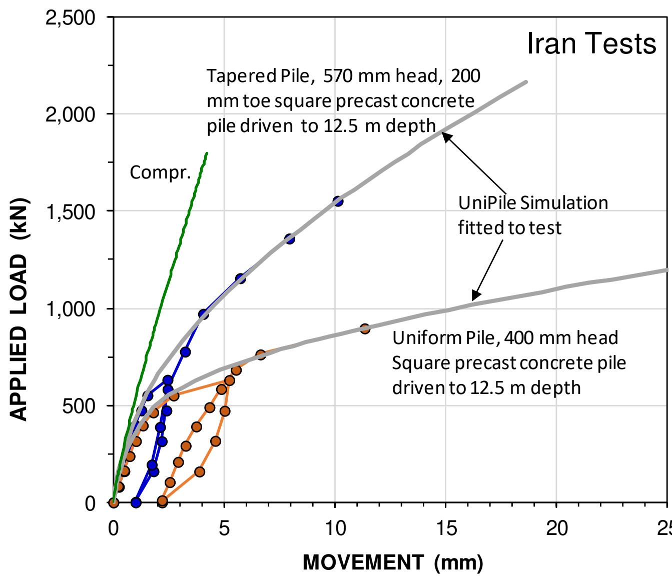

Fig. 1 Comparison between 12.5 m long tapered and uniform square precast piles with UniPile simulations

The limited information prevents a systematic back-analysis of the tests for meticulous assessment of the difference in response of the uniform and tapered pile shafts. However, it is still possible to obtain qualitative analysis results useful for a comparison of uniform versus taper response. My personal experience of back-analyses of loading tests on piles in cohesive soil suggests that the simulated shaft resistance be per a hyperbolic t-z function with a $\beta = 0.25$ target resistance at 5 mm movement (equal to 90 % of the resistance at large movement; theoretically infinite) and a simulated toe resistance per a Gwizdala q-z function with an $N_t = 13$ toe resistance and function coefficient of 0.6. The latter means that the ratio of any two toe resistances would be equal to the ratio at exponent 0.6 of their movements mobilized by those resistances. For reference, assuming no toe resistance would require $\beta = 0.4$, which would clearly be incompatible with a zero toe resistance. And a very low-end β-coefficient of 0.15 would have required $N_t = 20$. The two coefficients would be incompatible unless the soil at the pile toe would be stiffer than up along the pile.

Figure 2 shows the t-z and q-z functions assumed for the UniPile6 interactive calculations. The simulation of the uniform pile applied a Chin-Konder function coefficient (C_1) equal to 0.0090 and, as mentioned, $Nt = 13$. The simulation of the tapered pile assumed the same target ß-coefficient test and imposed that the Nt would be the same for toe and "donut" response, which resulted in an I = 40, which is not very compatible with a ß = 0.25. Furthermore, to achieve the fit at large movement, the t-z function coefficient (C_1)had to be increase to 0.0085.

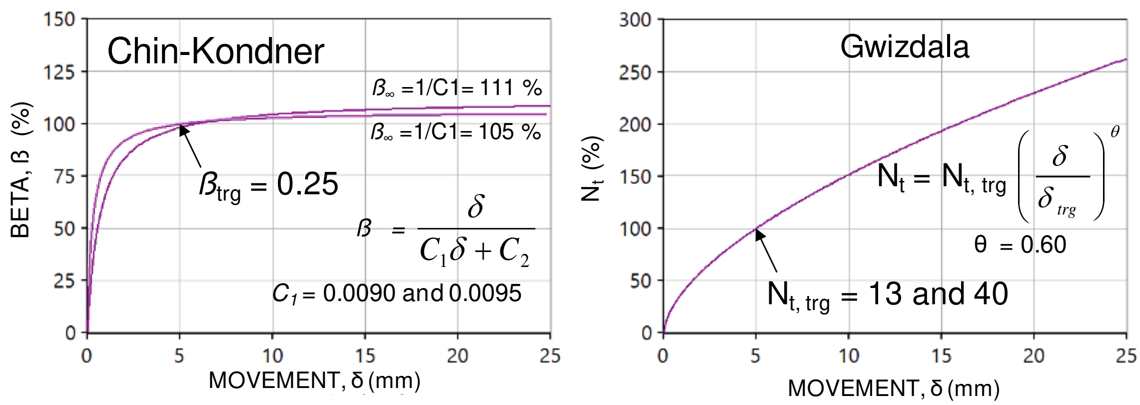

Fig. 2 The t-z and q-z functions applied in fitting the simulations to the measured curves

The UniPile6 software enables extracting the shaft resistance of the uniform and tapered piles for direct comparison of the calculation results. Figure 3 shows the shaft resistance of the piles for movement beyond about 10 mm, indicating that the shaft resistance of the tapered pile had mobilized more than twice that of the uniform pile.

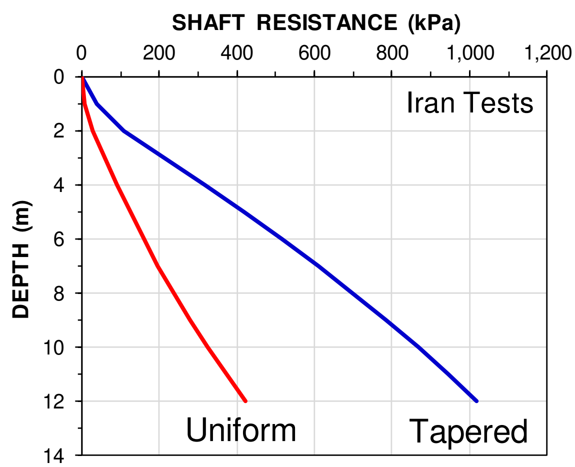

Fig. 3 Distribution of shaft resistance in uniform and tapered piles

Porto Marghera Tests

The third full-scale case is from static loading tests in Porto Marghera, near Venice, Italy, on two pairs of spun piles, 10 m and 15 m long, respectively (Gambini 1973). The tests were in push followed by pull and each pile pair comprised one uniform and one tapered pile, which enabled telling the shaft and toe resistances separately. The piles were driven through an about 5 m thick mixture of loose fine soil deposited on about 3 m of silt on 2 m of soft clay followed by clayey sand and sand. Figure 4 shows the CPTU diagram from a cone pushed at the test site (G. Togliani 2025, personal communication). The groundwater table was at 3 m depth.

The 10-m uniform pile had 330 mm diameter and the 10-m tapered pile had a 390 mm head diameter that reduced to a 240 mm toe diameter. The 15-m uniform pile had 400 mm diameter and the 15-m tapered pile went from a 460 mm head diameter to a 240 mm toe diameter. For both piles, the taper was 15 mm/m; 0.85°. The total shaft area of both tapered piles was equal the that of the matching uniform pile. The pile toe areas were 855 cm^2 and 1,257 cm^2 for the uniform piles and 452 cm^2 for the tapered piles, i.e., the ratios of toe areas of taper pile to uniform pile were 53 and 36 %, respectively.

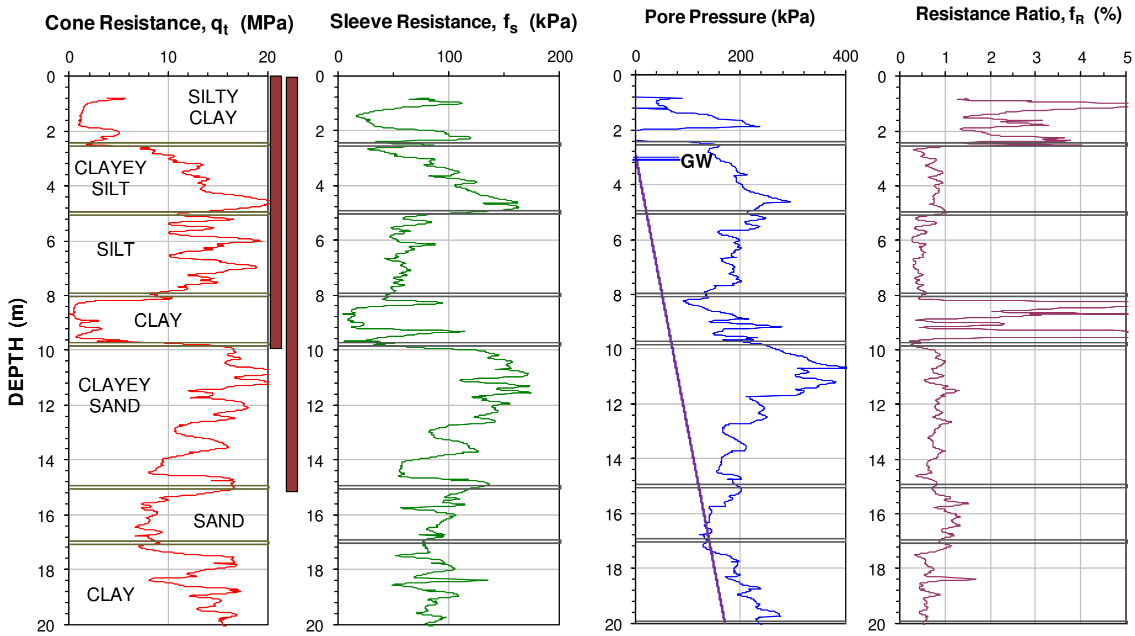

Fig. 4 CPTU diagrams for the Porto Marghera test site (Gambini 1973)

Figure 5 presents the results of the loading tests, comprising head-down tests on the four piles followed by pull tests. The difference of the pull test response may be due to the difference in pile diameter. The pull tests on the 15-m piles incorporated an unloading-reloading event and this distortion may be why no similar difference occurred. The gray curves show the results of UniPile simulation fitted to the curves. A β-coefficient of 0.50 for a 5 mm movement a target point and a Chin-Kondner t-z function with a coefficient (C_1) of 0.0060 (ß = 0.80 at infinite movement) fitted the average pull test result of the 10-m piles. For the 15-m piles, the analysis imposed that the fit must apply the coefficient found for simulation of the 10-m pile to the 10 - 15 m depth. A fit to the average of the measured 15-m pile pull tests curves was obtained for a β-coefficient of 0.12 and a Chin-Kondner t-z function with a coefficient (C_1) of 0.0022 (ß = 0.50 at infinite movement). The fit of the push tests was obtained using the same β-coefficients that gave the agreement of the pull test simulations. For the uniform piles, the fit was obtained by adding target N_t-coefficients (40 and 110, respectively) and q-z functions. (The CPTU records applying the various CPT-methods for calculating ultimate shaft resistance resulted in β-coefficients that were from twice and more larger than the mentioned). For the tapered piles, the fit was obtained by adding the same target N_t-coefficient (95) to both lengths. For the push test of both pile types, the q-z functions indicated plastic toe response, which is likely apparent, only, and caused by presence of significant residual force in the test piles.

Eyeballing the curves and considering the fact that the surface area of the two pile types piles were equal, the taper shape appears to have close to doubled the shaft resistance at the 10 and 15 m pile lengths plotted in Figure 6, which also shows the shaft and toe resistances distributions along the full pile lengths. The dot symbols are the values derived directly from the test records and the curves show the distributions determined in the UniPile simulations. For movement larger than the values chosen as target for the analyses of the records of the tapered piles, the records as well as the simulations, show even larger ratio between the uniform and tapered piles.

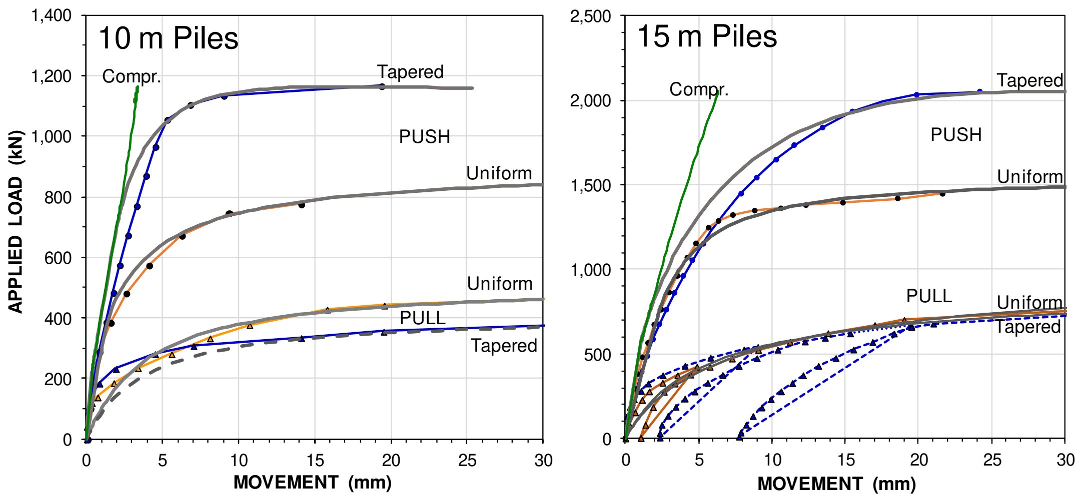

Fig. 5 Comparison between 10 m and 15 m long tapered and uniform spun piles (Gambini 1973)

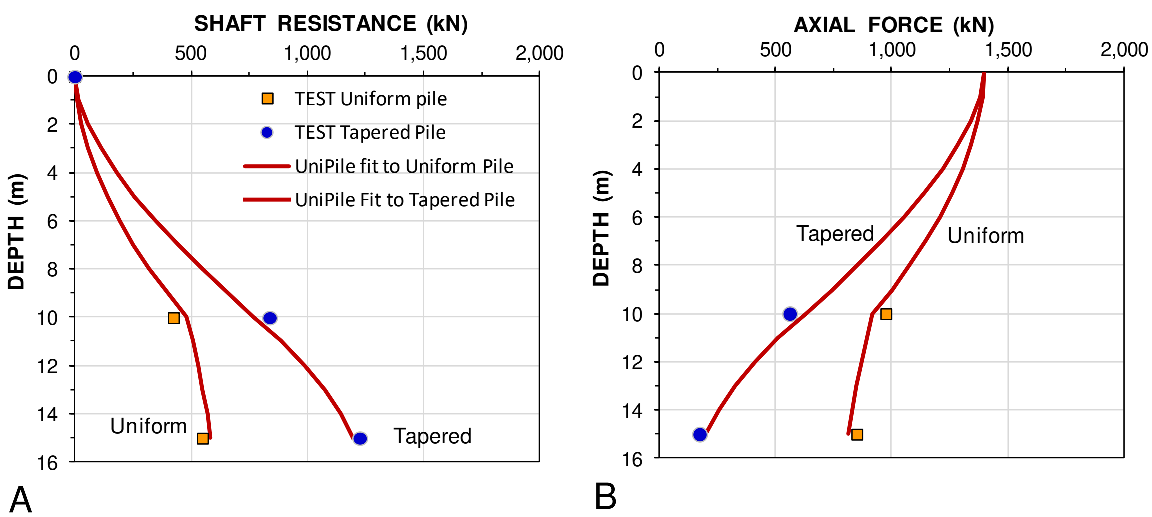

Fig. 6 Shaft resistance and axial force distribution from tests and from back-analysis

Full-scale Tests in Mobile, Al
Piles, instrumentation, soil profile, and loading tests

On March 26. 2025, five test piles, TP1 - TP5 were driven in Mobile, Al, to 17.4 m predetermined depth. The piles were concrete-filled, closed-toe, pipe piles with 9.5 mm wall with 457 mm diameter. Piles TP1 and TP2 were uniform and Piles TP3 - TP5 were TSFP piles, i.e., same diameter down to 7.6 m above the pile toe, from where the section tapered to 203 mm toe diameter, i.e., a taper of 16 mm/m; 1.0°. The steel area, A_{steel}, of the 457 mm section was 169 cm^2. The shaft surface area of the tapered length was 83 % of the uniform pile and the pile toe area was 20 % of the pile toe area of the uniform pile.

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 down in this paper, the concreting operation adversely affected both the strain-gage instrumentation and the bidirectional cell in TP5, bidirectional test pile. The latter to the point that the bidirectional test could not be performed. A head-down test was carried on Pile TP5, instead.

The piles were instrumented by means of vibrating-wire strain-gages Type Geovan Model GV-2410 and full-length compression telltale rods. 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 attempting to field adjust to the 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

But for an about 2-m thick zone of loose gravelly sand with 30 % fines content between 4.6 and 6.4 m depth, the soil consists to 90 % of sand size grains, and the density is 1,850 kg/m^3. The density over and below this zone is 2,050 kg/m^3 and the consistency is compact to about 6.4 m depth, loose to about 16.4 m, and, then, dense. When the static loading tests were carried out, the groundwater table (GW) was at 5 m depth. Figure 7 shows results of a CPT sounding performed at the site. The q_t-graph is supplemented with the distribution of SPT N-indices.

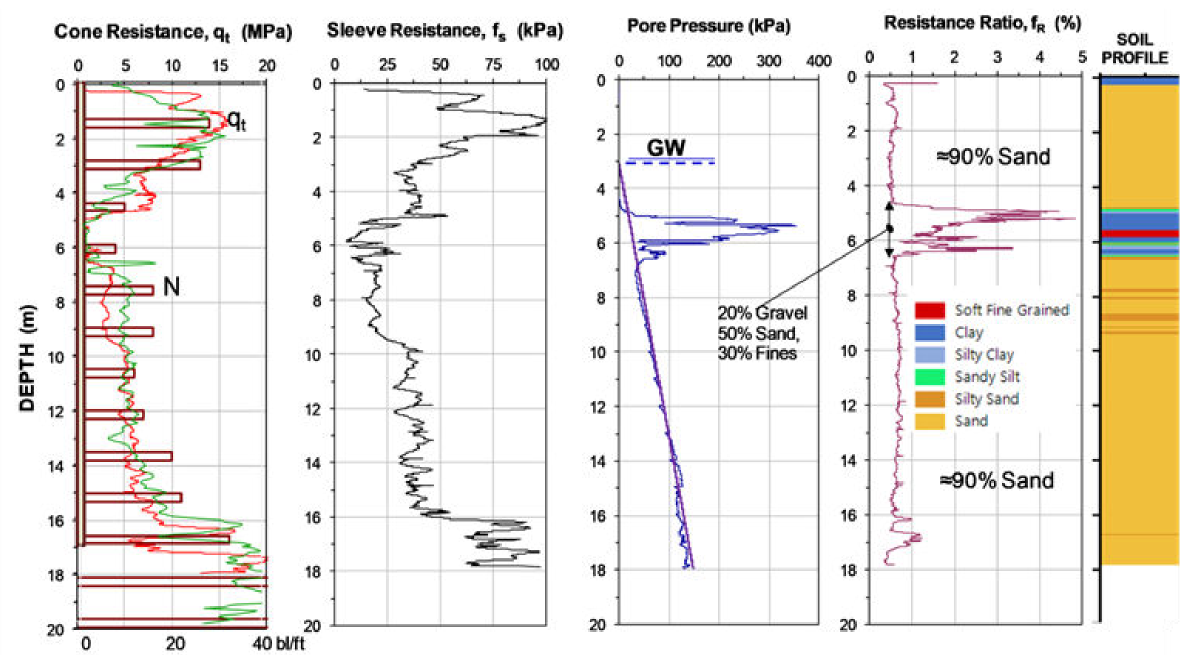

Fig. 7 Soil profile by CPT and SPT records

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. This necessitated placing the jack to load on the pile rim, which, as addressed later, resulted in a progressive loss of adhesion between the steel pipe and the concrete core (delamination) as the test progressed, compromising the strain measurements.

Moreover, piles were to be grouted with a 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 engineers decided to use the delivered concrete. As the instrumentation cage could not be pushed into this stiff consistency concrete, the cages were placed in the empty pipe before placing the concrete in the pile. For unknown reason, no slump test or cylinders were prepared from the concrete and, therefore, the concrete strength is unknown.

Dynamic Tests

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 the 17.4-m predetermined depth. The pile driving was monitored using Pile Driving Analyzer (PDA) during initial driving and at restrike (RSTR1) on March 28. A second restrike (RSTR2) was performed on all test piles; on April 29, 33 days after end of driving.

The CAPWAP determined load-movement curves are compiled in Figure 8A - 8C for Piles TP1 through TP5, respectively, together with the load-movement curves measured in the static loading tests, which were carried out 7, 14, 12, 13, and 18 days after the pile were installed. The CAPWAP determined load-movement curves plot consistently below those measured in the static loading test (red curves).

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.

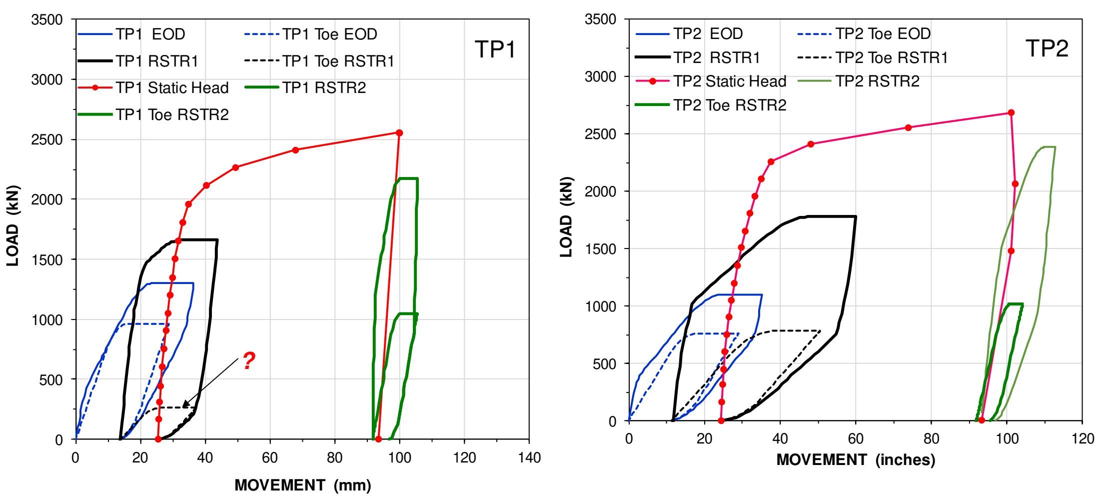

Fig. 8A Piles TP1 and TP2. Load-movement curves of static test and CAPWAP

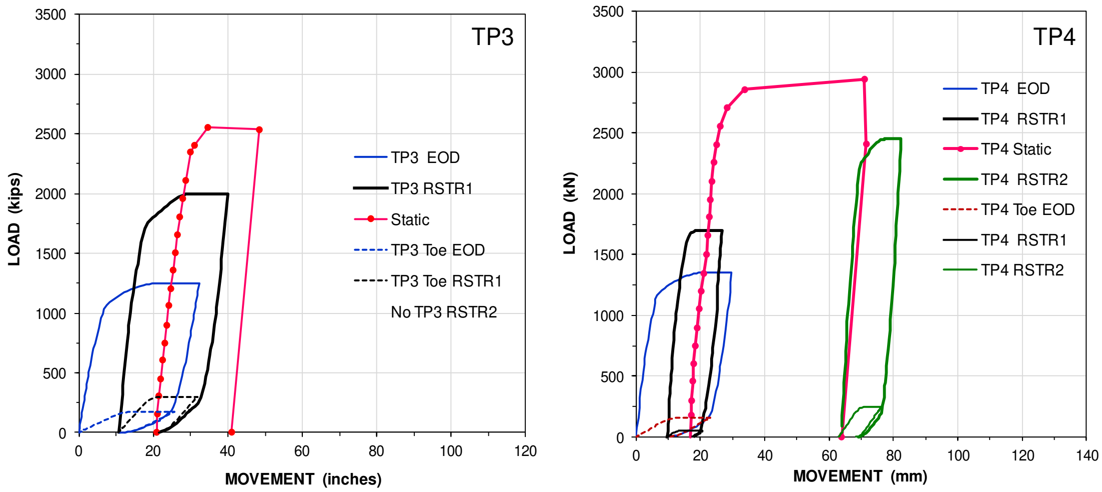

Fig. 8B Piles TP3 and TP4. Load-movement curves of static test and CAPWAP

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.

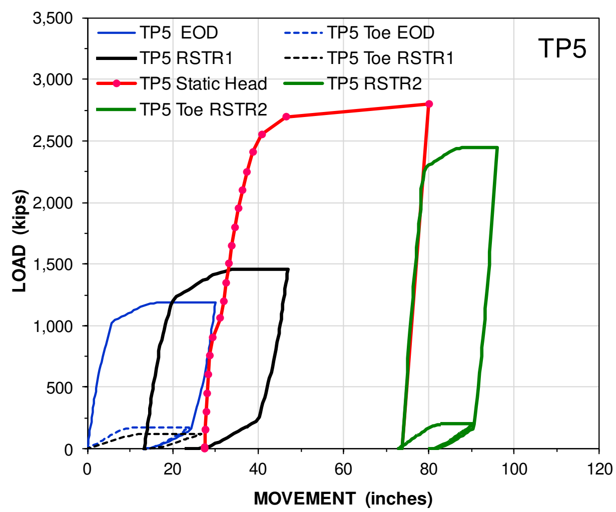

Fig. 8C Pile TP5. Load-movement curves of static test and CAPWAP

Figure 9 indicates a compilation of the static outcome of the EOD 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 CAPWAP analyses indicated that the shaft resistance along the taper length was several times larger for the taper pile opposed as to the uniform pile.

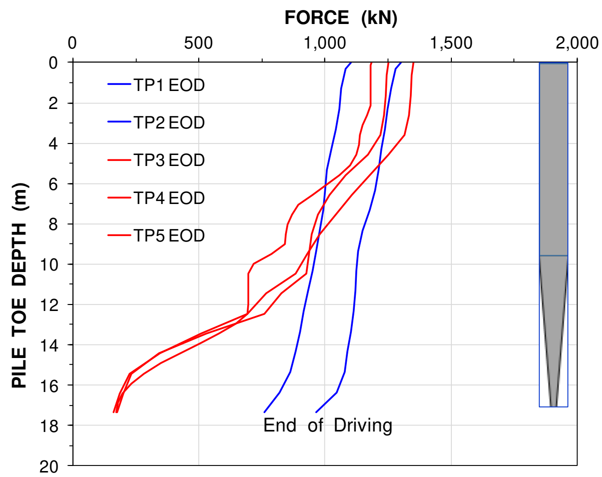

Fig. 9 CAPWAP-determined force distribution for Piles TP1 -TP5 at EOD

Static Loading Tests

The static loading tests were 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. Three free space between mat and pile was 1.3 m. The set-up included measurements to verify that the reference bean was unaffected by the transfer of load from the mat to the pile. The reaction 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 and the movement was 13 mm. For the following tests, additional concrete weights were placed on the platform.

Figure 10 shows the resulting load-movement curves of all test piles (records are from the end of each load increment). Labels TP1 and TP2 denote uniform piles and labels TP3 - TP5 denote taper piles. The curves show that, for movement larger than about 5 mm, the taper piles carried close to 20% more load than the straight piles.

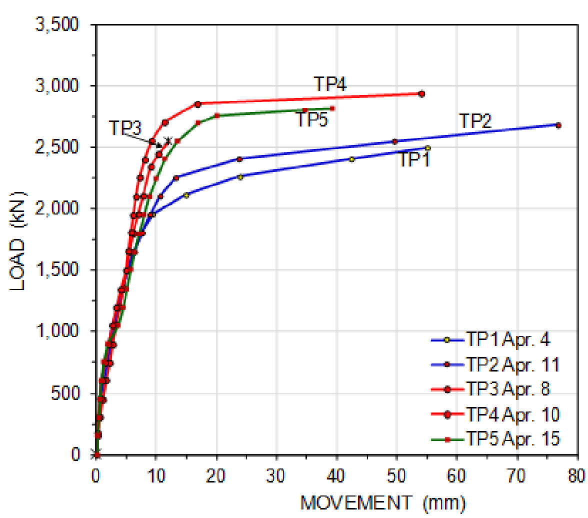

Fig. 10 Load-movements for the five test piles

All gage pairs survived the pouring of the concrete and delivered consistent data representative for the strain developed in the concrete at the gage location. The pouring of the concrete down the pipe was feared to have damaged the gages, but the gage attachment proved to be sturdy enough to take the abuse. However, air pockets might exist throughout the pipe. Moreover, the unfortunate placing of 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 precise records of the concrete strain will now not reliably reflect the average strain in the pile and provide a correct value of the force at the gage level. The delamination caused the uppermost gage level, SG5, to become 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. Moreover, as the axial force down the pile increased, delamination was introduced that became progressively larger as the test proceeded. This removed the reliability of the strain measurements as the data ceased to reflect the average strain and compromised the conversion to force. Moreover, because most of the force in the pile load will be in the steel pipe, the test pile compressed more than a pile with full interaction between steel and concrete.

Figure 11 shows the applied load vs. strain for the piles. Most strain records appear strange. As mentioned, the records of SG5 are of no use. Had SG5 been measuring correctly, it would essentially have shown a straight line with a slope indicating the EA-parameter of the pipe and concrete combination. As SG3 and SG4 appear to develop a reasonable slope, they might at first glance appear delivering reliable records. However, the two curves are too close to each other, indicating unreasonably small change of axial force between the two levels. A reasonable EA-slope is 7.5 GPa corresponding to an E-modulus of 30 GPa considering inside pipe area and presence of the gage cage and guide-pipes. An EA-parameter of 10 GN would infer an E-modulus of 47 GPa. Plainly, such value would be too large.

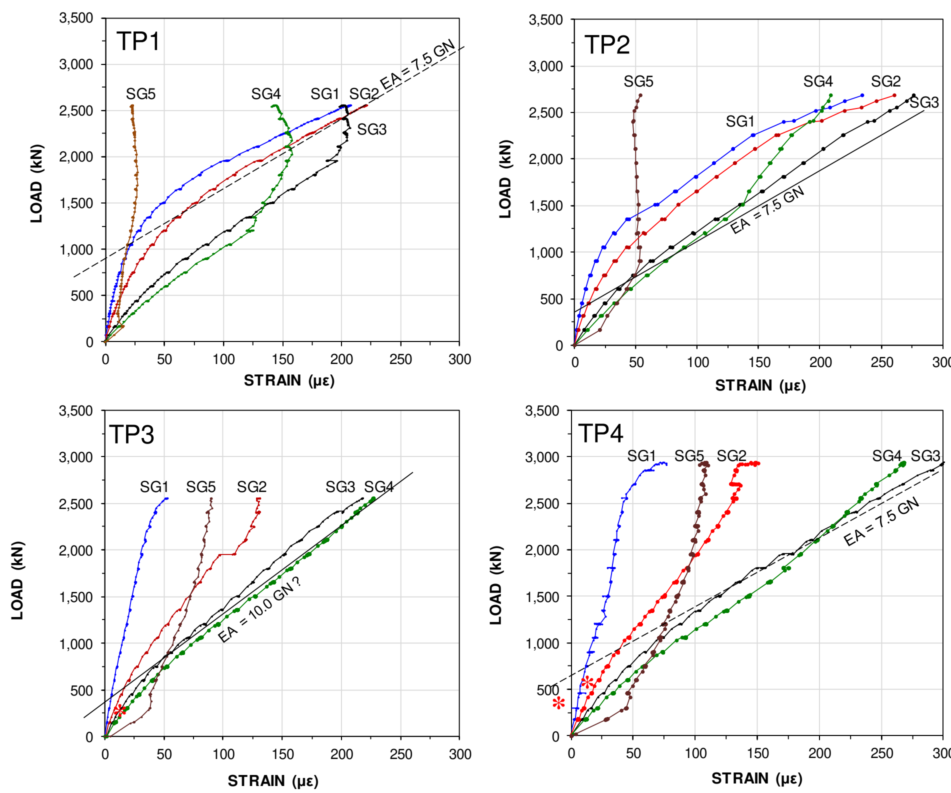

Fig. 11 Applied load vs. measured strain for Piles TP1 and TP2 (uniform) and TP3 and TP4 (tapered)

To obtain a more representative EA-parameter, the strain records can be plotted as change of load over change of strain (i.e., tangent EA-Parameter) versus strain; the plotted values will usually converge toward a more or less constant value. However, there are two important conditions for this to be true: first, the soil response (shaft resistance) must be fully mobilized and truly plastic. If the shaft resistance is strain-hardening or strain-softening, the tangent EA-Parameter will be correspondingly larger or smaller, respectively. This is why a calibration gage level, such as Gage SG5, independent of the soil shear, is necessary. However, depending on the uniformity of the concrete as placed and/or the mineral of the ballast material, sometimes a reducing E-modulus for increasing stress, i.e., a slope, may appear even for the calibration gage level.

The tangent method for determining the EA-parameter is far from exact. It is a differential method and small errors will be magnified and an inconsistent appearance would appear that looks flawed due to the built-in differentiation of the plot. For use to determine the EA-parameter at a gage level, it requires a well executed test without unloading/reloading events and prolonged load-holdings. Definitely, there must be no delamination between concrete and steel.

Figure 12 shows the tangent EA-Parameter versus strain. No record shows a distinct EA-parameter. Choosing EA = 7.5 GN from the load-movement graphs provides a reasonable value although the graphs do not indicate records truly suggesting this value.

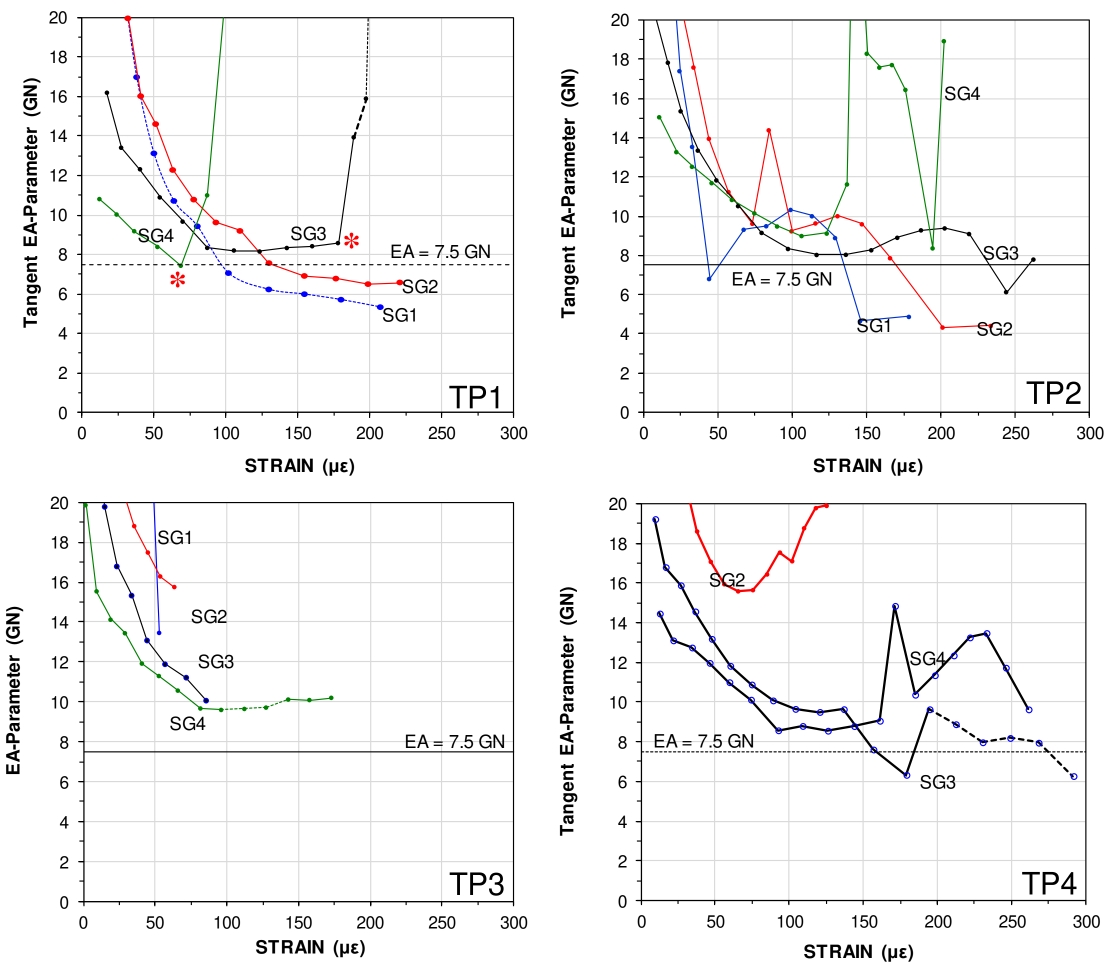

Fig. 12 Change of load vs. change of strain (EA-parameter)

Figure 13 shows the force distributions of Piles TP1 through TP4 calculated from the gage records considered reliable, applying EA = 7.5 and proportional values for SG2 and SG1. For the uniform piles, TP1 and TP2, the force distributions makes sense. SG4 was affected by the delamination, but SG3 values appeared realistic, but for the three last in TP1. The numbers for the toe-gage, SG1, also appeared realistic. In contrast, for the tapered piles, TP3 and TP4, either of SG2 and SG4 is false, or both are. And, the SG1 numbers suggest unrealistically low. It appears that the concrete in the tapered piles is affected by air pockets or similar anomaly resulting from the dumping of the stiff concrete over the instrumentation cage. The concrete strain, albeit accurately measured by the gages, does not likely reflect the true axial force in the tapered piles. After several back-and-force calculations, it became clear that the strain records (but for, possibly, SG1 in TP1) would be best excluded from the back-analysis of the pile response to the applied load.

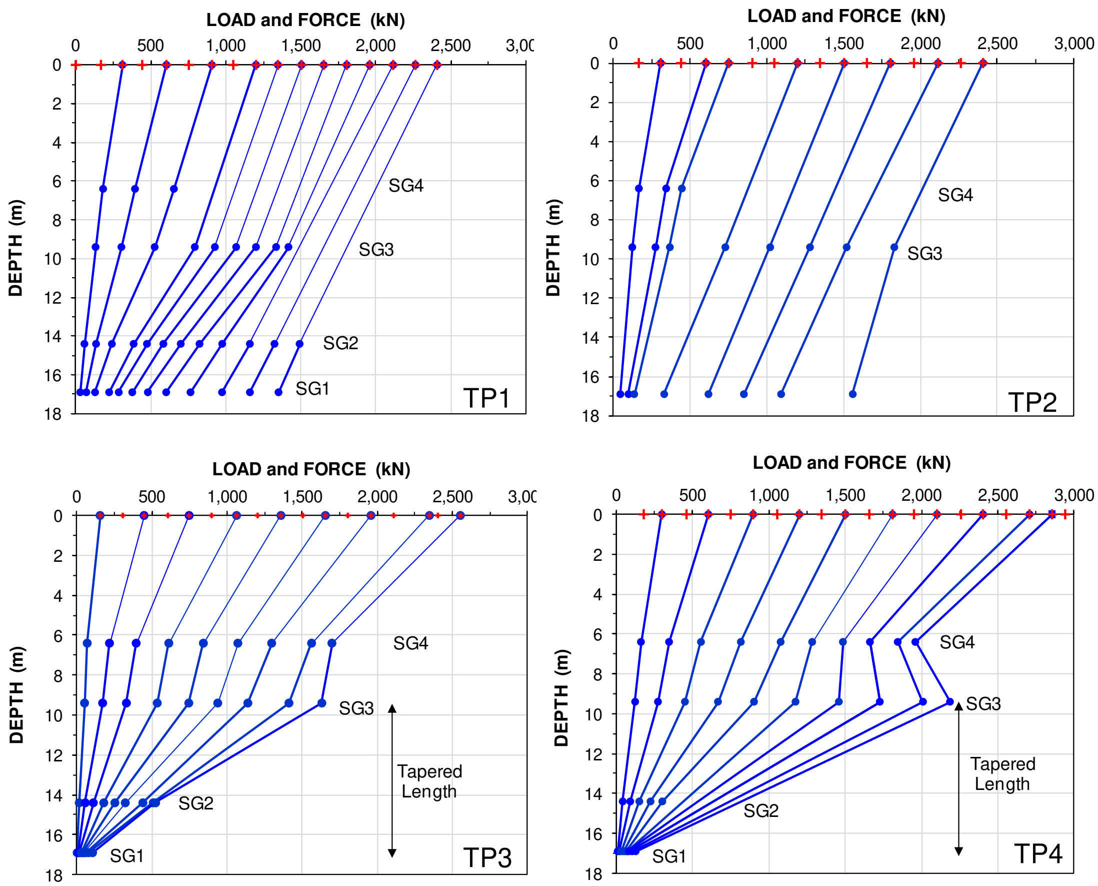

Fig. 13 Force distributions

Test and Analysis Results

The procedure was applied to the test records to obtain a final fit of calculated to measured applied load, pile-toe force, shaft resistance, and pile compression versus movement for the test piles. However, as mentioned, because of the erratic strain records, mainly due to suspected air pockets near the gage levels, the back-analysis disregarded all gage records, but for SG1 in TP1. Figures 14A and 14B show the measured and fitted back-analyzed pile-head movements for TP1 with TP2 added, and for TP3 with TP4 added, respectively. Blue curves are plotted SG-values and red curves are simulated using UniPile6.

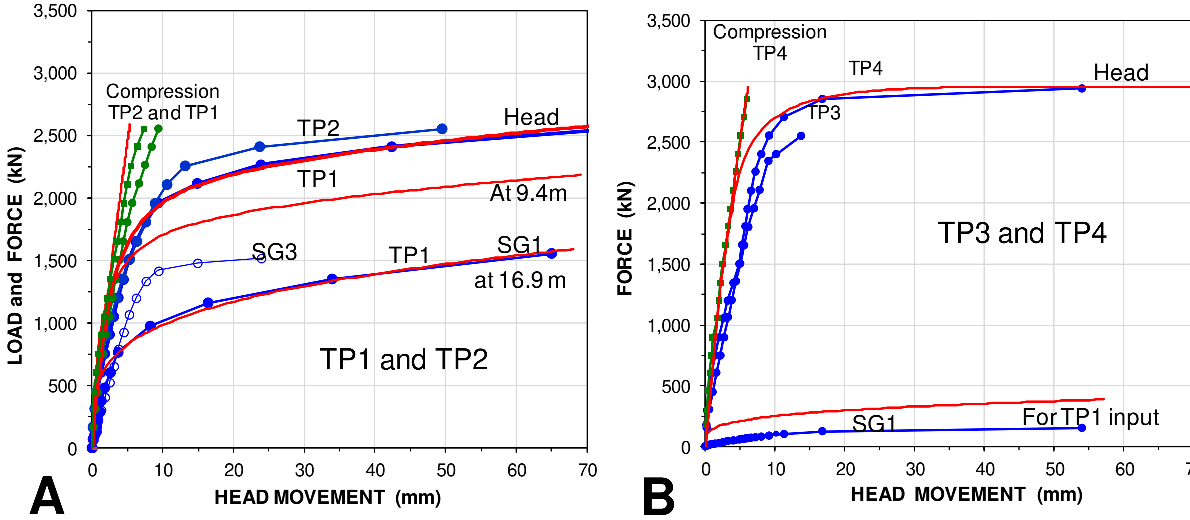

Fig. 14 Pile-head load-movement curves and force-movement curves

Figure 14A includes the simulated curve for the 9.4 m depth and the SG3 curve at that depth. If the latter were correct, in fitting the calculated, and strain-gage independent, pile-head load-movement curve, to the measured, it would indicate an unreasonably low shaft resistance below 9.4 m depth and, consequently, an unreasonably larger shaft resistance above.

The SG1 force-movement of TP4 (Figure 14B) appear to be unrealistically small. Rather than fitting a curve to the gage records, the graph shows the simulated SG1 force calculated applying the input for the uniform pile (TP1) and with the actual TP4 toe area, as the more realistic curve.

Both graphs include measured and simulated pile compression. Note that for TP1 and TP2, the measured compressions are significantly larger than the calculated. This suggests that the concrete is not fully participating in conveying the axial force down the pile due to delamination and that, consequently, the steel pipe carried most of the applied load and, accordingly, compressed more.

The fit to the tapered pile (TP4) applied the same target values of β-coefficients and t-z/qc functions as used for the uniform pile (TP1) and obtained the fit by adding shaft resistance by means of an N_t-coefficient applied to the "donut" area. The q-z function for the "donut" was set equal to the t-z function.

Figure 15 shows the final target coefficients and t-z/q-z functions employed in UniPile6 to fit simulated load- and force-movements to measured. The β-targets were chosen with subjective reference to the CPTU distribution (c.f., Figure 3) for the soil layers defined by the gage depths, but the shaft response was simplified to employing the same t-z function for the entire length of the pile. The N_t-target and q-z function were fitted to the SG1 records in TP1.

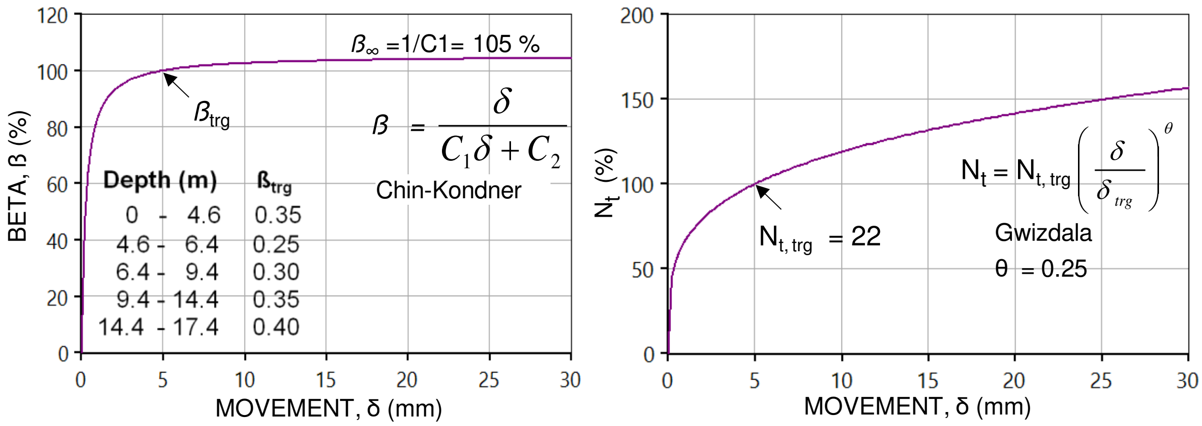

Fig. 15 The t-z and q-z functions and target coefficients used to achieve the fit between calculated and measured load and force response

The simulation for the uniform piles. TP1 and TP2, was based on the fit to the SG1 and the pile-head load-movement records of TP1 applying *ß*-coefficients. The fit to the measured TP1 pile-head load-movement is very good. However, the fit could have been achieved using a different set of ß-coefficients. Indeed, a slightly changed choice of *ß*-coefficients and t-z function would have resulted in a similarly good fit to the TP2 curve.

The t-z function indicates an essentially plastic response of the soil, a not uncommon observation for compact sand. The q-z function has a significantly stiff initial portion as opposed to a more commonly seen less steep curve expressed by a function coefficient, θ, equal to 0.5 or larger. This an indication of presence of residual toe force. If so, and known, the fitted N_t-coefficient would be smaller than the real. The effect of presence of corresponding residual shaft force is disguised by the use of the same t-z function for the full length of the pile made necessary by the uncertainty of the SG2 - SG4 gage records and the loss of the bidirectional test on TP5.

Figure 16 compares UniPile-calculated force distributions for an array of applied load, same for both pile types, demonstrating the taper effect of increasing the shaft resistance more than offsetting the reduced toe area of the tapered pile. The simulation and fit of the UniPile calculation of the tapered piles employed an N_t-coefficient equal to 55 for the "donut" effect combined with the same ß-coefficients as used for TP1 and the same N_t-coefficient (22) as found in the fit of the toe response of TP1. It is likely, however, that the driving of the tapered pile increased the toe response. and, therefore, the N_t-coefficient for the tapered pile should instead be larger and equal to that applied for the shaft element. The dashed curve indicates the distribution calculated with equal (50) N_t-coefficient applied to the "donuts" as to the pile toe of the tapered pile in order to adjust to the associated, then, slightly reduced shaft resistance.

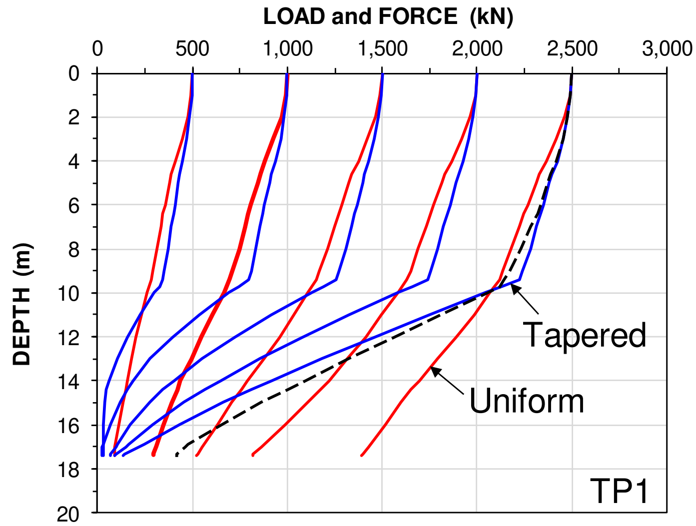

Fig. 16 Force distributions in uniform and tapered piles for different applied loads

Figure 17 applies the fitted parameters in calculating the shaft resistance of the uniform and tapered piles for movement beyond about 10 mm showing that the 16 mm/m; 1.0°-taper resulted in a doubling of the shaft resistance.

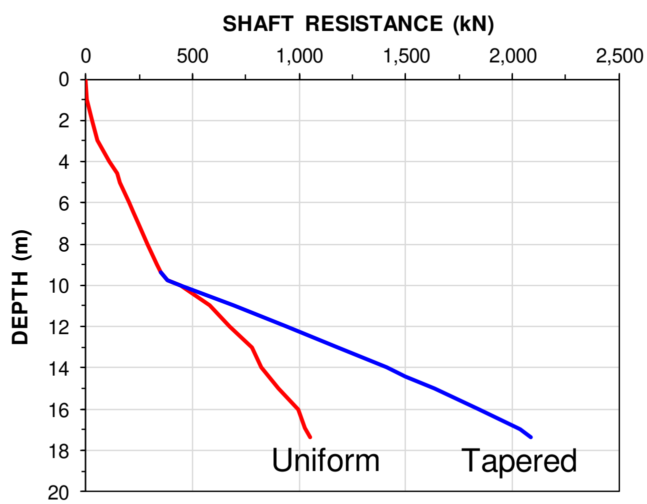

Fig. 17 Distribution of shaft resistance in uniform and tapered piles

SUMMARY

The three full-scale tests confirm the results of the many model tests in regard to the fact that a taper significantly enhances pile bearing. However, the full-scale test records do not allow a detailed analysis of just how the taper mobilizes the larger shaft resistance. For this, the test piles must be instrumented to measure axial force along the pile and, at least, separate shaft resistance from toe resistance with due attention to potential presence of residual force.

The subject full-scale tests showed that the tapered (TSFP) 18-inch piles achieved the same and somewhat larger, bearing as the 18-inch uniform piles, confirming the findings of the model tests and assumed outcome of the two previous full-scale tests.

The 1° taper of the TSFP resulted in a doubling of the shaft resistance along the tapered length as compared to the uniform pile along the same length and depth.

All strain gage-pairs survived the dumping of the stiff concrete into the strain-gage cage was placed and were able to records the concrete strain at the gage level.

Placing the applied load on the rim of the pipe pile resulted in the records of the uppermost gage pair being useless and caused delamination of concrete and steel at the lower gage levels, compromising the strain records as the test progressed.

Placing stiff concrete in the piles as opposed to using the assigned grout resulted in trapping of air pockets in the pile causing uneven concrete cross section area at gage levels and, thus, strain records that indicated unrepresentative force distribution.

Placing stiff concrete in the piles as opposed to using the assigned grout also resulted in loss of means to pursue the bidirectional test (TP5) and establish any presence of residual force in the test piles.

REFERENCES

Dougherty, J.J. 2017. The development of the TAPERTUBE® PILE, DFP foundation products, LLC. http://www.pilelineonline.com/devtt.htm.

El Naggar, M.H. and Wei, J. Q. (1999). Axial capacity of tapered piles established from model tests. Can. Geotech. J., 36(6), 1185–1194. DOI: 10.1139/t99-076

El Naggar, M.H. and Sakr, M. 2000. Evaluation of axial performance of tapered piles from centrifuge tests. Can. Geotech. J., 37(6), 1295–1308

Fellenius, B.H., 2004. Unified design of piled foundations with emphasis on settlement analysis. Honoring George G. Goble—Current Practice and Future Trends in Deep Foundations. Geo-Institute Geo-TRANS Conference, Los Angeles, July 27-30, 2004, Edited by J.A. DiMaggio and M.H. Hussein. ASCE Geotechnical Special Publication, GSP125, pp. 253-275.

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

Fellenius, B.H., 2025b. Static loading tests in Mobile, AL, Straight and tapered piles (TSFP), Final Report, 21 p.

Fellenius, B.H., Brusey, W.G., and Pepe, F., 2000. Soil set-up, variable concrete modulus, and residual load for tapered instrumented piles in sand. ASCE Specialty Conference on Performance Confirmation of Constructed Geotechnical Facilities, University of Massachusetts, Amherst, April 9-12, 2000, GSP94, pp. 98-114, 18p.

Fellenius, B.H. and Ataee, A., 1999. Experimental study of axial behavior of tapered piles. Discussion. Can. Geot. J., 36(6) 1202-1203.

Gambini F,. 1973. Manual on precast r.c.c. piles for foundations. (In Ch. 14 Bearing capacity of cylindrical and conical precast piles. Proc. of XI Geotechnical Conference, Milan, March 1977). 335p.

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.

Gupta P.S.K. and Rajagopal, K. 2015. Review of research on taper and stepped piles. The 6th International Geotechnical Symposium on Disaster Mitigation in Special Geoenvironmental Conditions, Chennai, India, January 21-23, 2015, 325 - 328 pp.

Horvath, J.S. and Trochalides, T., 2004. A half century of tapered pile usage at the John F. Kennedy International Airport. Proceedings of the Fifth Case History Conference on Geotechnical Engineering, NewYork, Paper No.11-02. 8p.

Ibsen L-B and Barari A. 2025. Tapered pile response during large scale axial static and high strain dynamic testing Acta Geotechnica 20(5):2307-2326. DOI:10.1007/s11440-024-02507-5.

Khan, M.K., El Naggar, M.H., and Elkasabgy, M. 2008. Compression testing and analysis of drilled concrete tapered piles in cohesive-frictional soil. Can. Geotech. J., 45 (3) 377–392.

Kodikara, J.K. and Moore, I.D., 1993. Axial response of tapered piles in cohesive frictional ground. Journal of Geotechnical Engineering, 119 (4), 675–693.
doi:10.1061/ (ASCE)0733-9410(1993)119:4(675).

Massarsch, K.R., Westerberg, E., and Broms, B.B. 1997. Footings supported on settlement-reducing vibrated soil nails. XIV, International Conference on Soil Mechanics and Foundation Engineering, Hamburg 97, Proceedings, Vol. 3, 1533-1539.

Nordlund, R. L. 1963. Bearing Capacity of Piles in Cohesionless Soils, ASCE J. 89(SM3), 1-35.

Peck, R. B., 1958. A Study of the Comparative Behavior of Friction Piles, Special Rpt, 36, HRB, Wash., DC. 78 p.

Paik, K., Lee, J., and Kim, D. 2010. Axial Response and Bearing Capacity of Tapered Piles in Sandy Soil. Geotechnical Testing Journal, 34(2) 103-111. DOI: 10.1520/GTJ102761.

Paik, K., Lee, J., and Kim, D. 2013. Calculation of the axial bearing capacity of tapered bored piles. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, 166(5): 502–514. doi:10.1680/geng.10.00127

Robinsky, E.I., and Morrison, C.F. 1964. Sand displacement and compaction around model friction piles. Canadian Geotechnical Journal, 1(2): 81–93.

Rybnikov, A.M. 1990. Experimental investigations of bearing capacity of bored cast-in-place tapered piles. Soil Mechanics and Foundation Engineering, 27(2) 48–52. doi:10.1007/BF02306100.

SACL 2025. Scientific Applied Concepts Limited, Ontario. Pile Loading Test Program, Conventional Static, Bidirectional, and PDA, AL-DOT Site, Mobile, Alabama.

Shafaghat A. and Khabbaz, H. 2020. Recent advances and past discoveries on tapered pile foundations: a review. Geomechanics and Engineering, Vol. 17(2) 1 - 29.

Wei, J.Q. and El Naggar 1998. Experimental study of axial behavior of tapered piles. Can. Geotech. J., 35(3) 641-654. DOI: 10.1139/t98-033.