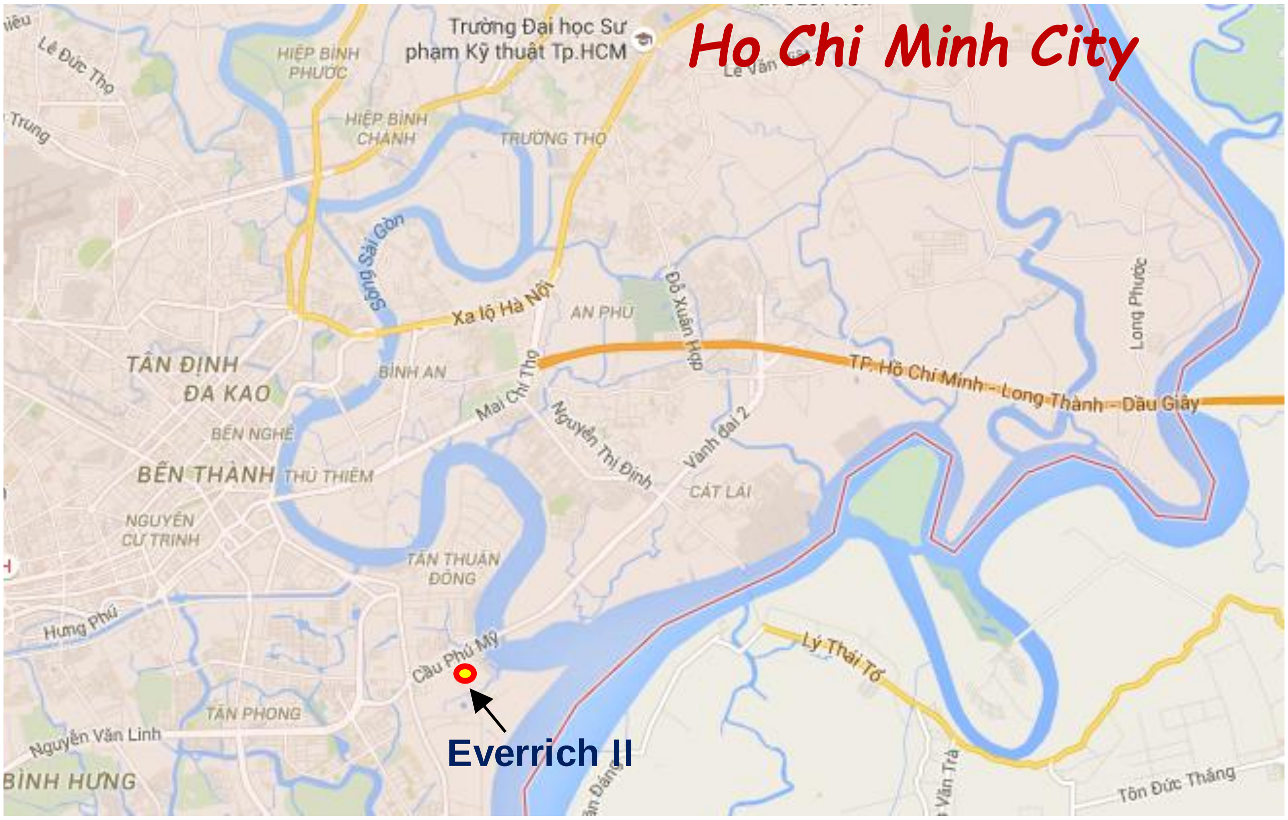

356. Nguyen, M.H. and Fellenius, B.H. Bidirectional cell tests on not-grouted and grouted large-diameter bored piles. Journal of Geo-Engineering Sciences, IOS Press, 2(3-4) 105-117.

Journal of Geo-Engineering Sciences xx (20xx) x–xx
DOI 10.3233/JGS-140025
IOS Press

Bidirectional cell tests on not-grouted and grouted large-diameter bored piles

Nguyen Minh Hai[1)] and Bengt H. Fellenius[2)] P.Eng.
[1)] *Dept. of Civil Engng., University of Texas at Arlington, TX 76019, USA; haitdmu@gmail.com*
[2)] *Consulting Engineer, 2475 Rothesay Avenue, Sidney, BC, Canada, V8L 2B9; Bengt@Fellenius.net*

ABSTRACT The 37-storey apartment buildings of the Everrich II project in HoChiMinh City, Vietnam was designed to be supported on a piled foundation consisting of bored piles each assigned a 22-MN working load. The foundation design included performing bidirectional-cell, static loading tests on four test piles. The soil profile consisted of organic soft clay to about 28 m depth followed by a thick deposit of sandy silt and silty sand with a density that gradually increased with depth from compact to dense, becoming very dense at 65 m depth. In March 2010, the test piles, one 1.5-m diameter pile and three 2.0-m diameter piles, were installed to 80 m through 85 m depth and constructed using bucket drill technique with bentonite slurry and a casing advanced ahead of the hole. The bidirectional-cell assemblies were installed at 10 m through 20 m above the pile toes. The piles were instrumented with pairs of diametrically opposed vibrating wire strain-gages at three to four levels below and six to seven levels above the respective cell levels. After completed concreting, the shaft grouting was carried out along a 20-m length above the pile toe for two piles: the 1.5-m diameter pile and one of the 2.0-m diameter piles. The static loading tests were performed about 34 through 44 days after the piles had been concreted.

The analysis of strain-gage records indicated an average Young's modulus value of about 25 GPa for the nominal cross sections of the piles. The average unit grouted shaft resistances on the nominal pile diameters were about two to three times larger than the resistance along the non-grouted lengths. The measured load distribution of maximum mobilized shaft resistances corresponded to effective stress proportionality coefficients, ß, of about 0.2 through 0.3. The ultimate shaft resistance for the pile lengths below the bidirectional cells reached an ultimate value after about 8 to 10 mm movement, whereafter the load-movement was plastic. The pile toe stress-movement responses to toe stiffness were soft with no tendency toward an ultimate value.

Keywords. Bored piles, bidirectional test, E-modulus, shaft-grouting, shaft movement, toe movement, design length.

INTRODUCTION

The Everrich II project in HoChiMinh City, Vietnam, consists of a series of 37-storey apartment buildings with a height of approximately 152 m covering an about 112,000 m^2 area (about 200m width by 600 m length). Figure 1 shows an architect's view of the complex. The soils at the site are typical for the Mekong Delta basin which is filled in with deposits from the Mekong River and consists of a thick deposit of alternating alluvial soil layers of organic soft clay, compact silty sand with some gravels, and medium dense to dense silty sand, underlain by dense to very dense silty sand (Workman 1977). Regional settlements occur in the area. The building complex was originally designed to be supported on a piled foundation consisting of 1,500 and 2,000-mm diameter bored piles with 80 and 85-m embedment, respectively. The maximum working load (under the tallest building sections) was 22 MN sustained (dead) load per pile. As a part of the design, a test programme was carried out on four strain-gage instrumented piles each constructed using bucket drill technique with bentonite slurry and casing advanced ahead of the hole. Two of the test piles were post-construction grouted along about 20 m above the pile toe. The static loading tests employed the bidirectional cell test method (Osterberg 1998).

Fig.1 Architect's view of the Everrich Project II

SOIL CONDITIONS

The soil profile at the site of the test piles consists of organic soft clay to about 28 m depth, compact silty sand with some gravels to 38 m depth, medium dense sandy silt to 44 m depth, and compact sand to about 60 m depth followed by dense to very dense silty sand to at least 100 m depth. The pore pressure distribution is hydrostatic and corresponds to a groundwater table at 1 m depth below the ground surface. Figure 2 shows the distribution of water content, consistency limits, grain size distribution, and SPT N-indices. The average natural water content was about 70 % to 28 m depth and about 18% below this depth to 100 m depth. The saturated density of soft clay is about 1,600 kg/m^3. The density of silty sand below is about 2,100 kg/m^3.

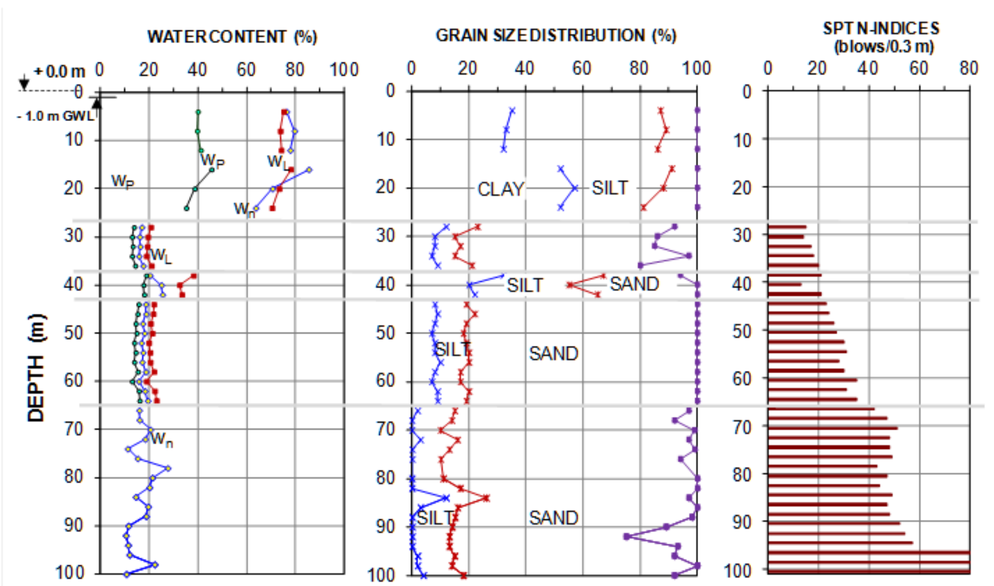

Fig. 2 Water content, plastic and liquid limits, grain size distribution, and SPT N-indices from a representative borehole drilled at the site

CONSTRUCTION OF TEST PILES

Four test piles were constructed: Piles TP1 through TP3 and Pile TPH. Piles TP1 through TP3 had 2,000-mm nominal diameter and Pile TPH had 1,500-mm nominal diameter. Each pile had a reinforcing cage of forty-four 28 mm bars, resulting in a steel reinforcement area of 246 cm^2 and reinforcement ratios of 0.78% and 0.84 % of the 1.77 m^2 3.14 m^2 total nominal pile cross sections, respectively. Two piles, Piles TPH and TP3, were post-grouted over the last 20 m length, as described below. The test piles were the first piles of a Phase 1 construction consisting of a total of 406 piles, which final size and depth were to be based on the outcome of the tests. Figure 3 shows the total foundation plan, Phase 1 boundaries (red bar), and locations of the test piles.

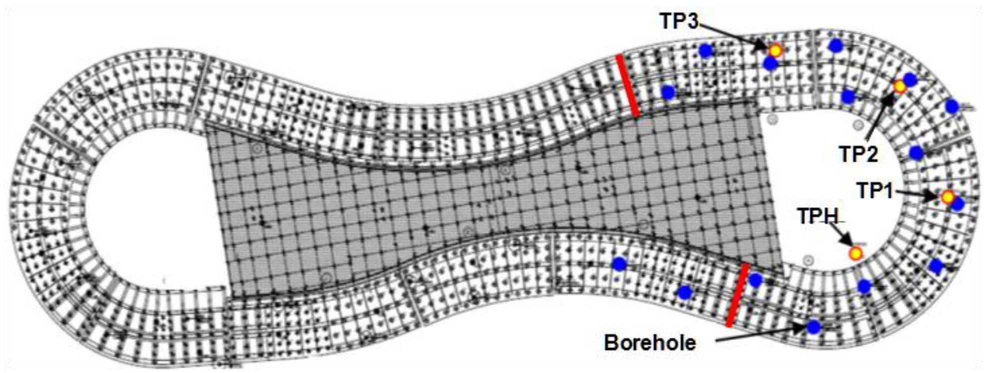

Fig. 3 Layout of Boreholes and Phase 1 test piles

The construction started at the end of January 2010, by installing a casing to 9 m depth. The casing inside diameter was 20 mm wider than the nominal pile diameter. The piles were then constructed using bucket drill and bentonite slurry inside the casing as it was advanced ahead of the drilling. Piles TP1, TP2, TP3, and TPH were drilled to Depths 80.3, 80.6, 84.7, and 80.3 m, respectively, as based on a preliminary design. The reinforcing cage with a bidirectional cell assembly attached was lowered into each test pile 2, 4, 4, and 6 days after drilling, respectively. The cell assemblies were located 20.3, 18.4, 20.6, and 18.7 m respectively, above the pile shaft bottom (pile toe). All four test piles had the same two 670-mm diameter bidirectional cell arrangement. The pore pressure at the cell depths was about 600 kPa for TP1 through TP3 and 700 kPa for TPH. Each reinforcing cage was instrumented with single pairs of diametrically opposed vibrating wire strain-gages at four levels below and six levels above the cell level. Figure 4 shows the layout of gages and cells.

After cleaning the shafts (on the day after the drilling), the reinforcing cage was lowered into the slurry-stabilized hole. Concrete was then delivered via a 300-mm outside diameter, tremie pipe into the bottom of the shaft displacing the bentonite slurry. When the concrete reached the ground surface, the temporary casing was immediately withdrawn.

The volume of concrete placed in the test piles was monitored. Figure 5 shows the nominal and the as-placed actual volumes of concrete versus depth. The actual volume ratios were 1, 3, and 2 % larger than the nominal for Piles TP1 through TP3, respectively, and 4 % for Pile TPH. The volume measurements suggest that the pile cross sections were quite uniform throughout the pile lengths. The concrete 28-day strength was about 65 MPa for all four test piles.

To arrange and facilitate the shaft grouting of Piles TPH and TP3, six 60-mm-diameter grout pipes had been symmetrically attached around the perimeter of the reinforcing cage throughout the shaft length. Over the lower 20 m length, the grout pipes were perforated for grout release and covered by a tight fitting rubber sleeve to serve the post grouting of the shaft. Grouting was carried out by means of inserting a "Tubes-à-Manchette" grouting tube with packers ("manchettes") that allow the grouting to be directed to a specific length (2.0 m) of the grout pipe at a time. A few days after completed concreting, the shaft grouting was implemented by first cracking the pile concrete cover pumping water through the grout tubes at high pressure. That the cracking of the concrete cover had been accomplished was signaled by a sudden drop of the water pressure. The water was then turned off and cement grout was pumped out into the soil surrounding the pile shaft. The grout volumes were not measured.

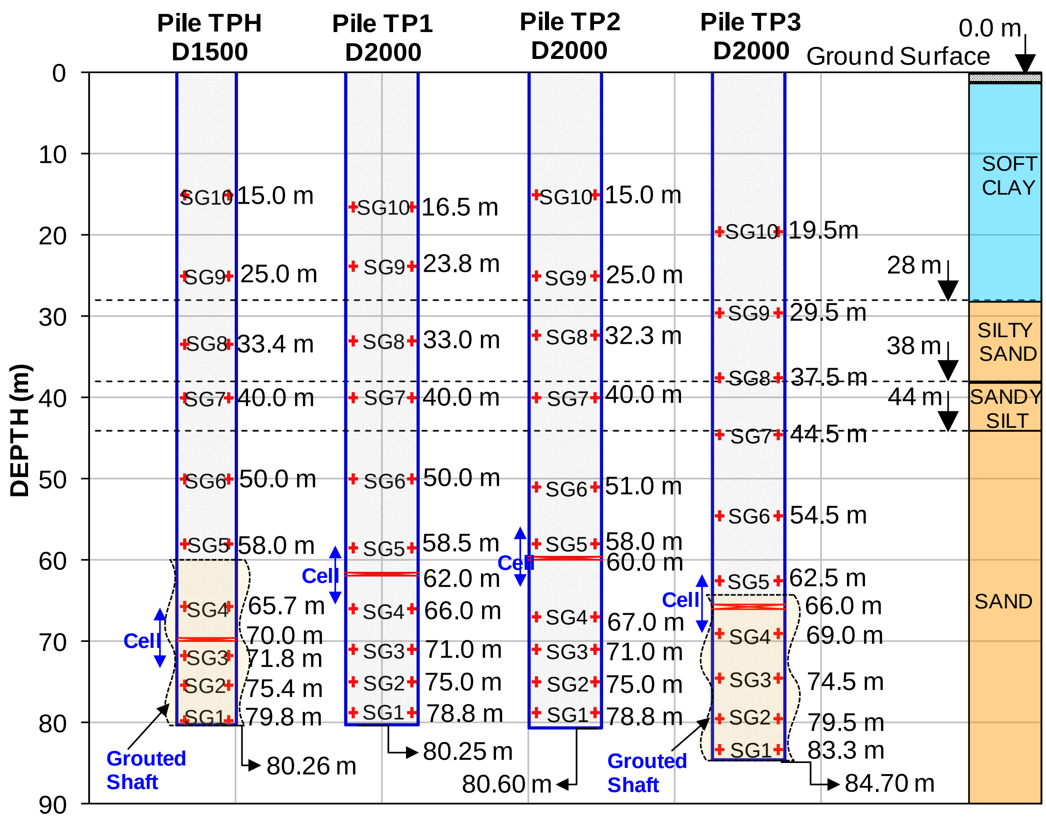

Fig. 4 Depths to bidirectional cells and strain-gage levels in the test piles

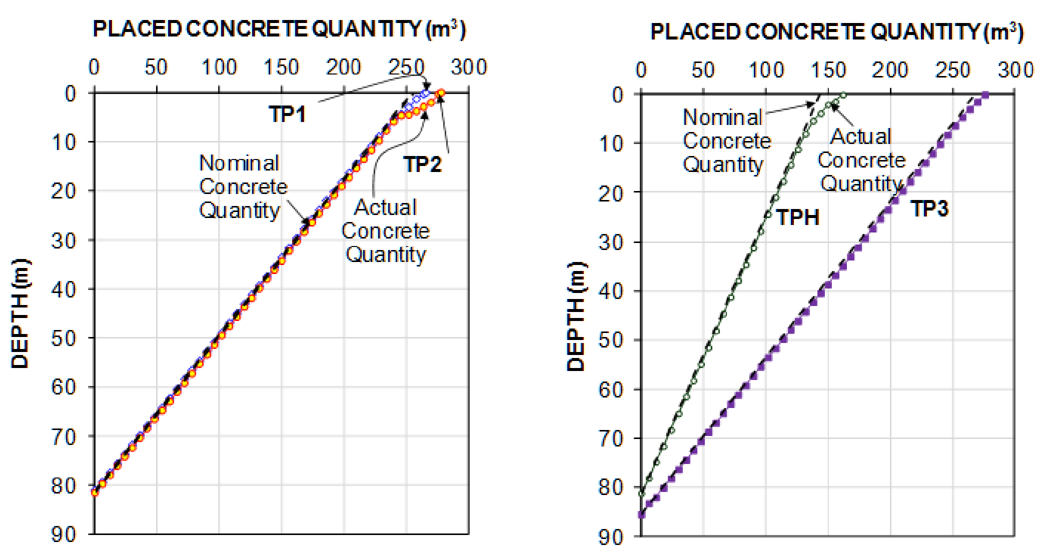

Fig. 5 Nominal and as-placed concrete volumes

TESTS AND MEASUREMENTS
Loads versus Movements

The loading method was performed by raising the bidirectional cell load in a number of equal increments: 1.6 MN for TP1 and 0.8 MN for the other test piles The first 8 to 10 load levels were held constant during 1 hour (See Figure 6). However, the next increment's load-holding time was increased to 6 hours. Thereafter, the load-holding time returned to a 1-hour duration for Pile TP1, TP2, and TP3, but for Pile TPH for which the load-holding duration was 2 hours. For all piles, the last load level was held constant for 24 hours.

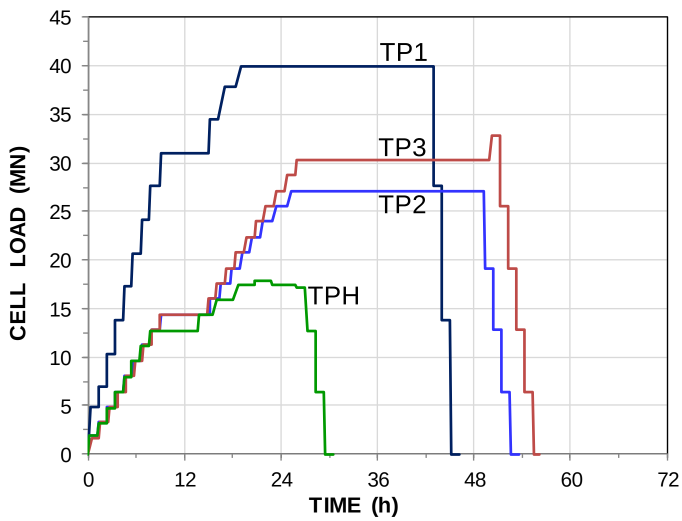

Fig. 6 Loading schedule for the test piles

Nothing is gained by keeping the load level for a longer duration at one or other load levels. It is unfortunate that it yet was made, because such change of load-holding duration adversely affects the evaluation of pile axial stiffness for the load levels following. It could have been worse, the testing schedule could have included unloading/reloading steps, which would have made the analysis of the strain-gage records very challenging and the results disconcertingly ambiguous (Fellenius 2015).

Table 1 compiles the measured maximum loads and movements of the bidirectional cell tests. Figure 7 shows the measured upward and downward load-movement curves of the tests. (The buoyant pile weight has not been subtracted from the measured upward loads and the water force acting on the downward force at the cell level has not been added to the measured downward loads). In neither of the tests was the upward shaft resistance fully mobilized. The downward movement was more than 100 mm for TP1, TP2, and TPH, which suggests that their downward shaft resistance was fully mobilized.

TABLE 1 Loads and movement of the bidirectional tests

Pile	Date	Days after	Maximum	Maximum Movement			
	Tested	Concreting	Cell Load	Upward	Downward	Head	Toe
(#)			(MN)	(mm)	(mm)	(mm)	(mm)
TP1	March 12	38	18.5	5.2	114.2	1.6	109.1
TP2	March 5	34	27.1	16.5	105.7	11.4	100.2
TP3	March 16	41	32.8	10.4	8.9	4.0	2.9
TPH	March-22	44	17.8	16.5	148.5	0.8	142.2

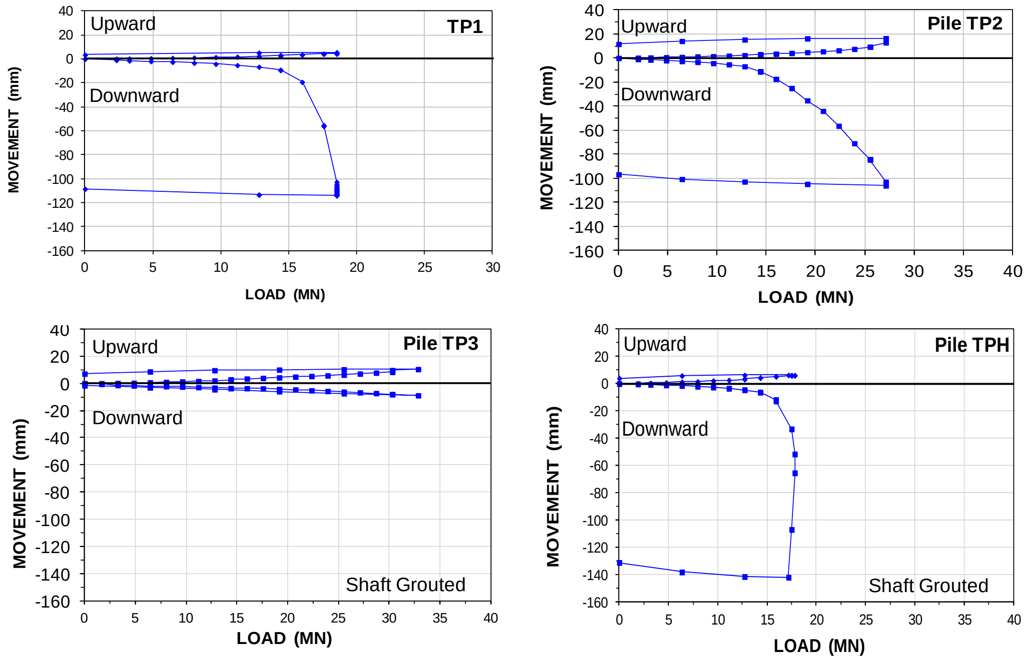

Fig. 7 Measured upward and downward cell-load vs. movement curves

Axial Stiffness of the Test Piles

In order to determine the load values from the strain-gage measurements, knowledge of the pile stiffness, EA ("elastic" modulus, E, times cross sectional area, A), is required. For steel piles, the E-modulus is accurately known and, usually, so is the pile cross section, A. In contrast, the modulus of concrete piles can vary significantly from pile to pile and from concrete mix to concrete mix. A theoretical evaluation of the modulus, E, from, say, the concrete strength or from some other parameter is not very precise. For bored piles, often, the pile cross section is not that well known either. However, the actual stiffness, EA, can be evaluated from the measured loads and strains. Figure 8 shows the measured load-strain curves for the strain-gage levels of the four test piles. Although the load-strain representations trend to linearity, closer look shows that they are curved. Obviously, stress-strain relation and, therefore, also pile stiffness are not constant. A linear trend is indicated, which could be taken as representing an upper-bound average pile stiffness, EA, for the gage levels closest to the cell level.

The average slope of Piles TP1 and TP2 suggests stiffnesses of 80 GN and 84 GN, when correlated to the nominal cross sectional areas of 3.14 m^2, which correspond to an E-modulus of about 25 GPa. It is reasonable to expect that the stiffness of the concrete in the shaft-grouted piles, TP3 and TPH, is the same. Applying a 25 GPa E-modulus to the stiffness averages of the latter piles, 130 and 65 GN, suggests that the shaft grouting has resulted in 30 and 47 % increase of cross section, respectively. That is, the shaft grouting would have resulted in an about 10 to 20-% increase of the pile shaft diameter. Unfortunately, as the grout volumes were not measured, the calculated increase cannot be correlated to the grout volume.

The loading test consists of applying known increments of load to the pile at the cell level. The strain increments induced by the load increments are measured and register the strain due to the portion of the applied load reaching the gage level. Because the shaft resistance is not known, the load reaching the gage level is unknown. However, once the shaft resistance between the gage level and the cell has become fully mobilized (assuming that shaft resistance response is neither strain-softening nor strain-hardening), all the load increments applied next will reach the gage level and the load increase will now be known. The load-strain relation, rather, the load-increments vs. strain increment relation, will be linear and the slope of the line is parallel to the pile tangent stiffness, E_tA.

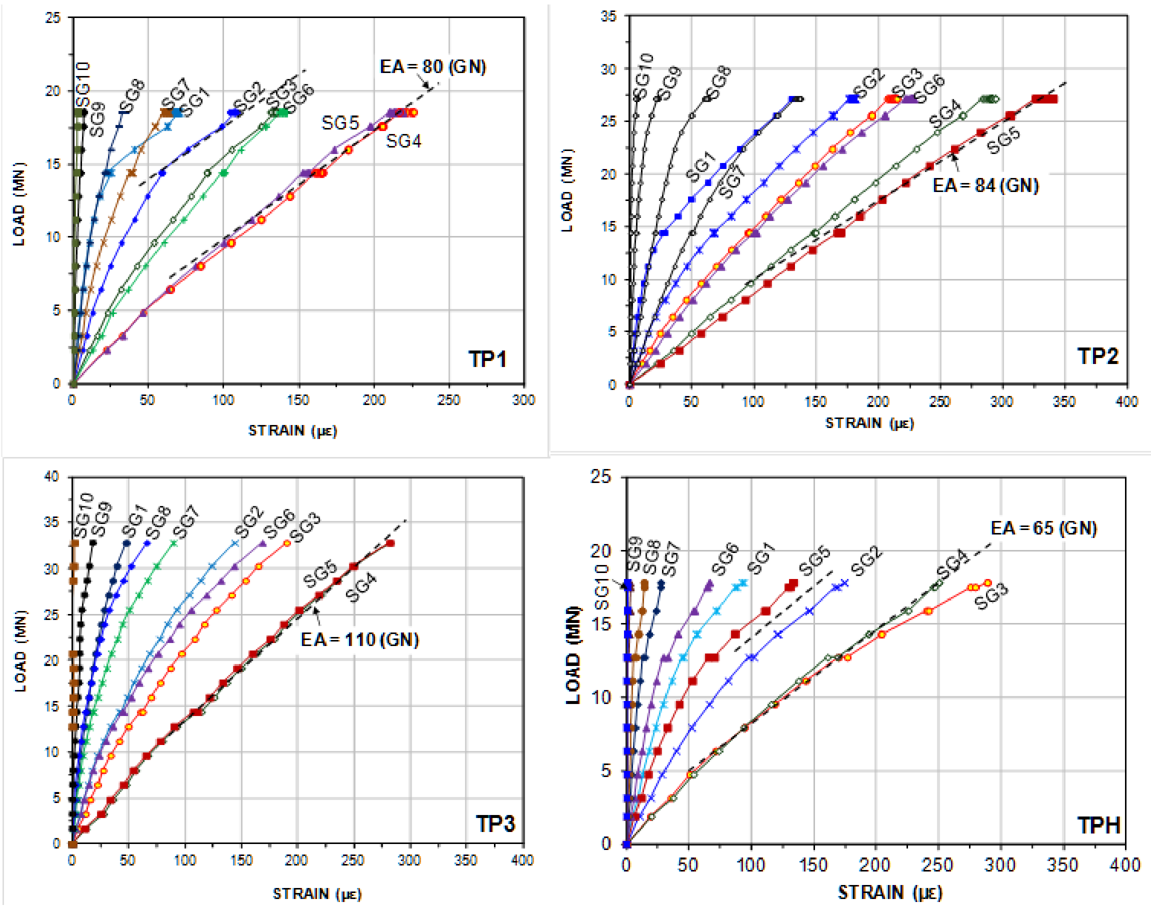

Fig.8 Load versus strain. "SG" stands for "Strain-Gage" pair.

Indeed, the best way of determining the pile modulus is by means of a so-called "tangent modulus" or "incremental stiffness" plot (Fellenius 1989; 2015), that is, the applied increment of load over the induced increment of strain plotted versus the measured strain.

This method aims to determine the incremental stiffness, the tangent modulus, E_t, from which the secant modulus, E_s, can be calculated. It is a differentiation process and, therefore, it is very sensitive to small variations of accuracy of both load and strain measurements, as well as disturbance originating from unequal duration of load-holding. Moreover, it requires that the shaft response shows only moderate strain-hardening or strain-softening tendencies. The incremental stiffness method assumes that, for load increments applied after the shaft resistance at the studied gage level has been fully mobilized, the continued incremental stiffness values will plot along an approximately straight line—slightly sloping, because of the fact that the modulus of concrete reduces with increasing strain. The line defines the tangent modulus relation for the pile cross section. The tangent modulus, E_t, is then converted to secant modulus, E_s.

Because of the combined effect of the strain-softening and the scatter of values, exacerbated by unequal load-holding duration, notably the 6-hour load-holding, the incremental stiffness method did not provide consistent tangent stiffness values. We eliminated some of the variations by disregarding the incremental values for the two sets of measurements obtained immediately after the 6-hour load-holding. The so-adjusted records were used for the incremental stiffness diagrams shown in Figure 9 for Piles TP1, TP2, TP3, and TPH, respectively. Because the maximum strains recorded in the test are rather small, the evaluated relations are uncertain and probably upper-bound values.

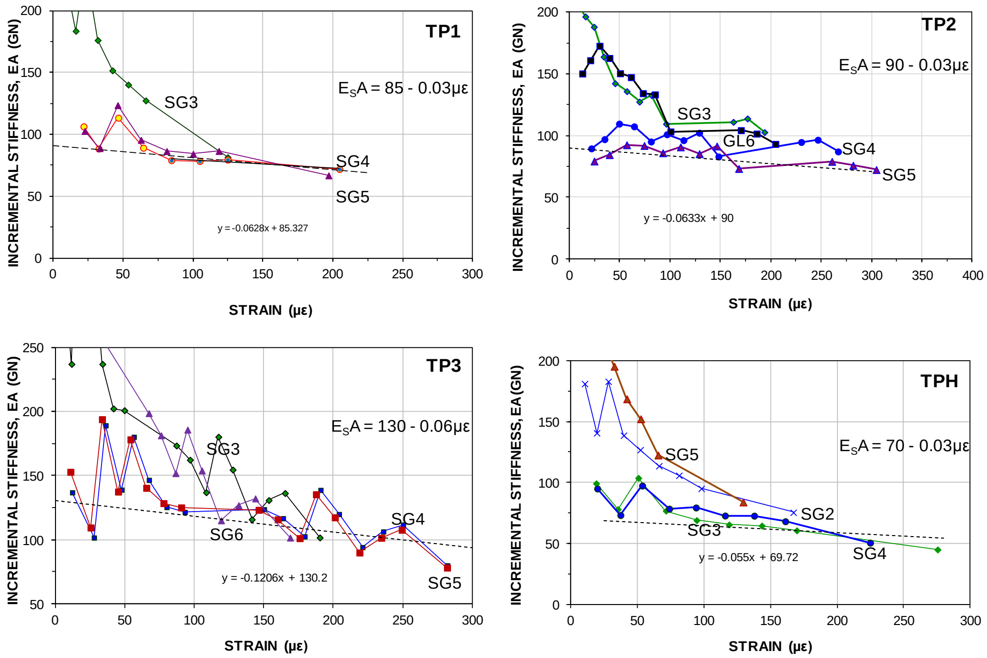

Fig. 9 Incremental stiffness plots with approximate secant stiffness values

Load Distributions

To convert the measured strain data into load, the stiffness (E_sA) values evaluated from incremental stiffness method were used to convert the measured strains to load and load distributions as presented in Figure 10 with adjustment for the water pressure at the cell level. The figures include the equivalent head-down distributions. The latter correlate the bidirectional measurements to the usual manner of presentation of load distribution of conventional head-down static loading tests and is obtained by "flipping over"—mirroring—the bidirectional curve above the cell level. Because the upward shaft resistance was not mobilized in the tests, the head-down distributions do not represent the fully mobilized resistance above about 40 m depth. Also included are the distributions produced in an effective stress analysis fitted to the equivalent head-down distribution.

Figure 11 shows the distributions of the beta-coefficients and unit shaft shear that were applied to achieve the effective stress analysis fit to the measured load distributions. The analyses were made using the UniPile Version 5 software (Goudreault and Fellenius 2013). Where the shaft resistance was fully mobilized, it correlated to a beta-coefficient ranging from about 0.2 through about 0.3 for Piles TP1 and TP2.

Piles TP1 and TP2 are almost "twin" piles. The shaft resistances are similar in magnitude, the difference in total resistance is mostly due to the fact that TP2 mobilized a larger toe resistance, as opposed to Pile TP1 despite their toe movements being essentially equal. (See Table 1). The back-calculated unit shaft resistances below the cell level are about equal. Above the cell level, Pile TP1 mobilized a smaller total shaft resistance than did Pile TP2, which is due to the latter's smaller upward movement, about 2 mm as opposed to 11 mm. The reaction resistance below the cell level limited the amount of the shaft resistance that could be engaged above the cell level.

TP3 and TPH, the two shaft-grouted piles, showed a unit shaft resistance in the grouted zone (from pile toe and 20 m up) that is about 30 to 50 % larger than that of the non-grouted piles, which agrees with the mentioned about 20-% increase of pile diameter implied by the stiffness increase due to the grouting.

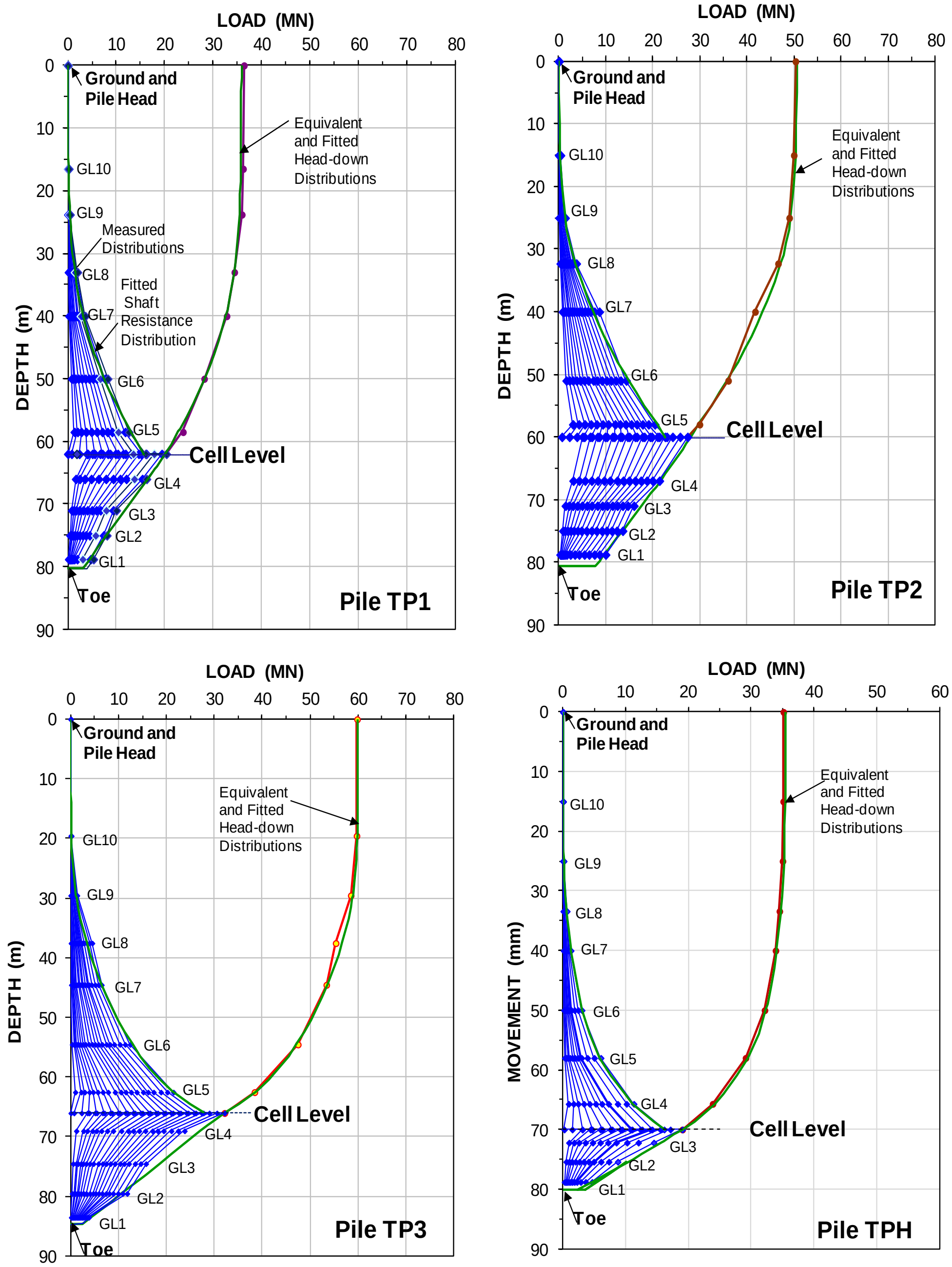

Fig. 10. Load distributions: Measured and as fitted in an effective stress analysis

Shaft shear resistance mobilization

Figure 12 shows the average unit shaft shear resistances between the gage levels versus downward movement, which was obtained by dividing the difference in gage-level loads by the surface area between the gage levels and then plotting against movements at the mid-points between the gage levels (interpolated linearly from the measured movements at the locations of bidirectional-cell assembly, pile head, and pile toe). The figure shows the shaft shear resistance curves of Piles TP1, TP2, TP3 and TPH. The left column of diagrams shows the unit shaft resistances versus downward movement for the gage levels below the cell levels. The right column of diagrams shows the unit shaft resistances versus upward movement for the gage levels above the cell levels.

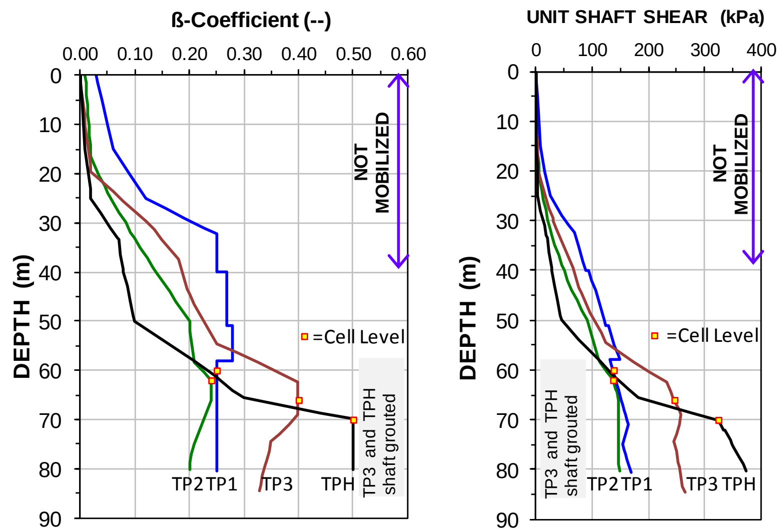

Fig. 11 Distributions of mobilized beta-coefficients and unit shaft shear

For Piles TP1 and TP2, the unit shaft shear resistances below cell were mobilized fully at a movement of about 8 to 10 mm whereafter shaft resistance became plastic. Above the cell level, the movement for TP1 was smaller than 4 mm and the shaft resistance was not fully mobilized. For TP2 between the cell and SG8 (27 m above the cell and 32 m below ground) the upward movement was larger than 10 mm and the shaft resistance was fully mobilized.

We consider it likely that the apparent plastic shear resistance response is caused by excess pore pressures developing along the pile due to the large shear movements beyond about 10 mm. The pore pressure causes a reduction of effective stress and, therefore, reduces the shaft shear. The evaluated beta-coefficients are calculated on effective stress determined from the neutral pore pressure distribution and they may not be true of the actual coefficients.

Toe resistance

The toe resistance was not measured directly, but was determined as the load at Gage Level SG2 (nearest the pile toe) subtracted by the shaft resistance between the gage level and the pile toe, as estimated in the analyses of the test records. Figure 13 shows the toe resistance versus the pile toe movement measured in the tests on Piles TP1, TP2, and TPH. (Pile TP3 toe response is not of interest as the pile showed next to no toe movement in the test). The toe movements are representative for what is usually observed for bored piles in the area (e.g., Fellenius and Nguyen 2013). The curves display no tendency toward an ultimate resistance, confirming the fact that toe capacity does not exist (Fellenius 1999; 2015). The dashed curves show the fit of the q-z Ratio Function (Fellenius 2015) to the respective toe-curves. The fit to the TPH, TP1, and TP2 toe curves has been made for ratio exponents of 0.4, 0.4, and 0.7, respectively. The shaft grouting may have affected the toe response of Pile TPH.

FOUNDATION DESIGN BASED ON THE TEST RESULTS

The design was eventually made for 2,000-mm diameter piles. To ensure adequate pile capacity, the project piles were constructed with shaft-grouting along the lower 20 m length. Because of the ongoing regional settlement in the area and the planned intermittent placing of up to 2 m thick permanent fill at the site, it was important that the neutral plane—the force equilibrium and settlement equilibrium depths—be located below about 60 m depth, that is, below the compact sand and into the dense sand layer, which was not expected to settle appreciably. This required that the uncertain toe response shown for the tests, be compensated for by extending the pile embedment length to 95 m.

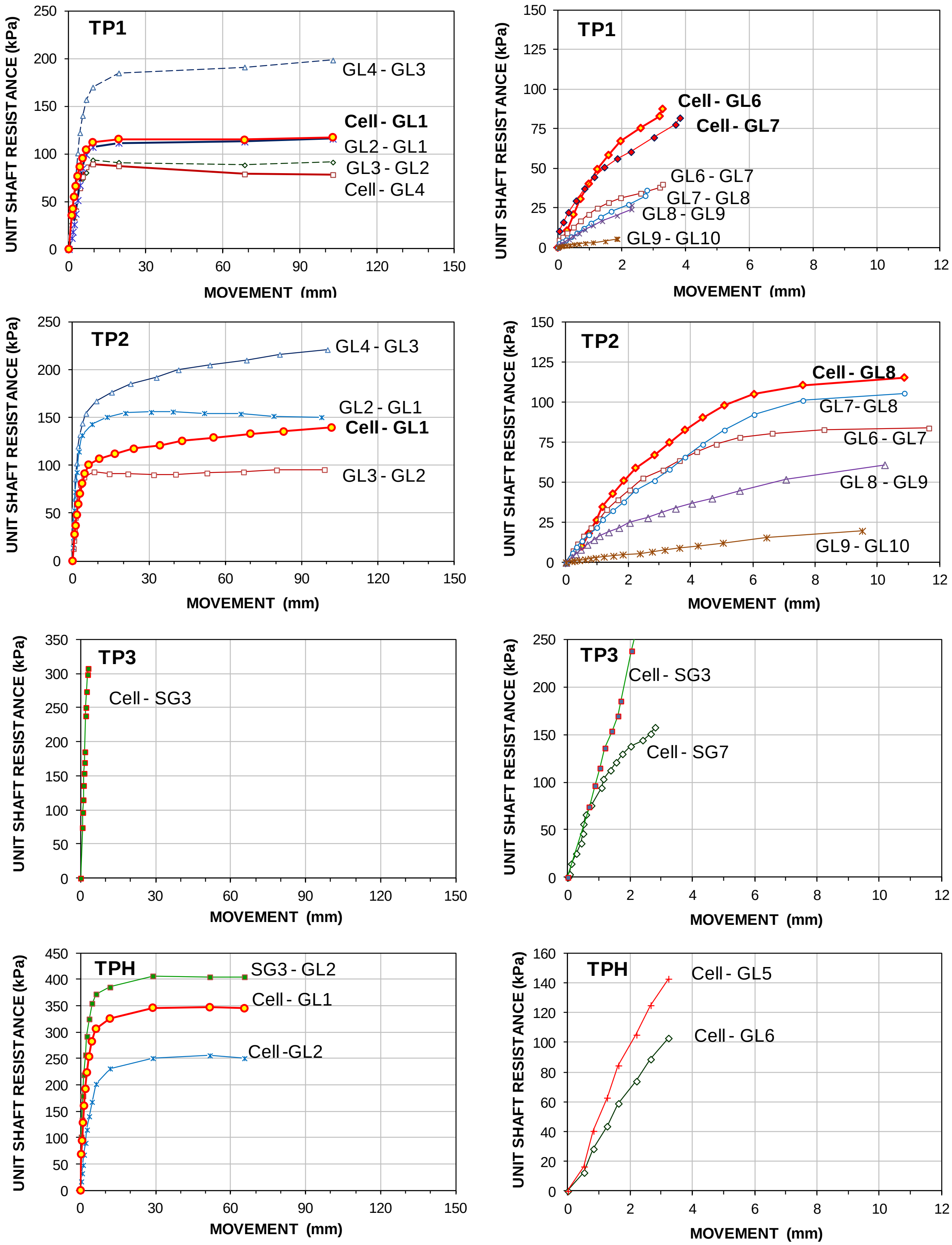

Fig. 12 Unit Shaft Resistances

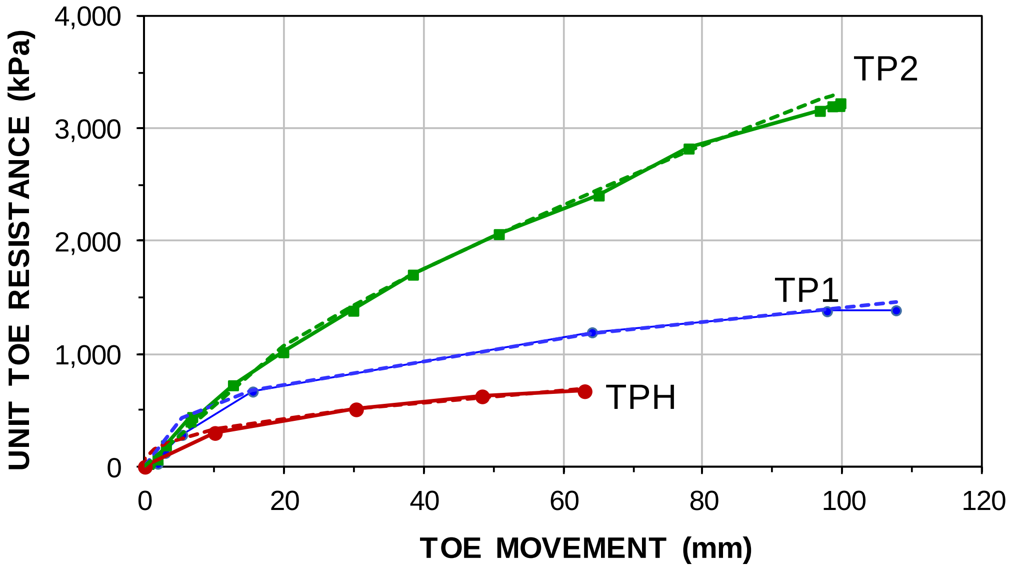

Fig. 13 Toe stress versus toe movement for Piles TP1, TP2, and TPH

SUMMARY

The pile secant modulus, E_s, ranged from about 28 GPa at small strain through about 24 GPa at a strain beyond about 300+ $\mu\varepsilon$.

The effective stress analysis fitted to the evaluated equivalent head-down load distributions showed that the beta-coefficients in the sand ranged from 0.2 through 0.3.

The ultimate shaft resistance for the pile lengths below the bidirectional cells reached an ultimate value after about 8 to 10 mm movement, whereafter the load-movement was plastic, which may have been caused by excess pore pressures developing for movement beyond about 10 mm.

The shaft grouting of the lower 20 m length of the pile resulted in significant increase of shaft resistance as compared to that of non-grouted piles.

The pile toe stress-movement showed no tendency toward an ultimate value. The pile toe resistance was very soft and indicated that debris might have been left at the bottom of the pile shaft after cleaning the slurry and the tremie concreting. For the final design, therefore, toe resistance was disregarded and each pile was shaft grouted along the lower 20 m length.

The equivalent head-down distributions for the test piles indicated that, for piles similar to the test piles, if the contribution from pile toe resistance is disregarded, even for shaft-grouted piles, the neutral plane for the long-term conditions would be located in settling soil layers. For the final design, therefore, the pile lengths were extended to about 95 m depth, which resulted in a neutral plane at a depth of about 60 m, where the long-term settlements were considered to be minimal.

ACKNOWLEDGEMENTS

The bidirectional-cell tests were performed by Loadtest Asia Pte. Ltd. under the direction of Mr. Khoo Han Sen.

REFERENCES

Fellenius, B.H., 1989. Tangent modulus of piles determined from strain data. ASCE, Geotechnical Engineering Division, the 1989 Foundation Congress, F.H. Kulhawy, Editor, Vol. 1, pp. 500-510.

Fellenius, B.H., 1999. Bearing capacity—A delusion? Deep Foundation Institute, Hawthorne, NJ, Proc. of Annual Meeting, Dearborn, MI, Oct. 14-16, 1999, 17 p.

Fellenius, B.H., 2015. Basics of foundation design, a text book. Revised Electronic Edition, [www.Fellenius.net], 432 p.

Fellenius, B.H., and Nguyen, M.H., 2013. Large diameter long bored piles in the Mekong delta. International Journal of Case Histories, 2(3) 196-207.

Goudreault P.A and Fellenius B.H. 2014. UniPile 5 Version 5, Users and Examples Manual. UniSoft Geotechnical Solutions Ltd. [www.UniSoftLtd.com]. 120 p.

Loadtest International Pte. Ltd., 2010. Reports on Bored Pile Testing, Everrich II, HCMC, Vietnam, LTI-2860-1, 2860-2, 2860-3 and 2860-4, 105 p, 111p, 112p and 97 p.

Osterberg, J.O., 1998. The Osterberg load test method for drilled shaft and driven piles. The first ten years. 7th International. Conference on Piling and Deep Foundations, Deep Foundation Institute, Vienna, Austria, June 15-17, 1998, 17 p.

Workman, D.R., 1977. Geology of Laos, Cambodia, South Vietnam, and the eastern part of Thailand. Overseas geology and mineral resources, 50, Her Majesty’s Stationery Office, London, 33 p.