



Bidirectional-cell tests on two 70 m long bored piles in Vietnam

Nguyen M.H. and Fellenius, B.H., 2014. Bidirectional-cell tests on two 70 m long bored piles in Vietnam. *GeoInstitute Geo Congress*, February 23 – 26, ASCE, Reston, VA, Magued Iskander, John E. Garlanger, and Mohamad H. Hussein, editors, *Geotechnical Special Publication, GSP 233*, pp. 482-496.

Bidirectional-Cell Tests on Two 70 m Long Bored Piles in Vietnam

Nguyen Minh Hai¹⁾ and Bengt H. Fellenius²⁾ M.ASCE, P.Eng.

¹⁾ Faculty of Civil Engng, Thu Dau Mot Univ., Binh Duong Province, Vietnam; haitdmu@gmail.com

²⁾ Consulting Engineer, 2475 Rothesay Avenue, Sidney, BC, Canada, V8L 2B9; Bengt@Fellenius.net

ABSTRACT Static loading tests using single-level bidirectional-cells were carried out in mid-2008 on two bored piles constructed at the Binh Loi Bridge, crossing the Saigon River at Ho Chi Minh City, Vietnam. The soils consist of surficial layers of soft clay and silt to 11 and 15 m depth on compact silty sand to about 55 m depth followed by dense to very dense silty sand. Two bored piles, 2.0 m and 1.5 m in diameter were installed to 73.5 m and 73.0 m depth, respectively, and constructed using reverse circulation technique and bucket drill with casing advanced ahead of the hole. Drilling slurry was composed of polymer slurry for the 2.0-m diameter pile and bentonite slurry for the 1.5-m diameter pile. For both piles, the cell assembly was attached to a reinforcing cage lowered into the stabilized hole to 11.4 and 11.6 m, respectively, above the pile toe, 5 and 2 days after completing drilling and cleaning, respectively. Each reinforcing cage was instrumented with two pairs of diametrically opposed vibrating wire strain-gages at three levels below and five levels above the cell level. The static loading tests were performed 49 and 28 days, respectively, after the piles had been concreted.

The strain-gage records, when based on the nominal pile diameters, indicated Young's modulus values of about 25 GPa for the nominal cross sections of the piles. Analysis of the records showed that residual load had developed during the wait time. The unit shaft resistance above the cell for the 2.0-m diameter, polymer-slurry constructed pile was moderately larger than that for the 1.5-m diameter, bentonite-slurry constructed pile. Detailed analysis showed the shaft resistance to be post-peak softening. The pile toe stress-movement responses were essentially linear and almost identical for the two piles.

INTRODUCTION

Static loading tests using single-level bidirectional-cells tests were carried out for two bored piles constructed using different drilling techniques at the Binh Loi Bridge, a cable-stayed bridge crossing the Saigon River at Ho Chi Minh City, Vietnam. The Binh Loi Bridge is a part of Tan San Nhat – Binh Loi Outer Ring Road Project connecting Tan Son Nhat International airport at Ho Chi Minh City to the city outer ring road. The Binh Loi Bridge will link up the Binh Thanh to Thu Duc district of Ho Chi Minh City across the Saigon River. It has a total length of 957 m with a 150 m long main span. The height above the river water level is 15 m.

The bridge is supported on long bored piles constructed through deltaic alluvial soils, consisting of an about 40 m thick deposit of alternating layers of soft clay, compact silty sand, and soft to firm sandy clay, underlain by compact silty sand, becoming very dense at 70 m depth. The main span is supported on 2,000 mm diameter piles and the approach spans on 1,500 mm diameter piles. The working loads were 10 MN and 6 MN, for the 2,000 mm and 1,500 mm diameter piles, respectively. As the assigned factor of safety was 2.5, the required pile capacities were 25 MN and 15 MN, respectively.

To verify that the design of the pile support is satisfactory, static loading tests were performed on one test pile of each diameter, constructed at opposite banks of the river, and 338 m apart, at a pier location nearest the river. The 2,000 mm diameter test pile, Pile D2000, was constructed on the east bank and the 1,500 mm diameter test pile, Pile D1500, on the west bank (Figure 1). The piles had almost the same embedment depth, 73.5 m and 73.0 m, respectively. Both piles were strain-gage instrumented. The static loading test was performed by means of the bi-directional-cell test (Osterberg 1998). The results of the test are presented and correlated to the site conditions. The test data and back-analysis are considered to be of interest beyond the design of the foundations for the Binh Loi Bridge.

SUBSURFACE CONDITIONS

The Mekong River basin is composed of a Quaternary soil overlaying a Tertiary soil (Workman 1977). The upper part of the Mekong basin is filled in with recent deposits from the Mekong River and regional settlements occur in the general area. Pre-Holocene Cenozoic deposits frequently crop out, forming terraces along the Dong Nai and Saigon Rivers. The Saigon River has a total length of 256 km and cuts through an estuary delta stretching from Dau Tieng Lake to Dong Nai River confluence. The seasonal variation of the river water surface elevation at the site is about 3 m.

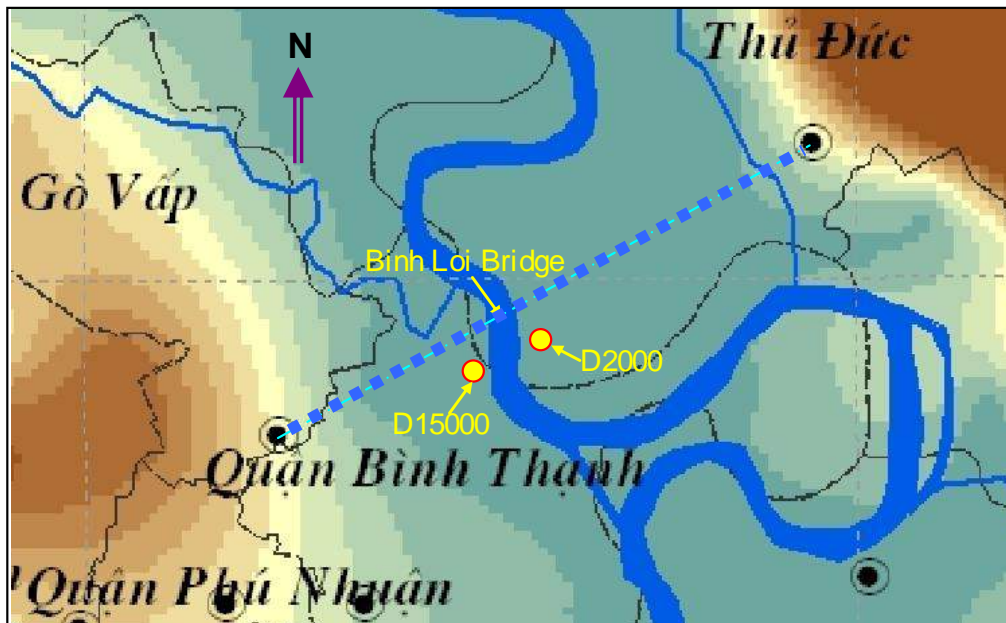


Fig. 1 Location of test piles and Boreholes

The soil profile at the locations of the test piles (Piles D2000 and D1500) consists of surficial layers of soft clay and silt to 11 m and 15 m depths, respectively, underlain by compact silty sand to about 55 m depth followed by dense to very dense silty sand (Figure 2). At Pile D2000, a zone of clayey silt was encountered between 30 to 40 m depth in the borehole. Table 1 summarizes the soil parameters for the soil layers. The pore pressure distribution is hydrostatic and corresponds to a groundwater table at 3 m depth. Figure 3 shows the measured distribution of cone stress, sleeve friction, and U2 pore pressure from two CPTU soundings pushed close to the test pile locations. The soundings were terminated once the cone encountered the compact silty sand. Figure 4 shows the distribution of SPT N-indices close to each test pile location

TEST PROGRAMME

Details of the Test Piles and Instrumentation

The two pre-production test piles, Pile D2000 and D1500, were drilled on May 20, 2008, and July 22, 2008, to 73.5 m and 73.0 m depth below grade, respectively. The diameter of Pile D2000 was 2,000 mm and the pile was supplied with a reinforcing cage of forty-four 32 mm bars, resulting in a steel reinforcement area of 350 cm² and a reinforcement ratio of 0.8 % of the 3.14 m² total nominal pile cross section. The diameter of Pile D1500 was 1,500 mm and the pile was supplied with a reinforcing cage of thirty-two 32 mm bars, resulting in a steel reinforcement area of 260 cm² and a reinforcement ratio of 1.5 %, of the 1.77 m² total nominal pile cross section. The concrete cylinder strength was determined one week after casting to 35 MPa and 37 MPa for the concrete used in Piles D2000 and D1500, respectively.

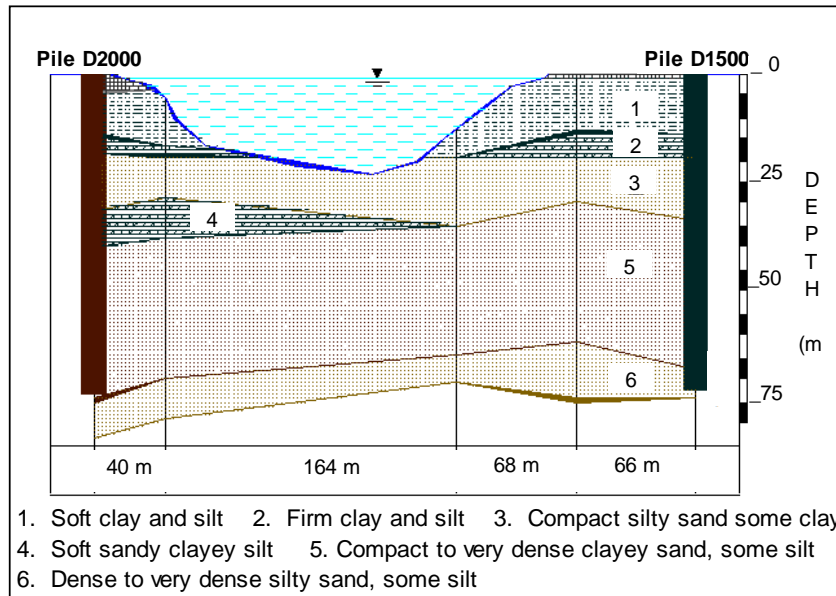


Fig. 2 Soil profile between the test piles

TABLE 1 Soil Parameters

Items		Layer	Layer	Layer	Layer	Layer	Layer
		1	2	3	4	5	6
Water content (w_n)	%	92	29	22	43		
Saturated density (ρ_{sat})	kg/m ³	1,470	1,500	2,050	1,800	2,100	2,100
Void Ratio (e_0)	- - -	2.48	0.80	0.60	1.16	0.54	0.54
Liquid Limit (LL)	%	96	54			-	
Plastic Limit (PL)	%	34	23			-	
Clay Fraction (%)	(- -)	42	47	13	28	21	6
Silt Fraction (%)	(- -)	55	46	20	45	12	12
Sand Fraction (%)	(- -)	3	7	67	27	67	82
Friction Angle (ϕ)	⁰	3°	13°	20°	13°	31°	23°
Vane Shear Strength (τ_{vb})	kPa	14	28		11		
SPT Index range	bl/0.3 m			11-30	7-15	18- >50	>50

The test piles were constructed by first inserting 2,032 mm and 1,520 mm outer diameter temporary casings, respectively, to 10 m depth. Thereafter, the D2000 and Pile D1500 shafts were drilled to 73.5 m depth using reverse circulation drilling and to 73.0 m depth using a bucket drill, respectively. The test piles were constructed by two separate contractors, each with a preference for the type of slurry to use; polymer slurry (Pile D2000) and bentonite slurry (Pile D1500). No records were kept on the actual procedures.

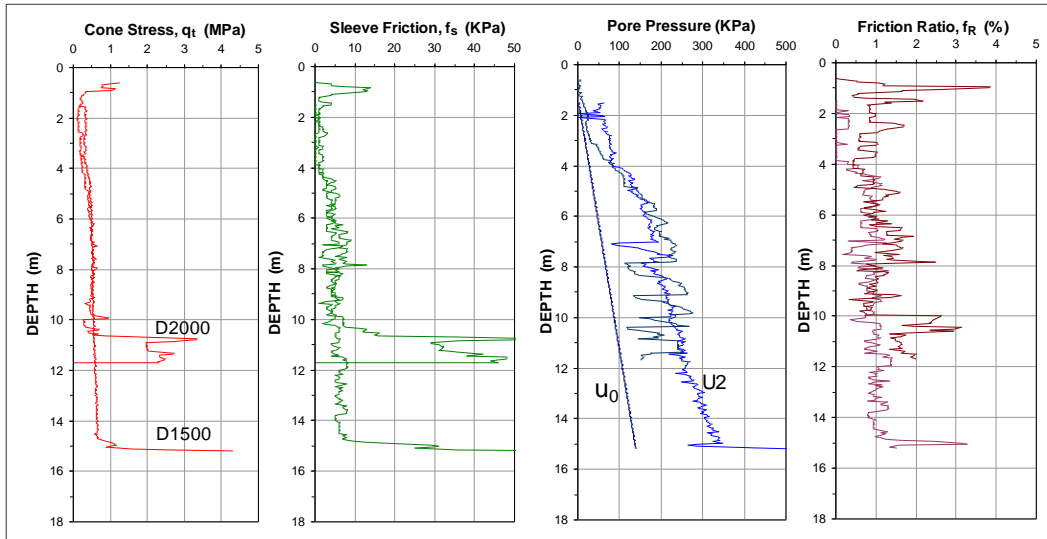


Fig. 3 CPTU diagrams from two soundings near Piles D2000 and D1500

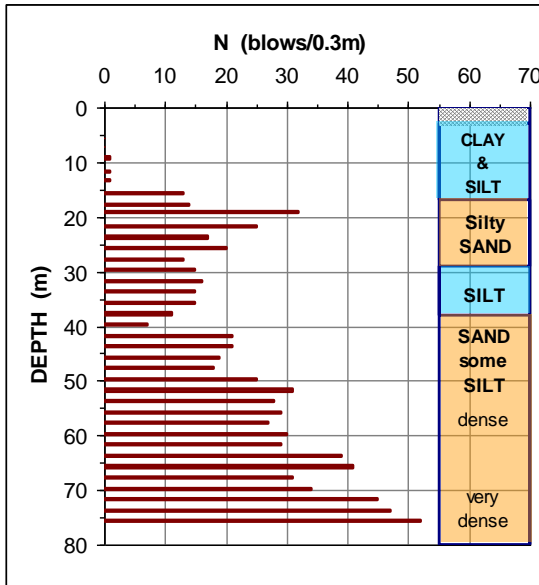


Fig. 4a SPT N-indices at Pile D2000

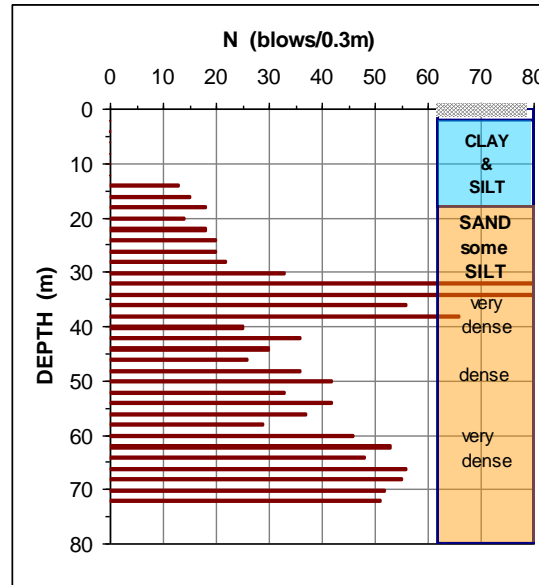


Fig. 4b SPT N-indices at Pile D1500

Before concreting each shaft, the shafts were cleaned and a reinforcing cage with the bidirectional-cell assembly attached 11.4 and 11.6 m above the cage end, respectively, was lowered into the shaft. Pile D2000 had a single 670 mm diameter cell and Pile D1500 had three 330 mm cells.

On May 25, 2008, 5 days after drilling, the D2000 drilled shaft a 300 mm O.D. tremie pipe was inserted to the bottom of the shaft to start tremie placing concrete, displacing the polymer slurry. However, when the tremie pipe had reached the pile shaft bottom, it became blocked by debris that had accumulated on the shaft bottom. The pipe was extracted and the debris was removed from the shaft bottom. Thereafter, tremie placing of the concrete was resumed.

The bottom 1 m length of Pile D2000 had been equipped with a 114 mm O.D. coring tube attached to the reinforcing cage. When the core was core drilled after the concrete had cured, debris and slurry was found to exist below the pile toe. As a remedy, on July 5, 2008, the pile toe was pressure-grouted through the cored hole with a 1:1 cement-sand slurry. Grouting pressure was not recorded and no strain-gage-measurements were taken.

On July 24, two days after the drilling, tremie placing concrete displacing the bentonite slurry in Pile D1500 was carried out, without mishaps. Also Pile D1500 had been constructed with a coring tube. However, the tube turned out to be damaged at the cell-assembly level and while the tremie placing of the concrete appeared to have been successful, coring of the pile and the soil below the pile toe was not possible. Therefore, pressure-grouting of the soil below the pile toe could not be carried out.

The pile and instrumentation details are shown in Table 2. Each test pile was instrumented with two pairs of diametrically opposed vibrating wire strain-gages at three levels below and five levels above the cell, as indicated in Figure 5. The static loading test on Pile D2000 was performed on July 15, 49 days after concreting Pile D1500 was tested on August 21, 28 days after concreting. The complete test reports are available in Loadtest (2008).

Table 2. Pile and Instrumentation details

Item	Pile D2000	Pile D1500
Length (m)	73.5	73.0
Diameter (mm) to 10 m	2,032	1,520
Diameter (mm) 10 m to toe	2,000	1,500
Length (m) above cell	62.1	61.4
Length (m) below cell	11.4	11.6
Buoyant weight above cell (KN)	2,900	1,660
Rebar number and size	44x Φ 32	32 Φ 32
Spiral size (75 m spacing)	Φ 10	Φ 10

The Bidirectional-Cell Test

The test consists of applying load increments to the pile by means of incrementally increasing the pressure in the jack, which causes the bidirectional-cell to expand, pushing up the pile shaft above the cell level and pushing down on the pile shaft below the -cell. The measurements recorded are the cell pressure (producing the load), the expansion of the cell, the shortening of the pile shaft above and below the cell level, and the movement of the pile head, which combine to provide the upward and downward movements of the top and bottom cell plates.

Loading Procedure

The method of testing was in accordance with the “quick test” of ASTM Guidelines D1143-07. Pile D2000 was loaded in 23 equal increments of 790 KN to a maximum bidirectional-cell load of 18.14 MN and then unloaded in six decrements. Pile D1500 was loaded in 20 equal increments of 465 KN to a bidirectional-Cell load of 9.31 MN and then unloaded in five decrements. Each successive load level was held constant for ten minutes by manually adjusting the cell pressure as needed. Unloading was with 5-minute load-holding.

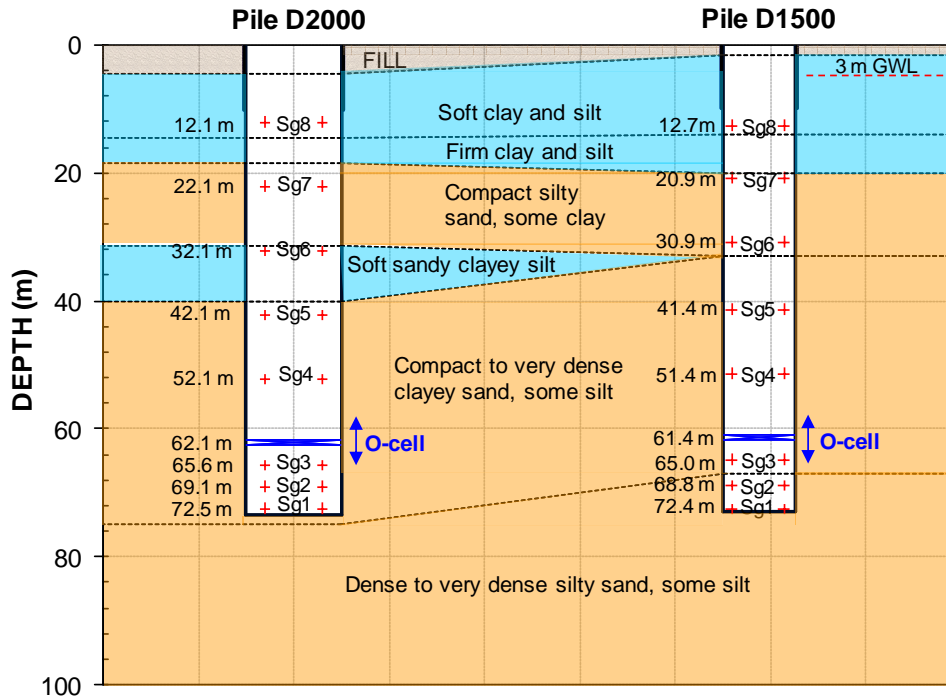


Fig. 5 Details of Instrumented Test Piles

TEST RESULTS

Load-Movement

The loads applied by the bidirectional-cell act in two opposing directions, mobilizing resistance above and below the cell. Figure 6 shows the load-movement records of the two cell tests. At the maximum cell loads (18.14 and 9.31 MN), the cell downward movements were 77.8 and 123.0 mm, respectively. Theoretically, the cell does not impose an upward movement until its expansion force exceeds the buoyant weight of the pile above the cell plus the residual load, if any, acting at the cell level. The pile buoyant weights above the cell for Piles D2000 and D1500 were 2.91 MN and 1.66 MN, respectively. Thus, the maximum upward resistances were 15.23 and 7.64 MN, respectively, generating upward movements of 5.3 mm and 8.5 mm, respectively. The maximum pile head movements (upward) were 0.5 and 2.3 mm, respectively.

The measurements of Pile D2000 do not indicate presence of any significant residual load at the cell level. However, for Pile D1500, a small amount of residual load, about 1 MN, could be present at the Pile D1500 cell level, as indicated by the fact that significant upward movement did not start until the applied load was about 1 MN larger than the buoyant weight of the pile above the cell.

Movement and Strain Measurements and Determining Axial Modulus

The upward shaft movements measured in the tests were small. At Gage Levels GL4 and GL5, the maximum relative movements between the pile shaft and the soil were no more than about 2 to 3 mm in both piles. This movement is not likely sufficient to have mobilized the shaft resistance fully. In contrast, the downward movements of 77 mm and 120 mm are larger than the minimum movement necessary to fully mobilize the shaft resistance along the pile portion below the cell levels. The full shaft resistance mobilization is reached where the slope (stiffness) of the load-strain curves (Figures 7a and 7b) become approximately linear.

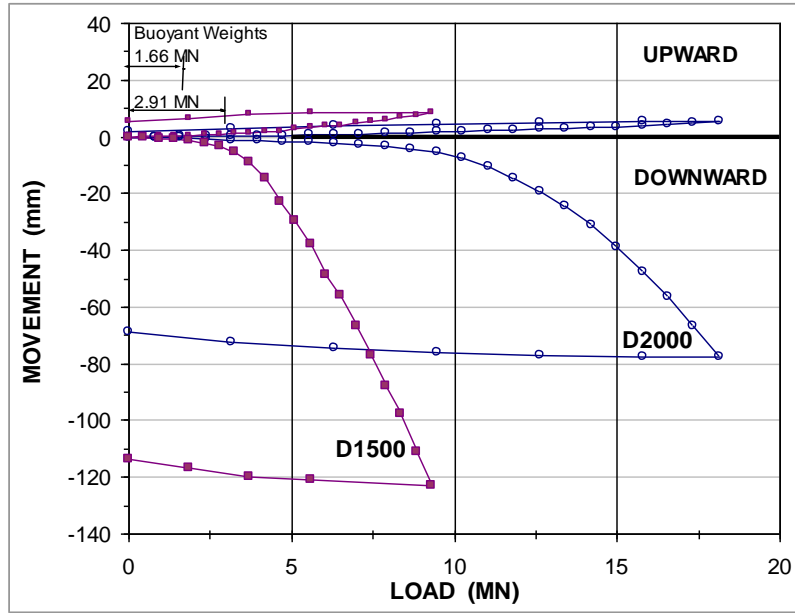


Fig. 6 Load-Movement Curves of Piles D1500 and D2000

Figures 7a and 7b report the measured load-strain curves. The values from Pile D2000 show that the slopes for the lowest, GL1, and highest, GL3, positioned gage levels below the cell level were approximately similar, whereas the GL2 curve deviated significantly from the other two. The deviation is considered due to the difficulties with the initial tremie and cleaning of the pile below the cell level. It is likely that the pile is bulging at GL2 resulting in a larger cross section and, therefore, a seemingly larger stiffness. Gage GL2 data have not been used in the further analysis.

Furthermore, both load-strain relations from Pile D2000 GL1 and GL3 plot along two approximately straight sloping lines. For each, one slope, called “early”, was about 80 GN, appearing from the applied loads of about 10 and 8 MN, respectively, when the downward movements were 7 and 4 mm, respectively. The slopes then changed to a “late” slope of about 60 GN, appearing from the applied loads of about 13 and 10 MN, respectively, when the downward movements were about 30 and 7 mm, respectively.

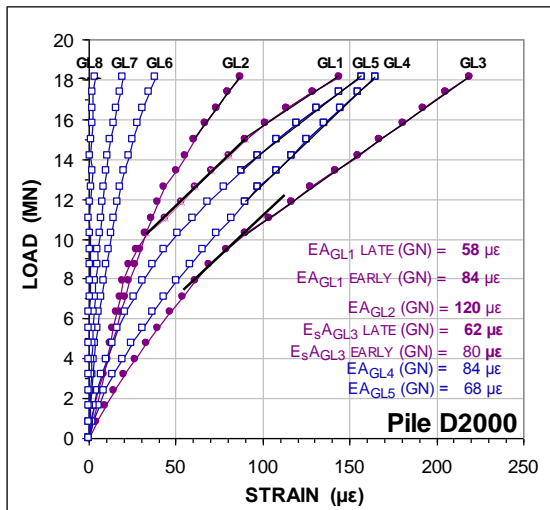


Fig. 7a Load versus strain, Pile D2000

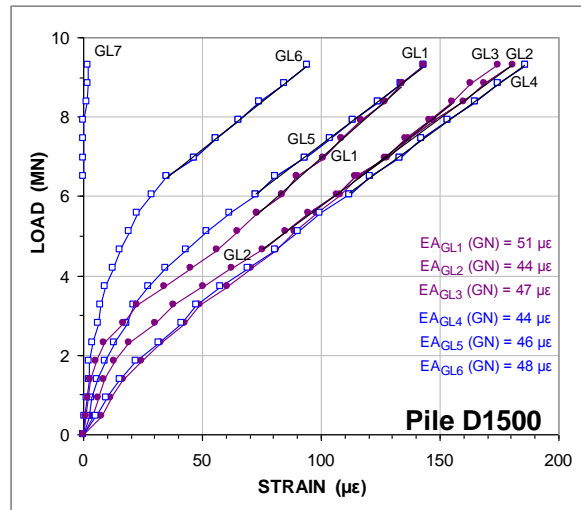


Fig. 7b Load versus strain, Pile D1500

The change of slope is considered due to the fact that the shaft shear response went into a strain-softening mode above the particular gage level, the load increment reaching the gage level became larger than the applied load increment, resulting in a larger strain change due to the softened shaft resistance.

The load-strain relations from Pile D1500 (Figure 7b) do not show a similar two-slope load-strain curves.

In order to convert the strain values to load, it is necessary to know the modulus of the concrete. Determining the modulus to use is simple in theory, but complex in practice. The modulus of concrete differs between different batches and it, therefore, needs to be determined for the actual batches used in the pile. Although there is a relation between concrete strength—cylinder or cube—and modulus, the relation is not sufficiently rigorous to provide accurate strain to load relations for the test data. The relation is best determined from studying the actual load-strain relations of the test. As indicated by Fellenius (1989; 2014), the load at a gage level is a function of the secant modulus or stiffness, E_s or AE_s , for the combination of the concrete and the steel reinforcement in the pile. The secant modulus of concrete is not a constant parameter, but reduces with increasing strain over the large strain range usually imposed in a static loading test. However, for the rather low strain range imposed in the subject tests—the maximum strains measured in the Piles D2000 and D1500 were 180 and 240 $\mu\epsilon$ —the modulus can be assumed to be essentially constant. Thus, a satisfactory accuracy can be obtained from the slope of the curves in a diagram showing the applied load and measured strains, as indicated in Figures 7a and 7b for Piles D2000 and D1500, respectively. Generally, once the shaft resistance acting along the pile between the load application (the bidirectional-cell in the subject tests) and the strain gage location has been mobilized, the load-strain curve will become a practically straight line. That is, provided that the shaft resistance is reasonably plastic and neither strain-hardening nor strain-softening after reaching its peak strength. The slope of the line is the pile axial stiffness as a function of the induced strain.

Usually, the best way of determining the pile modulus is by means of a so-called "tangent modulus" or "incremental stiffness" plot, that is, the applied increment of load over the induced increment of strain plotted versus the measured strain. This is shown in Figures 8a and 8b for Pile D2000 and D1500, respectively. The tangent modulus determined from the incremental stiffness procedure establishes the secant modulus for a modulus relation that reduces with strain. It is a differentiation process and, therefore, it is very sensitive to small variations of accuracy of both load and strain measurements. Moreover, it requires that the shaft response is reasonably plastic.

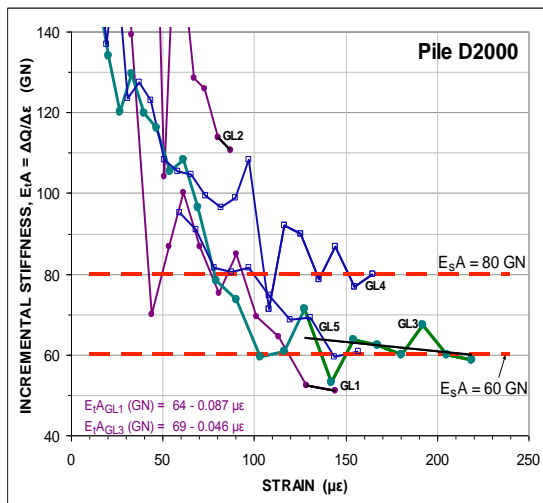


Fig. 8a Pile D2000 Stiffness Plot

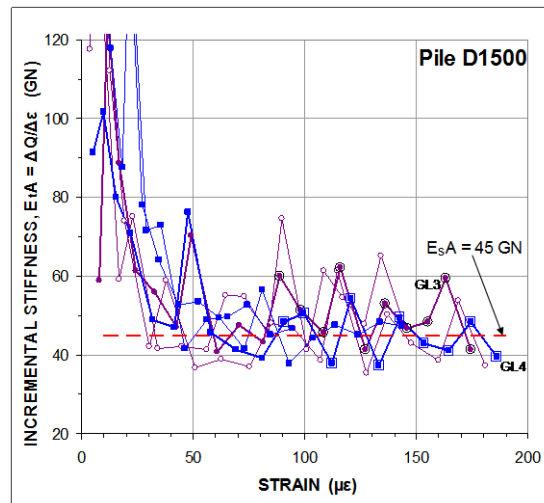


Fig. 8b Pile D1500 Stiffness Plot

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 a slightly sloping line. The line defines the tangent modulus relation for the pile cross section. The tangent modulus can then be converted to secant modulus as indicated by Fellenius (1989; 2014). Because of the combined effect of the strain-softening and the scatter of values, the incremental stiffness method did not provide reliable values for the tangent and secant stiffnesses. Therefore, for this case, the authors have preferred to rely on the linear portions of load-strain relations and convert the measured strains using constant stiffnesses, AE, of 80 GN and 45 GN for Piles D2000 and D1500, respectively. Correlated to the nominal cross sectional areas, both values indicate an E-modulus of 25 GPa.

Load Distributions

The evaluated stiffness (EA) values were used to convert the ten-minute load-holding strain values to load as shown in Figures 9a and 9b. The minimal shaft resistance shown along the upper about 20 m for both piles could be due to either or both facts that the gages are in the soft clay and silty clay layers or that the shaft resistances were not fully mobilized. However, with regard to the relative change of shaft resistance of the load distribution further down the piles, it is neither logical that the resistance between GL6 and GL5 in Pile D2000 would be much larger than the resistance between GL5 and the cell, nor that the resistance in D1500 between GL4 and the cell is so small. Actually, as shown in Figure 2, the soil at the depth of GL6 to GL5 at Pile D2000 was a soft silt, not sand. It is a reasonable expectation that the load distributions would have shown a gentle shape with a decreasing slope toward depth, reflecting increasing shaft resistance as the effective overburden stress and the sand density both increase with depth. That this is not the case is considered due to varying effect of the slurry filter-cake and, possibly varying pile cross section.

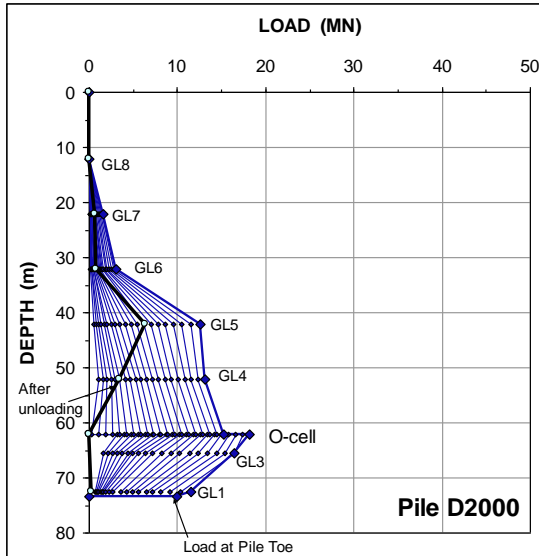


Fig. 9a Pile D2000 Load Distributions

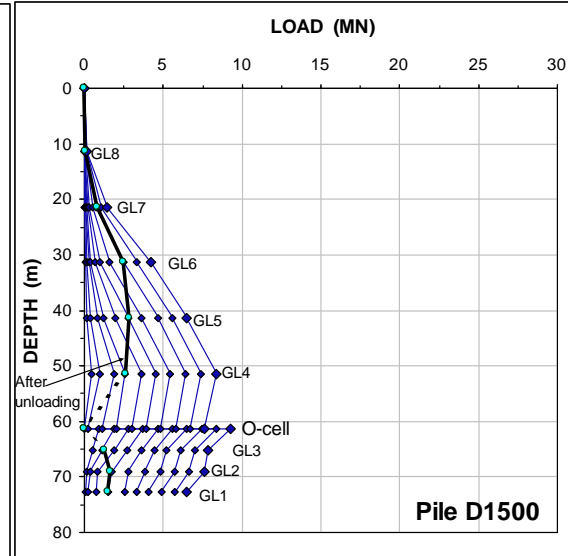


Fig. 9b Pile D1500 Load Distributions

Load and unit stress versus movement for shaft and toe resistances

Disregarding the strain-gage loads above the cell and assuming that the shaft resistance would show a uniform proportionality to effective overburden stress, Figures 10a and 10b show the actual and approximate equivalent head-down load-distribution curves. The latter are obtained by an effective stress calculation for β -coefficients of 0.14 and 0.12 for Piles D2000 and D1500, respectively, throughout the pile lengths.

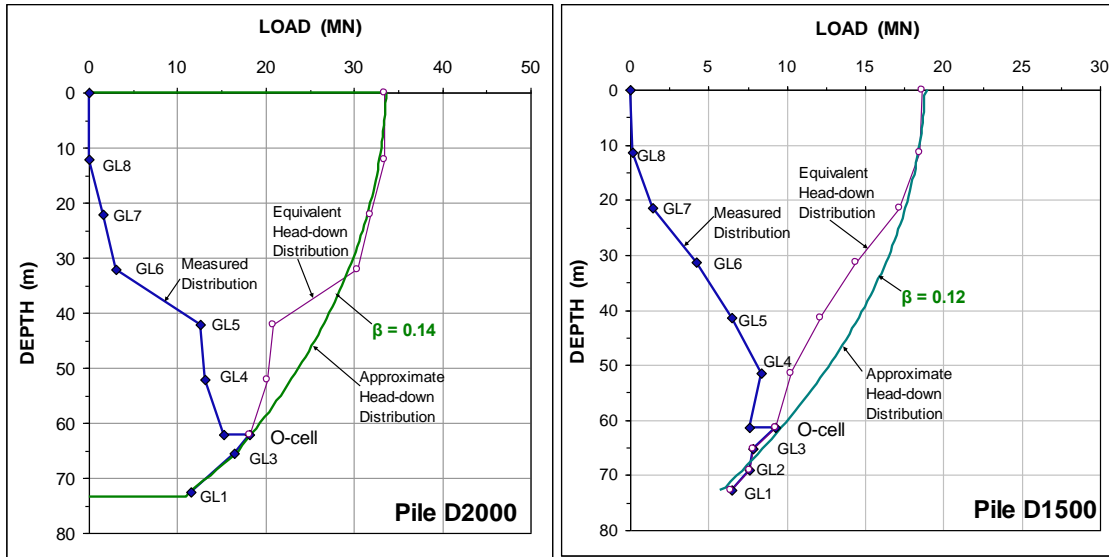


Fig. 10a Pile D2000 Load Distributions with Approximate Head-down Curve **Fig. 10b Pile D1500 Load Distributions with Approximate Head-down Curve**

The average unit shaft shear resistance between the gage levels can be determined as the difference in evaluated strain-gage load divided by the surface area between the gage levels. Figures 11a and 11b show those values plotted against movement for the particular gage levels. Thus, values between GL1 and GL2 are plotted against downward pile toe movement, values between GL2 and GL3 are plotted against the mean of movements of the pile toe and the downward cell movement, values between the cell level and GL3 are plotted against the downward movements. Similarly, values between the cell level and GL 4 are plotted against the upward movements, etc.

The records of unit shear resistance between GL5 and GL6 in Pile D2000 were considered unreliable and are therefore omitted. (The shear force vs. movement showed a straight-line trend; the maximum unit resistance was more than four times the undrained shear strength and would correspond to a beta-coefficient of about 0.6, which is not consistent with the soft silt found in-between GL5 and GL6).

The uppermost gage level movements and loads were too small to produce meaningful curves in either pile. The peak resistance was reached at very small movements, a few millimetres only. The tendency to post-peak softening is noticeable in several of the curves.

The lowest diagrams in Figures 11a and 11b show the unit toe resistances, as determined by dividing the measured load in GL1 with the respective nominal pile cross section and plotting the stress values against the measured toe movements. For comparison, the figure includes also the shaft resistance curves. The toe curves show no tendency toward an ultimate resistance, confirming that ultimate toe resistance is a concept not existing in reality (Fellenius 1999; 2014).

The unit shaft resistances estimated from the load distributions are small, as demonstrated by the small beta-coefficients (Figures 10a and 10b). However, the distributions shown are those for the maximum load, which was applied when the peak resistance had been surpassed. Judging by the unit resistance versus movement curves, the peak shear resistance would be more than twice as large as that shown for the maximum applied load. What the peak pile shaft capacity would be is not important for assessing the pile in a design, however. Design by applying a factor of safety, or resistance factor, to a capacity is a crude approach, and, therefore, what working load to accept from the test results must be determined using conservative values of shaft resistance. In the subject case, the post-peak values are used to estimate the shaft resistances and beta-coefficients.

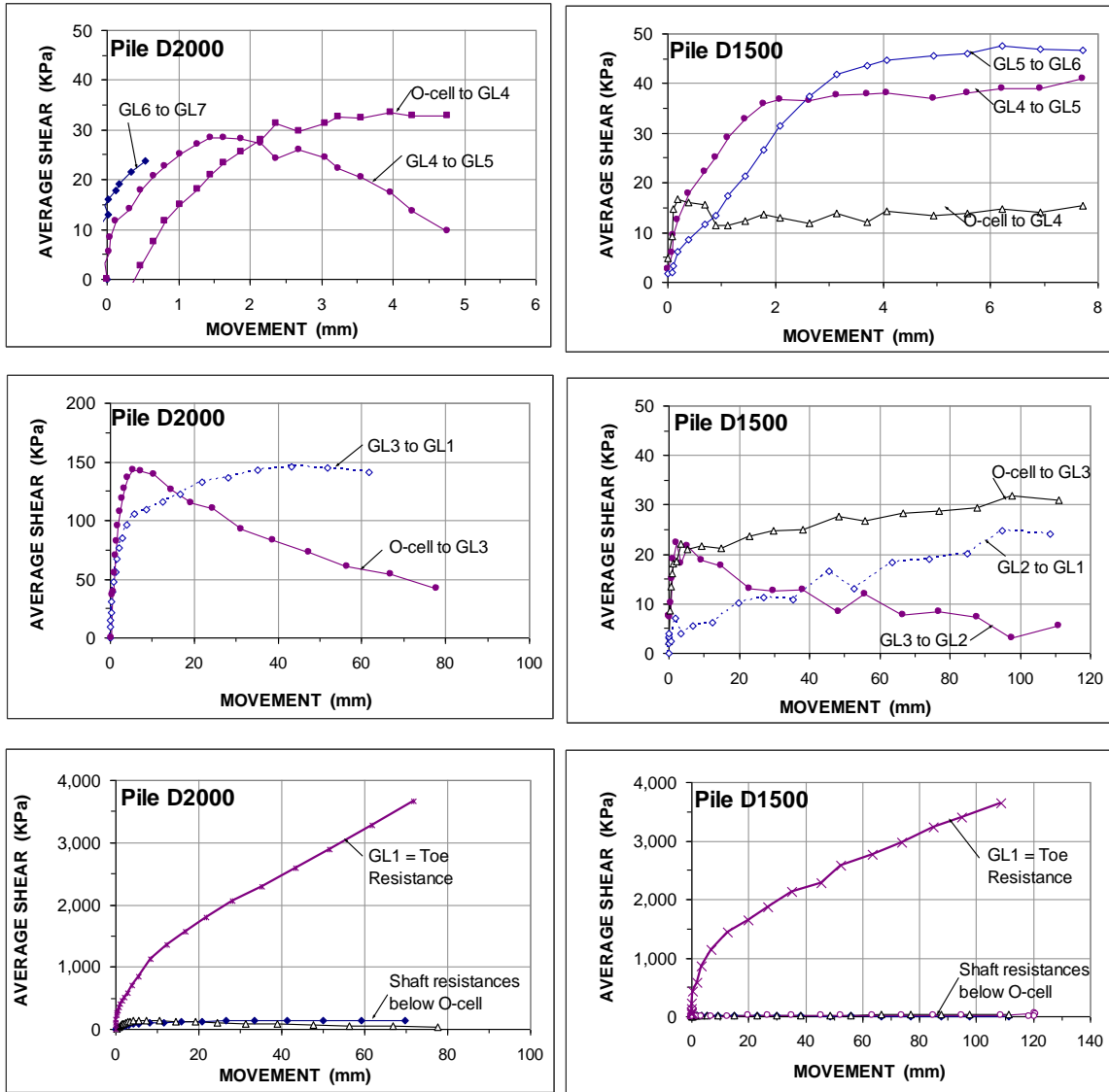


Fig. 11a Pile D2000 Unit Shaft and Toe Resistances

Fig. 11b Pile D1500 Unit Shaft and Toe Resistances

It is surprising that such distinct post-peak softening shaft resistance response would occur in a dense to very dense sand. As mentioned, it is therefore likely that the shaft resistance for both piles is affected by the presence of left-behind slurry filter cake between the pile and the soil. Furthermore, also the toe responses are very soft and not anywhere near what one would expect in a very dense sand. As mentioned, the toe responses are similar for the two test piles, which indicates that the pressure grouting at Pile D2000 failed to remedy the contamination of the pile toe.

Moreover, as the measured pile toe load-movement responses clearly show, the toe contribution to the pile capacity for use in a factor of safety or a LRFD approach must be defined as the toe resistance mobilized at some acceptable toe movement, often chosen to a values of about 10 to 15 mm—”a full half inch” and not some value of “pile toe capacity”.

The total shaft resistances at maximum cell load were about 25,000 and 13,000 kN, for the two piles, respectively. The pile toe resistances at about 10 mm toe movement were about 4,000 and 2,000 kN, respectively. It was decided that the tests had established pile capacities that satisfied the specified values.

About ten months earlier, at a bridge site (Phu My) about 10 km downstream and south of from the Binh Loi bridge, a 1,800 mm diameter pile had been tested using two bidirectional cell levels. The pile was drilled through an almost identical soil profile to a depth of 74.4 m using bentonite slurry. The only essential difference between the piles at the two sites was that the construction was executed by another contractor than the two who did the pile construction at the Binh Loi site. With the exception of one gage level, the strain-gage instrumentation in the Phu My test pile was not successful. However, the two-level cell test showed a unit toe resistance of about 8 MPa at a movement of about 70 mm, i.e., an about 2.5 to 3 times stiffer response than that shown for Pile D1500. Furthermore, the unit shaft resistance was at least about twice that of Pile D1500. The working load for the Phu My piles was 12 MN, twice that for the Binh Loi D1500 pile.

Moreover, about two years later, November 2010, an 1,500 mm diameter bidirectional-cell test was performed at the Sunrise City complex, a series of office towers. The contractor was different to the contractors for the Phu My and Binh Loi sites. The Sunrise City site was located about 8 km west of the Binh Loi site in a very similar soil profile. However, the site investigation characterized the soil at the pile toe level as a very dense sandy silt. The construction method employed bentonite slurry and bucket drilling and the pile length was 76 m. The results showed the same low toe stiffness as found for the Binh Loi D1500 test pile and the unit shaft resistance correlated the same narrow range of beta-coefficients as for the subject case. The pile working load was 8 MN.

The back-calculated pile stiffness, EA , for the test piles at the Phu My and Sunrise sites was about the same as for the Binh Loi piles suggesting that neither site had piles constructed with a diameter that deviated much from the nominal diameter. That is, applying the nominal diameter when calculating the unit shaft resistance was reasonable for the analysis of the strain-gage data.

Modeling the long-term pile response

The results of a bidirectional-cell test, or head-down test for which the pile toe response has been determined, are eminently suitable for a more exacting analysis of the pile design: analysis based on settlement of the piled foundation, rather than pile capacity. The process for the subject piles and project is illustrated in Figures 12a and 12b. The process combines—unifies—design for capacity, drag force, settlement, and downdrag. It is, therefore, called “The Unified Pile Design” method (Fellenius 1984; 1988; 2014, Fellenius and Ochoa 2009). The main feature of the process is that the sustained working load, the shaft resistance distribution, and the soil settlement distribution are known entities for each case analyzed and they interact with the neutral plane location, the downdrag, and the pile toe load-movement response, which are interdependent variables. The calculations shown in Figures 12a and 12b are performed with the software UniPile5 (Goudreault and Fellenius 2013).

The Unified Method assumes that the long-term shaft resistance is fully mobilized; indeed, in the subject case, acting at the post-peak condition. As to pile toe resistance, it applies the load-movement curve for the pile toe (measured or according to an assumed q - z relation). The toe load is then determined by the correlation between the toe movement relative to the soil (toe penetration), that is, the difference for the soil settlement at the pile toe level and for the pile toe movement that generated the pile toe load.

The analysis results show that the piles will be having a drag force that is about equal to the sustained working load on the piles. However, the maximum axial loads will be well within the structural strength of the piles and is therefore of no concern.

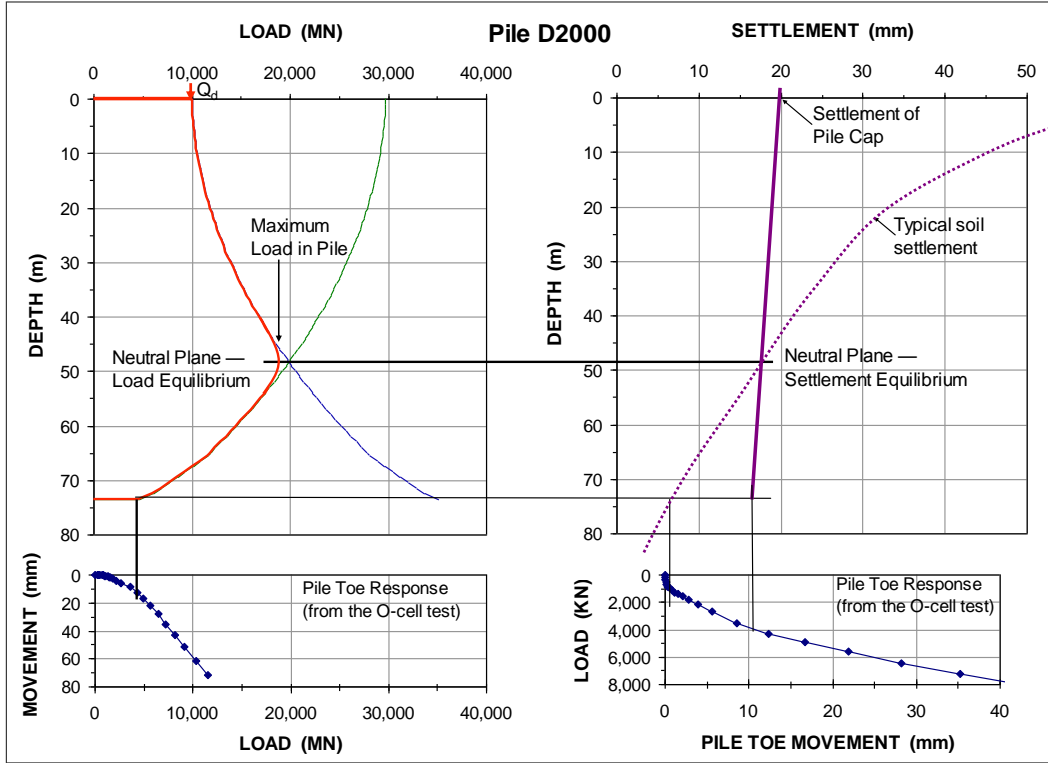


Fig. 12a Pile D2000 Load and Settlement Compilation for the Design

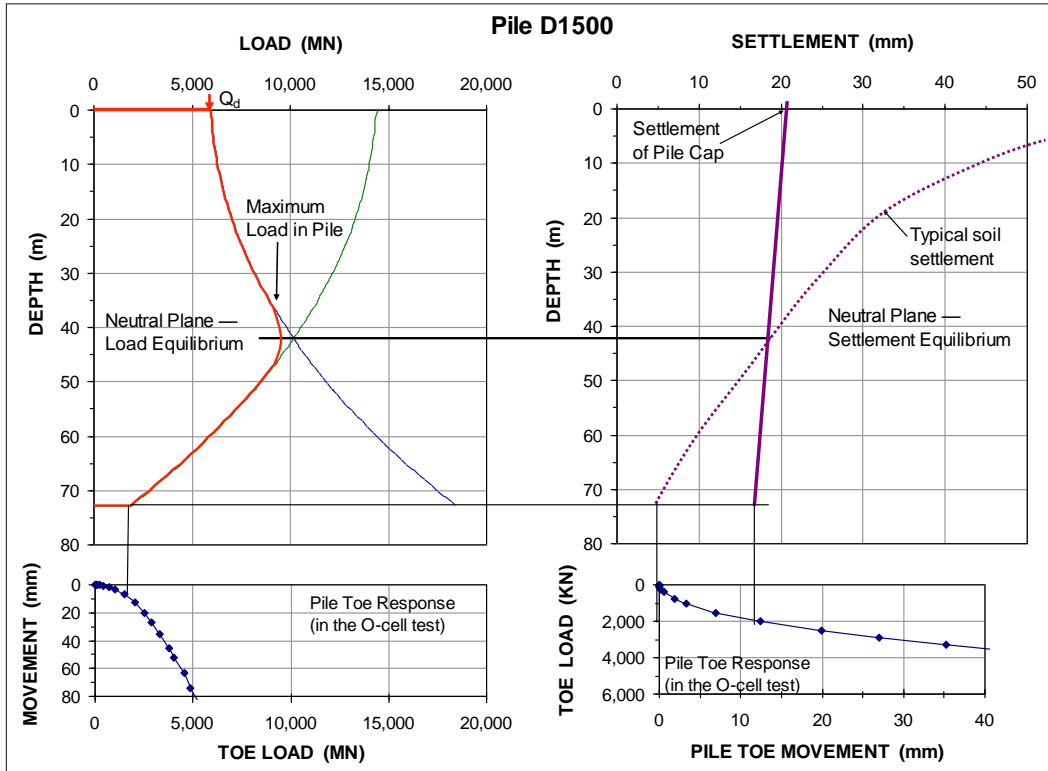


Fig. 12b Pile D1500 Load and Settlement Compilation for the Design

The analysis process is iterative. In a first attempt, an assumed pile toe load is chosen that will establish a force equilibrium—neutral plane—which is not likely to satisfy the settlement equilibrium, that is, show a location of the neutral plane which has a pile toe movement that agrees with that of the chosen pile toe load. However, after a couple of iterations, a load distribution will be found with a pile toe load that places the depth to the neutral plane that is the same magnitude as the pile toe load found for the settlement and toe movement diagram. The end result is the settlement at the pile head, which is the settlement of the structure supported by the pile(s). If that settlement is acceptable to the structure, the design requirements are reached.

Note, the load distribution shown rising from the pile toe load to a maximum value at the pile head is governed by the pile toe load, which in turn is determined by the soil settlement and pile toe movement. It is not the same load as that may have been chosen as the pile toe “capacity” used in a factor of safety or LRFD approach.

For the current piles, it is assumed that no residual load acts at the pile toe when the structure is built. In most cases, however, there would be some initial load present at the pile toe and the placement of the pile toe load-movement diagram in Figures 12a and 12b would have to be adjusted (moved to the left) accordingly. Such preloading condition would result in a reduced amount of toe movement, an increase of toe load, a lowering of the neutral plane, and a reduced pile cap settlement.

Unless extreme factors are applied, a design found to result in an acceptable settlement satisfactory by the Unified Method will also be shown to be safe in a factor of safety or LRFD. The reverse is not true, however.

SUMMARY AND CONCLUSIONS

The two bidirectional-cell tests on Piles D2000 and D1500 established that the pile capacities satisfied the specified values.

Analysis by the Unified Design method shows that if the structure can sustain long-term settlements of about 20 mm, the pile lengths and construction methods would be accepted.

The tests achieved maximum upward movements at the cell location of 5.3 mm and 8.5 mm, and maximum downward movements of 77.8 and 123.0 mm. The maximum upward movement of the pile heads were 0.5 and 2.3 mm, respectively, which means that the shaft resistance along the upper pile lengths were not quite fully mobilized.

The peak pile shaft resistance required only a few millimetres to develop and showed a substantial post-peak softening. The shaft resistance at large movement was small and correlated to an effective stress analysis beta-coefficient of only 0.14 and 0.12 for Piles D2000 with polymer slurry and D1500 with bentonite slurry, respectively. The difference is too small to be significant.

The measured pile toe stress-movements for the piles were almost identical, showing about the same stress and essentially linear trend. The pile toe response was very soft and not representative for a pile in very dense sand.

The use of different slurry composition—bentonite and polymer—for the two test piles showed no difference in distribution of shaft resistance or collection of debris at the bottom of the shaft. Brown (2012) has indicated a series of other differences and factors of importance for such comparisons. However, no pertinent observations were recorded that could have provided such factors.

It appears that the shaft and toe resistances for both test piles were affected by presence of slurry filter cake between the concrete and the soil, and this to a similar degree for the piles.

The toe stiffness and shaft resistance found for the Binh Loi and Sunrise City test piles were very similar to each other, but remarkably different from the test pile at the nearby Phu My site, constructed under soil conditions very similar to the Binh Loi soil profile, but by a different contractor. This indicates that cause of the low values of shaft resistance and toe stiffness at the Binh Loi and Sunrise City sites were due to less than desirable construction procedures.

The evaluated pile Young's modulus, E , for the two piles correlated to 25 GPa for the nominal pile cross section of both piles, as well as for the test piles at the Phu My and Sunrise City sites.

No measurements were made of shaft diameter and results of the cleaning effort. While the comparison between the three identical test piles (similar, but not identical diameters) at the subject and the two nearby projects, showed about the same pile axial stiffness, this does not mean that the actual diameter was the same as the nominal. Therefore, when applying the evaluated unit shaft resistance values to other piles in the area, it should be recognized that a mere two inches of diameter widening would result in a 10 % larger shaft circumference of a 1,500 mm diameter, and a proportional difference in the evaluated beta-coefficients.

The pressure grouting of the toe of Pile D2000 was not successful.'

ACKNOWLEDGEMENTS

The bidirectional-cell tests were performed by Loadtest Asia Pte. Ltd. under the direction of Mr. Khoo Han Sen. Thanks are owed to Dr. Mauricio Ochoa, PE, TWEI, Houston, TX, for his valuable review comments on the draft manuscript.

REFERENCES

- Brown, D., 2012. Recent advances to the selection and use of drilled foundations. ASCE GeoInstitute Geo-Congress Oakland, March 25-29, 2012, Keynote Lectures, State of the Art and Practice in Geotechnical Engineering, ASCE, Reston, VA, Rollins, K. and Zekkkos, D., eds., Geotechnical Special Publication, GSP 226, pp. 519-548.
- Fellenius, B.H., 1984. Negative skin friction and settlement of piles. Proceedings of the Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18 p.
- Fellenius, B.H., 1988. Unified design of piles and pile groups. Transportation Research Board, Washington, TRB Record 1169, pp. 75-82.
- 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., 2014. Basics of foundation design, a text book. Revised Electronic Edition, [www.Fellenius.net], 410 p.
- Fellenius, B.H. and Ochoa, M., 2009. Testing and design of a piled foundation project. A case history. J. of the S-E Asian Geot. Soc., Bangkok, 40(3) 129-137.
- Goudreault, P.A. and Fellenius, B.H., 2013. UniPile Version 5, Users and Examples Manual. UniSoft Geotechnical Solutions Ltd. [www.UniSoftLtd.com]. 99 p.
- Loadtest International Pte. Ltd., 2008. Reports on Bored Pile Testing, Binh Loi Than Son Nhat, Vietnam, LTI-2620-1 and LTI-2620-2, 68 p and 104 p.
- Osterberg, J.O., 1998. The Osterberg load test method for drilled shaft and driven piles. The first ten years. 7th Int. Conf. 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.