Non-Conventional Pile Loading Tests in Vietnam

Essais non conventionnels de chargement de pieux au Vietnam

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ABSTRACT: Two bidirectional tests used single-level jacks were performed on strain-gauge instrumented bored piles in Da Nang City, Vietnam. The soil profile consists of medium dense silty sand followed by thick firm clay underlain by highly weathered sandstone. The piles, 800 mm and 1,000 mm in diameter, were installed to 34 m depth and constructed using bucket drill technique with bentonite slurry. The jack assemblies were attached to a reinforcing cage above the pile toe from 0.5 m through 0.8 m. The static loading tests were performed 21 days after constructed piles. The maximum bidirectional test loads ranged from 3.8 through 4.8 MN and the measured maximum upward and downward movements ranged from about 3 through 28 mm and 7 through 49 mm, respectively. The analysis of strain-gauge records showed that the Young's modulus values were about 25 and 22 GPa as calculated on the nominal cross section of the 800 and 1,000 mm diameter piles, respectively, the shaft resistances were strain-softening, and the pile toe stiffness was very soft and essentially linear. The measured load distribution corresponded to effective stress proportionality coefficients, ß, of about 0.2 through 0.3.

RÉSUMÉ : Deux essais bidirectionnels utilisés à un niveau sur les pieux instrumentés à jauges de contrainte ont été effectuées à Da Nang, Vietnam. Le profil géologique du site se compose d'une couche de sable limoneux moyennement dense au-dessus d'une couche d'argile ferme épaisse reposant sur les grès très altérés. Les pieux de 800 mm et 1000 mm de diamètre, ont été installés jusqu'à 34 m de profondeur en utilisant la technique de seau de forage avec les coulis de bentonite. Les ensembles de vérins étaient attachés à une cage d'armature de 0,5 m à 0,8 m au-dessus de la pointe des pieux. Les essais de chargement statique ont été effectués à 21ème jours après l'exécution des pieux. Les charges maximales des essais bidirectionnels varient de 3,8 à 4,8 MN et les déplacements maximaux mesurés vers le haut et vers le bas varient respectivement de 3 à 28 mm et de 7 mm à 49 mm. L'analyse des données enregistrées des jauges de contrainte a montré que le module de Young était d'environ 25 à 22 GPa, tel que calculé sur la section nominale des 800 à 1.000 mm pieux, respectivement, les résistances se ramollissaient et la rigidité de la pointe des pieux était très faible et essentiellement linéaire. Le coefficient de proportionnalité (β) entre la charge mesurée et la contrainte effective correspondante était de 0.2 à 0.3.

KEYWORDS: bidirectional test, bored piles, strain-gages, shaft and toe resistances, strain-softening, movements.

1 INTRODUCTION

The 12-story Sea Bank Building is built over a 7.35 m by 20.30 m area among existing high-rise Buildings in Da Nang City, Vietnam. The foundations were placed on 800 and 1,000 mm diameter bored piles constructed to 34 m depth designed for working loads of 3.8 and 4.8 MN, respectively..

To validate the capacity of the piles, a pile loading test programme was carried out by means of the bidirectional O-cell test (Osterberg 1989) as being the best suitable for the limited project area. Both test piles were equipped with vibrating wire strain gages.

The results of the tests are presented and correlated to the soil conditions of the site. The test data and back-analyses are considered to be of interest beyond the design of the piled foundations for the Sea Bank Building.

2 SOIL PROFILE

The soil profile consists of medium dense silty sand to 18.5 m depth followed by 14.5 m thick firm clay underlain by highly weathered sandstone. Figure 1 shows the distribution of water content, consistency limits, grain size distribution, and SPT N-indices. The natural water content ranges from about 20 % through about 30 %. The density of the silty sand above the firm clay is 1,940 kg/m^3 (from w_n = 25 %). Total saturated density is about 1,950 kg/m^3 throughout the firm clay.

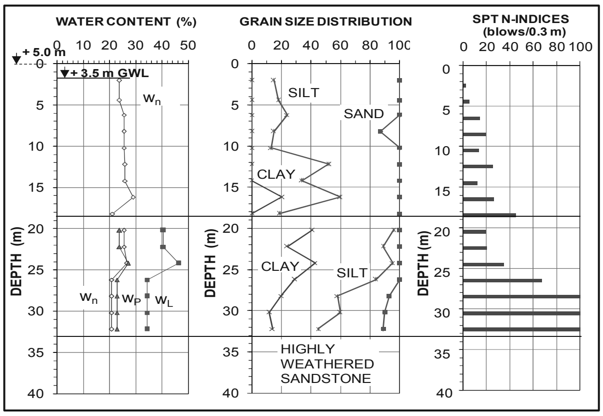

Figure 1. Water contents, grain size distrib., and SPT N-indices

Average of SPT N-indices is about 23 blows/0.3m to 27 m depth and more than 100 blows/0.3m below this depth. The underlain layer is highly weathered sandstones with rock quality designation of 0% through 10% and total core recovery of 12 through 21 % to 50 m depth below the original ground surface. Below this depth, the rock quality designation and total core recovery are 13 through 30% and 20 through 80%, respectively. The groundwater table is located at a depth of about 1.5 m below the ground surface.

3 PILE CONSTRUCTION AND TEST PROGRAMME

The two test piles, Piles TP01 and TP02 of 800 and 1,000 mm diameter, were constructed using bucket drill technique with bentonite slurry on 24 and 26 August 2012, to 34 m depth, respectively. The production piles were designed with the reinforcing cages of sixteen 20 and 22 mm bars to 11 m depth and eight 20 and 22 mm bars below this depth. The O-cells attached at 0.5 and 0.8 m above the pile toes, as shown in Figure 2. However, to avoid damage to the instrumentation during the lowering the reinforcing cages into piles the reinforcing cages of the test piles were supplied with sixteen bars.

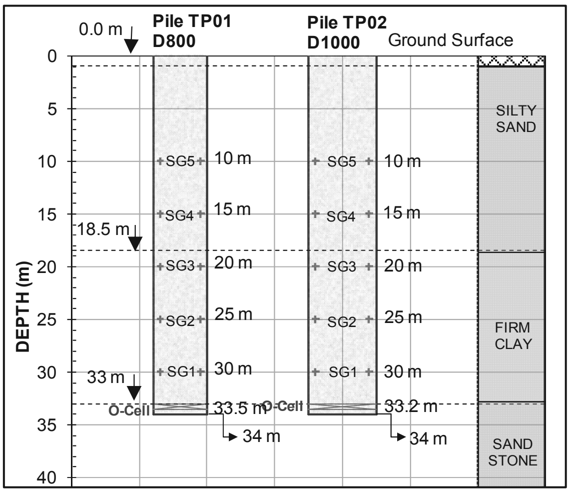

Figure 2. Details of instrumented test piles

The test piles were instrumented with a pair of diametrically opposed vibrating wire strain-gages at four levels and two pairs of diametrically opposed vibrating wire strain-gages at levels SG1 and SG4, respectively.

The test piles were constructed by first inserting 820 mm and 1,020 mm outer diameter temporary casings, respectively, to 7.6 m depth. Thereafter, the test piles were drilled to 34 m depth using a bucket drill with bentonite slurry. Before concreting each shaft, the shafts were cleaned and a reinforcing cage with the O-Cell assembly was lowered into the shafts. The O-cell assemblies consisted of three O-cells; 200 mm diameter in Pile TP01 and 220 mm in TP02.

On completion of the drilling, on August 24 and 26, 2012, a 219 mm diameter tremie pipe was inserted to the bottom of each shaft and tremie placing of the concrete was commenced, displacing the bentonite slurry.

The concrete cube strength was determined 28 days after casting to 40 MPa and 44 MPa for the concrete used in Piles TP01 and TP02, respectively.

The bottom 0.5 and 0.8 m length of Piles TP01 and TP02, respectively, was equipped with a 114 mm diameter coring tube attached to the reinforcing cage. The coring was performed after the concrete had cured and showed that an about 20 mm thick layer of debris and slurry existed below the toe of Pile TP01. Pile TP02 could not be cored because the coring tube was obstructed by steel reinforcement bars..

The static loading tests were performed on September 16 and 17, 2012, 23 and 22 days after concreting.

4 TEST RESULTS AND ANALYSIS

1.1 *Load – movement measurements*

The internal bond of the O-cells was broken at loads of at 190 and 480 KN load for Piles TP01 and TP02, respectively. Theoretically, the O-Cell does not impose an additional upward load until its expansion force exceeds the bond breaking load, the buoyant weight of the pile above the O-Cell and the residual load, if any, acting at the O-cell level. For both test piles TP01 and TP02, the initial upward movement records were taken at 570 and 960 KN; that is, these recorded loads included bond breaking load, the buoyant weight and residual load. The pile buoyant weights above the O-Cell for Piles TP01 and TP02 were 163 KN and 255 KN, respectively. Therefore, the residual loads determined at O-cell locations of Piles TP01 and TP02 are 217 and 225 KN, respectively.

Figure 3 shows the load-movement records of the two O-cell tests. After subtracting the buoyant weight of the piles TP01 and TP02 above the O-Cell, the maximum upward resistances were 3,636 and 4,544 KN, respectively. The maximum O-cell upward movements were 3.3 mm and 7.4 mm, the maximum O-cell downward movements were 28.0 and 49.3 mm, and the maximum pile head movements (upward) were 0.2 and 2.5 mm, respectively.

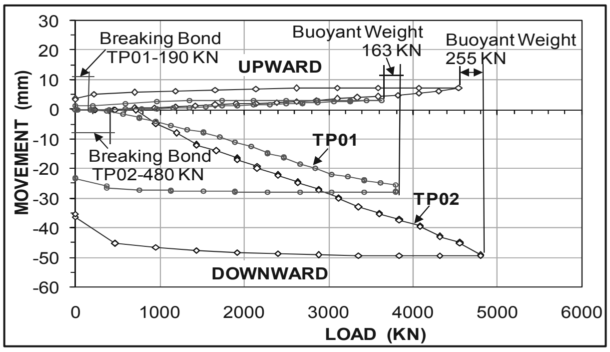

Figure 3. Load-movement curves of the piles

1.2 *Strain measurements and determining Axial Modulus*

To determine the secant modulus of the pile material, the best way is to use "tangent modulus" or "incremental stiffness" plot which is the applied increment of load over the induced increment of strain plotted versus the measured strain. The tangent modulus is then converted to the secant modulus (Fellenius 1989; 2011). The incremental stiffness plots for the two tests are shown in Figures 4 and 5. 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, representing the tangent modulus relation for the pile cross section. Because of the combined effect of the strain-softening and the scatter of values, the incremental stiffness method did not provide sufficiently precise 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 12.5 GN and 17.0 GN for Piles TP01 and TP02, respectively. Correlated to the nominal cross sectional areas, the values indicate that the E-modulus of Piles TP01 and TP02 is about 25 and 22 GPa, respectively.

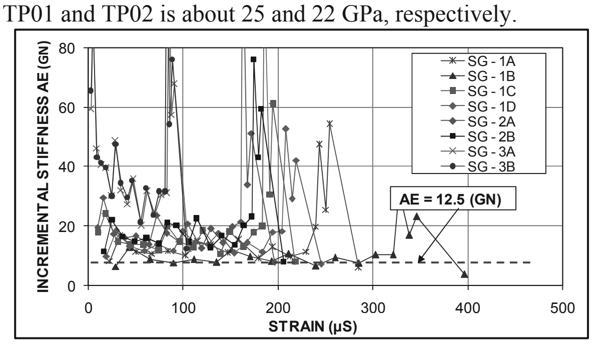

Figure 5. Increment stiffness plot of records from Levels 1 to 3, Pile TP01

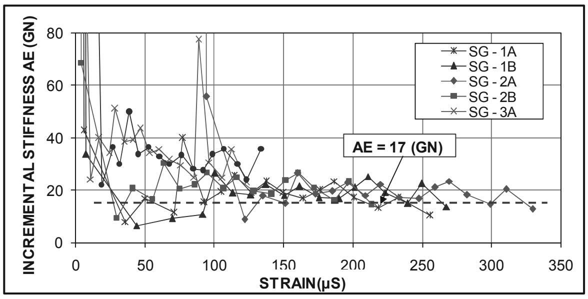

Figure 5. Increment stiffness plot of records from Levels 1 to 3, Pile TP02

1.3 *Load distribution along the pile shafts*

The evaluated stiffness (EA) values were used to convert the strain measurements to load. Figure 6 shows the so-evaluated load distributions. The shaft resistances shown along the upper about 30 m for both piles were not fully mobilized.

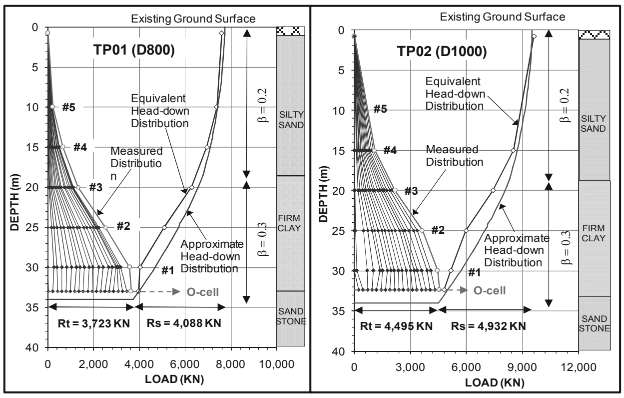

Figure 6. Load Distributions with Approximate Head-down Curve of Piles TP01 and TP02

As shown in Figure 1, the soil at the depth of SG1 to SG3 was a firm clay layer. 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 SPT N-indices 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.

Assuming that the shaft resistance would show a uniform proportionality to effective overburden stress, Figure 6 shows the actual and approximate equivalent head-down load-distribution curves. The latter are obtained by an effective stress calculation for ß-coefficient of 0.2 to 18.5 m depth and 0.3 below this depth to the pile toes. The effective stress analysis indicates the toe resistance and total shaft resistance of Pile TP01 are 3,723 and 4,088 KN, and of Pile TP02 are 4,495 KN and 4,932 KN, respectively.

1.4 *Unit stress versus movement for shaft resistances and pile toes*

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 8 and 9 show those values plotted against movements for the particular gage levels. The maximum values of the unit shaft resistance evaluated between the O-cell and SG1 for both test piles showed the average unit shaft resistances of about 60 kPa. Because of strain-softening, the shear resistances were smaller than expected. Moreover, the resistances from SG1 through SG3 are smaller than between SG2 and SG3 for both piles.. It is probable that the construction process left a filter cake between the concrete and the soil and, therefore, the shear movement occurred in the filter cake. The curves also show that, for both piles, ultimate shaft resistance along the length above the O-cell level has not occurred excepting for the curves from O-cell to SG1. The uppermost gage level movements and loads were too small to produce meaningful curves in pile. The records of unit shear resistance between SG4 and SG5 in Pile TP02 were unreliable and are therefore omitted.

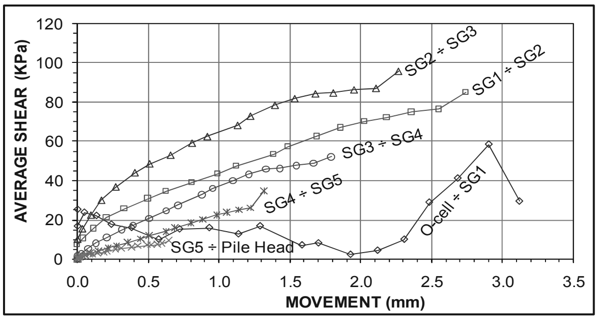

Figure 8. Average shear resistance between gage levels of Pile TP01

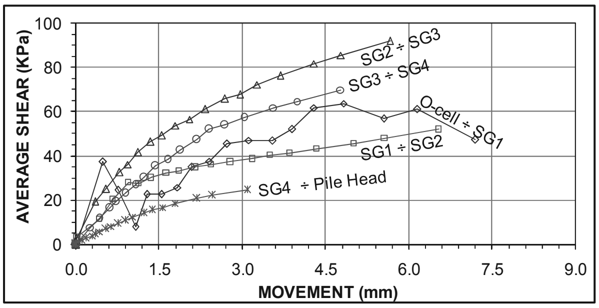

Figure 9. Average shear resistance between gage levels of Pile TP02

Figure 10 shows stresses applied to the pile toe versus the pile toe movements divided by the pile diameter in percent. The stresses are determined by dividing the measured load at O-cells with the nominal shaft areas below O-cell. The maximum toe movements were about 3 and 5 mm, respectively. The increasing stiffness measured for Pile TP01 is considered due to the debris at the pile toe having become compressed.

For both piles, the pile toe stress-movement response was essentially linear and showed no tendency toward an ultimate resistance. It is obvious that the pile toe response is not representative for pile toes placed in weathered sandstone or in sand. Similar observation for large diameter bored piles is reported by (Fellenius and Nguyen 2013).

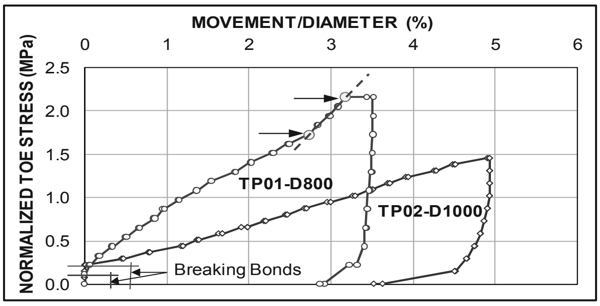

Figure 10. Stress-% downward movement curve of Piles TP01 and TP02

1.5 *Back-analysis to response long-term settlements of piles*

The back-analysis of two tested piles was performed by "The Unified Pile Design" method (Fellenius 1984; 1988; 2011).

The unified pile design approach is the combination of capacity, drag load, settlement, and downdrag. The main tenet of method includes the sustained working load, the shaft resistance distribution, and the soil settlement distribution; they interact with the neutral plane location, the downdrag, and the pile toe load-movement response.

The back-analysis of two tested piles is performed by the software UniPile (Goudreault and Fellenius 1999) and the results are presented in Figure 10. Figure 11 shows the long-term load distribution in the piles TP01 and TP02 starting from the 3.8 and 4.8 MN sustained load from the structure, respectively. Assuming that the drag loads accumulated from negative skin friction are fully mobilized, the maximum loads in the piles are about 5.8 and 7.2 MN at the neutral planes at 23 through 25 m depth below the ground surface. The maximum loads are well within the axial structural strength of both piles. The settlements at the neutral planes and the 'elastic' shortenings of the piles are about 11 through 13 mm and 9 through 11 mm, respectively.

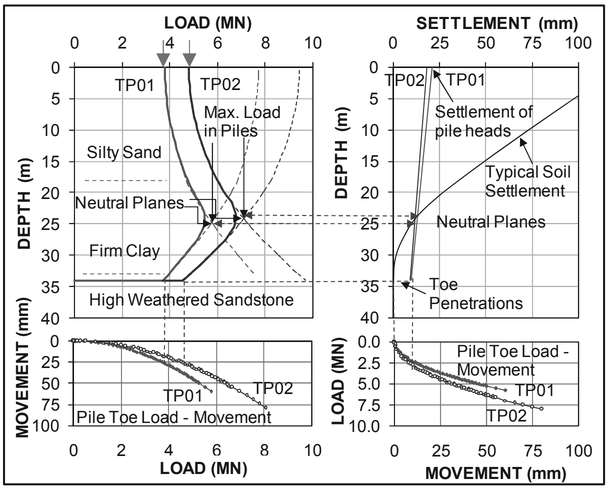

Figure 11. Long-term load distribution and settlement profile for working piles with same diameter and length as the two test piles.

As indicated in Figure 1, the soil at depths of the neutral planes is compressible; this can cause an increase of the pile penetration into the highly weathered sandstone, i.e., downdrag. However, this would partially be offset by the increase of the pile toe force with a subsequent lowering of the neutral plane.

For pile toe resistance, it is recognized that the pile toe does not exhibit an ultimate resistance but is a function of the pile toe stiffness response. Figure 11 also shows the pile toe load-movement curves established by the O-cell measurements. When drag loads in the piles TP01 and TP02 reach the maximum values, the pile toe resistances will increase to about 2.2 and 2.7 MN. The estimated final pile toe movements are about 10 and 9 mm, respectively.

The acceptable maximum long-term foundation settlement for the project is about 40 mm, which is larger than the estimated value. Thus, the testing and the design analysis indicate that there is no need for having the piles constructed deeper to bear on or in the bedrock.

5 SUMMARY AND CONCLUSION

The two bidirectional loading tests on Piles TP01 and TP02 established that the pile capacities satisfied the specified values.

The maximum load tests of 3.8 and 4.8 MN achieved maximum upward movements at the O-cell location of 3.3 mm and 7.4 mm and maximum downward movements of 28.0 and 49.3 mm, respectively. The maximum upward movements of the pile heads were 0.2 and 2.5 mm, respectively. The shaft resistances above 30 m depth were not full mobilized.

The evaluated pile Young's modulus, E, for the piles TP01 and TP02 correlated to 25 and 22 GPa for the nominal pile cross section of both piles. The shaft resistances were small and correlated to an effective stress analysis beta-coefficient of 0.2 and 0.3. The shaft resistances between O-cell and SG1 were strain-softening. It appears that the shaft resistances for both test piles were affected by presence of slurry filter cake between the concrete and the soil.

The measured pile toe stress-movements in percent of nominal diameters for the piles were essentially linear trend and have shown no tendency toward an ultimate resistance. The pile toe response was very soft and not representative for a pile in high weathered sandstone.

Back-analysis by the Unified Design method shows that the structure maximum long-term settlements of the piles will be about 20 mm, which is well below the acceptance value of 40 mm assigned to the project.

6 REFERENCES

Da Nang University of Technology, 2008. Reports on Bored Pile Testing for SeaBank Tower, Da Nang, Vietnam, 54p.

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., 2011. Basics of foundation design, a text book. Revised Electronic Edition, [www.Fellenius.net], 374 p.

Fellenius B.H. and Nguyen H.M., 2013. Large Diameter Long Bored Piles in the Mekong Delta. *International Journal of Geoengineering Case Histories,* 2(3), 196-207.

Goudreault, P.A. and Fellenius, B.H., 1999. UniPile Version 4.0 for Windows. Users Manual, UniSoft Ltd., Ottawa, [www.UnisoftLtd.com] 64 p.

Nguyen H.M., 2013. *Analysis of Bidirectional Test for Red River Bridge, Vietnam.* The 3th International Conference on Geotechnical Engineering, Hammamet, Tunisia, 9p.

Osterberg, J.O., 1989. *A new device for load testing driven piles and drilled shafts separates friction and end bearing.* Proc. of Deep Foundations Institute, Int. Conf. on Piling and Deep Foundations, London, May 15-18, pp. 421-427.