Analysis by the Menard Direct Design Method of O-cell Instrumented Pile Load Tests

Etude, par la méthode Ménard de conception directe des fondations, d'essais de pieux mettant en oeuvre des cellules Osterberg

N.M. Hai
Civil Engineering Faculty, Thu Dau Mot University, Vietnam

D.H. Dao
Bridge and Road Engineering Faculty, Da Nang University of Technology, Vietnam

N.T. Tien
Vietnam Society of Soil Mechanics and Geotechnical Engineering, Vietnam

ABSTRACT : The single-level O-cell tests were performed on two instrumented bored piles constructed at Sunrise City project, Ho Chi Minh City, Vietnam. The soil profile consists of very soft organic clay to about 20 m depth on firm to stiff clayey soil to about 30 m depth and underlain by a compact to dense sandy silt. Two test piles 1,500 and 1,800 mm in diameter were constructed using a bucket drill with bentonite slurry to 76 and 91 m depth, respectively. For both piles, the O-cell assembly was located above the pile toe about 15.4 and 19.8 m, respectively, the pairs of diametrically opposed vibrating wire strain-gages were installed at three levels below and eight through ten levels above the O-cell level. 30 days through 32 days after placing the concrete, the O-cell loading tests were carried out and failed to meet the required test loads due to soil debris left at the bottom of the drilled holes during construction. The maximum obtained O-cell test loads were 10 and 15 MN. Neither of the tests was able to fully engage the shaft resistance of the piles above the O-cell level, but did so below the O-cell level. This paper presents the results of O-cell tests and the analysis of the Menard Pressuremeter test data to respond to the not full mobilized shaft resistances.

RÉSUMÉ : Des essais au verin Osterberg ont été effectués sur deux pieux forés instrumentés construits au projet City Sunrise, Ho Chi Minh-Ville, Vietnam. Le profil de sol est composé d'argile organique très molle jusqu'à environ 20 m de profondeur au dessus d'une couche d'argile raide jusqu'à environ 30 m de profondeur et reposant par un limon sableux compact à dense. Deux pieux d'essaie en 1500 et 1800 mm de diamètre ont été construits à l'aide d'un " bucket" avec bentonite jusqu'à 76 et 91 m de profondeur, respectivement. Pour les deux pieux, l'ensemble de la cellule était située au-dessus de la pointe à environ 15,4 et 19,8 m, respectivement, les paires de jauges de contraintes opposées, des cordes vibrantes, ont été installées sur trois niveaux au-dessous et huit à dix niveaux au-dessus du niveau de la cellule. 30 à 32 jours après la mis en place du béton, les essais de chargement des cellules ont été réalisés mais n'ont pas réussi à répondre aux charges d'essai requis en raison de débris du sol résiduels au fond des trous forés pendant la construction. Le maximum obtenu pour les charges d'essai ont été de 10 et 15 MN. Aucun de ces tests a été en mesure de mobliser pleinement la résistance de l'arbre du fût des pieux au-dessus du niveau des cellules Osterberg, mais l'a fait en dessous de leur niveau. Cet article présente les résultats des essais dans les vérins et l'analyse des données d'essais pressiométriques Ménard pour répondre au manque de résistance du frottement mobilisé sur le fût des pieux.

KEYWORDS : pressuremeter test, limit pressure, pressure modulus, O-cell test, strain gage, load-movement curves.
MOTS CLES : Essai Pressiométrique, pression limite, module Ménard, essai Osterberg, jauge de contrainte, courbe effort- déformation.

1 INTRODUCTION

The static loading tests by O-cell method is widely used and regarded as a powerful tool to evaluate or to verify the pile capacity with high reliability, special for the large diameter and long bored piles to support foundation for mega constructions. However, one of the limitations of this method is only mobilizing fully one of the two shaft resistances above and below the single-level installed O-cells. Moreover, sometimes the shaft resistances of the tested piles are affected by presence of slurry filter cake between the concrete and the soil, therefore the capacity of the tested piles cannot reflect the actual soil condition. Therefore, the using of available data resources from soil investigation to evaluate additionally the shaft resistances not mobilized fully is necessary for each project.

This paper presents the analysis of the Menard Pressuremeter test data to respond to the not full mobilized shaft resistances of the single-level O-cell test failures on two instrumented bored piles at Sunrise City project (Lot W). The plan view of location of project, boreholes, pressuremeter test, and the two tested piles is shown in Figure 1.

The first author has presented a previous paper addressing test results, the applications of the O-cell method (Osterberg 1989; 1998), the quality and use of test results for the piled foundations design of the project (Fellenius and Nguyen 2013).

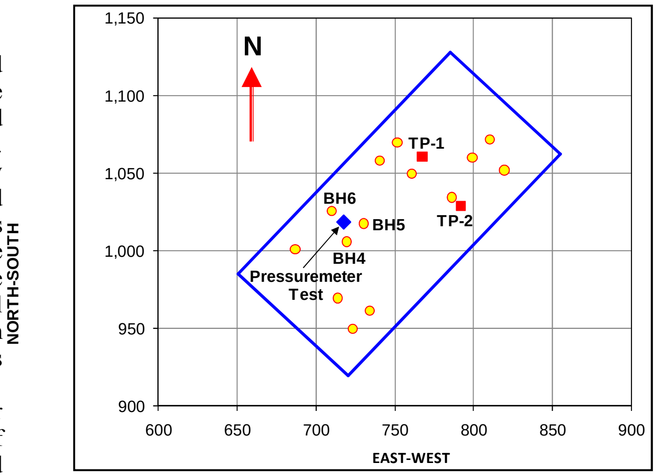

Figure 1. Plan view of location of boreholes, pressuremeter test, and the two tested piles

2 SOIL CONDITION

The soil profile at the site consists of very soft organic clay to 18 m depth followed to about 50 m depth by firm to stiff clay and clayey silt and underlain by compact to dense sand silt. Details are presented by (Fellenius and Nguyen 2013). The Saigon River level fluctuates seasonally by about 3 m. The groundwater table lies near the ground surface, becoming slightly artesian with depth.

Figure 2 shows the Menard pressuremeter data to about 52 m and SPT N-indices of the boreholes BH-4 through BH-6 to 80 m depth. As can be seen in Figure 2, the values of the limit pressure and pressure modulus vary from 0.29 through 0.32 MPa and 1.90 through 2.30 MPa to 18 m depth, 1.08 through 2.69 MPa and 20.03 through 29.04 MPa between 18 m and 50 m depth, and 2.55 through 2.37 MPa and 35.69 through 24.35 MPa below 50 m depth to 52 m depth, respectively.

The SPT N-indices displayed in Figure 2 is zero to 15 m depth and ranges between 0 and 13 at 15 through 20 m depth. The SPT N-indices varies from 5 to 27 below 20 m depth to 50 m depth and 14 to 32 below 50 m depth to 80 m depth.

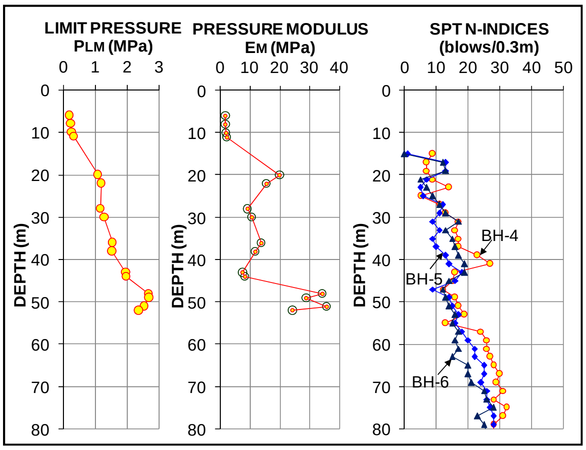

Figure 2. Mernard pressure modulus, limit pressure, undrained strength, and SPT N-indices

3 PILE CONSTRUCTION AND TEST PROGRAMME

A bored pile test progamme by single-level Osterberg Cell method to serve design of the piled foundation was performed at the Sunrise City project (Lot W). The two pre-production test piles, pile TBP-1 and TBP-2, were drilled on October 12, 2010, and October 07, 2010, to 76 m and 91 m depth, respectively, below ground surface.

The diameter of pile TBP-1 was 1,500 mm and the pile was supplied with a reinforcing cage of twenty 32 mm bars, resulting in a steel reinforcement area of 161 cm^2 and a reinforcement ratio of 0.9 % of the 1.77 m^2 total nominal pile cross section. The diameter of pile TBP-2 was 1,800 mm and the pile was supplied with a reinforcing cage of thirty 32 mm bars, resulting in a steel reinforcement area of 241 cm^2 and a reinforcement ratio of 0.95 %, of the 2.54 m^2 total nominal pile cross section. The concrete cylinder strength was determined 28 days after casting to 42.9 MPa and 44.0 MPa for the concrete used in piles TBP-1 and TBP-2, respectively.

The test piles were constructed by first inserting 1,830 mm and 1,540 mm outer diameter temporary casings, respectively, to about 8 m depth. Thereafter, the shaft of pile TBP-1 and TBP-2 was drilled to 76 m and 91 m depth using a bucket drill with bentonite slurry, respectively. Before placing concrete, the shafts were cleaned and a reinforcing cage with the O-Cell assembly attached 15.4 and 19.8 m above the cage end, respectively, was lowered into the shaft. O-cell level of Piles TBP-1 and TBP-2 had two 510 mm diameter cells and two 670 mm diameter cells, respectively. On October 12 and 07, 2010, one day and two days after drilling, the drilled shaft of piles TBP-1 and TBP-2, respectively, a 300 mm O.D. tremie pipe was inserted to the bottom of the shaft to start tremie placing concrete, displacing the bentonite slurry. The placed concrete quantity versus concrete displacements in the drilled shafts of Piles TBP-1 and TBP-2 as shown in Figure 3 shows.

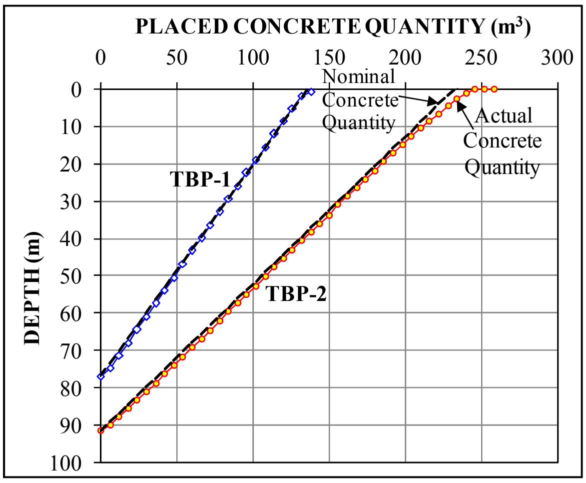

Figure 3. Concrete displacement curves of Pile TBP1 and TBP2

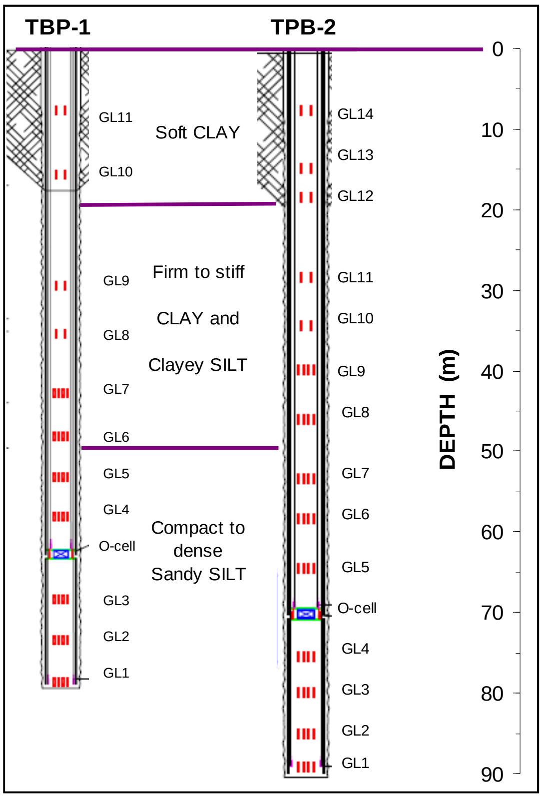

Figure 4. Schematics of the O-cell test piles

As can be seen in Figure 3, the concrete quantity computed basing on the nominal diameters is quite good agreement with the actual placed concrete quantity for both piles. This means that the actual tested pile shafts is the same as the nominal pile shafts excepting for the 10 m shaft of pile TBP-2 below ground surface lightly greater than the nominal shaft.

The test piles TBP1 and TBP-2 were instrumented with the pairs of diametrically opposed vibrating wire strain-gages at three through four levels below and eight through ten levels above the O-cell, as indicated in Figure 4. The static loading test on Pile TBP-1 was performed on November 11, 2010, 30 days after concreting. Pile TBP-2 was tested on November 08, 2010, 32 days after concreting. The complete test reports are available in Loadtest 2010.

4 MEASUREMENTS AND ANALYSIS

4.1 *Measured load-movement curves*

Figures 5 presents the load-movement measurements of the O-cell tests on Piles TBP-1 and TBP-2. The O-cell tests have failed to the required test loads of 32 MN and 50 MN, respectively.

The upward and downward maximum movements of Piles TBP-1 and TBP-2 at O-cell locations were 4.98 mm through 3.08 mm and 147.36 mm through 82.53 mm at maximum test loads of 10.25 MN and 15.23 MN, respectively. The maximum upward movement of the pile heads were 0.6 and 0.2 mm, respectively, which means that the shaft resistances along the upper pile lengths were only partially mobilized.

Figure 5 also demonstrates that the pile toe stiffness is very low and not display a capacity condition for the pile toes placed in compact to dense sandy silt. In addition, the shaft resistances below O-cell level have been mobilized fully at the small movement of about 4 mm.

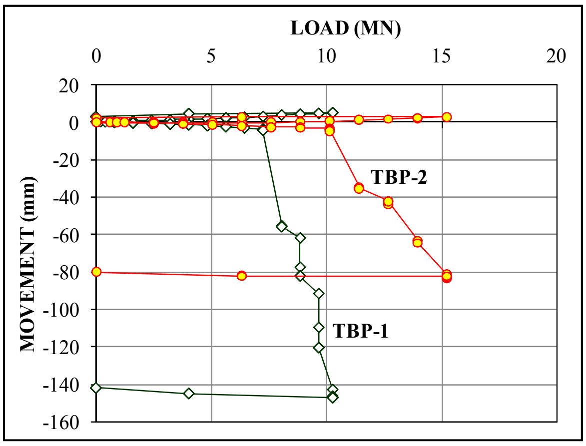

Figure 5. Load-movement curves of Pile TBP-1and TBP-2

4.2 *Unit shaft resistances versus movements*

The change of load between two gage levels denotes the amount of shaft resistance mobilized over the distance from one gage level to the next. Dividing that load difference with an area equal to the nominal circumference of the pile times the length between the two gage levels provides a value of the average unit shaft resistance between the gage levels. However, to determine the load values from the strain-gage measurements, knowledge of the pile "elastic" modulus is required. The E modulus of concrete determined is about 25 GPa on the nominal cross section areas and the unit shaft resistances versus the measured movements for both piles (Fellenius and Nguyen 2013). Therefore, this paper only presents the calculated unit shaft resistance values above O-cell level for each applied O-cell load and measured movements as shown in Figure 6 and 7. As reported, the gage records at GL 4, GL 5, and GL 8 for Pile TBP-1 and GL 6 for Pile TBP-2 showed the gages to malfunction and were therefore disregarded in the analysis.

Figures 6 and 7 indicate that the unit shaft resistances are displayed in two ways. The first, the unit shaft resistances are computed from the O-cell level to each gage level above O-cell and the second calculated for each pile segment between two adjacent gage levels. As mentioned earlier, the pile toe stiffness is very low and it is very easy to see that the TBP-1 pile toe stiffness is lower than the TBP-2 pile and thus the TBP-1 pile shaft resistances is less mobilized than the TBP-2 pile shaft resistances as shown in Figure 6a and Figure 7a, respectively.

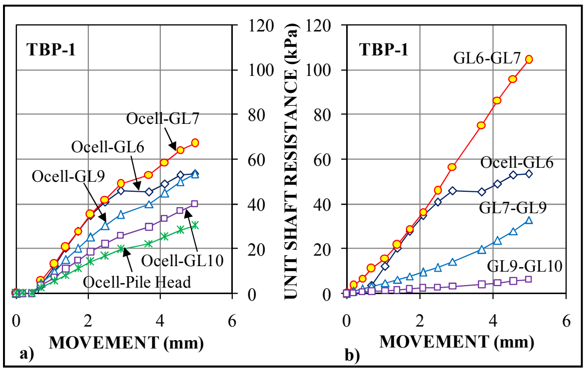

Figure 6. The shaft resistances above O-cell versus movements of Pile TBP-1

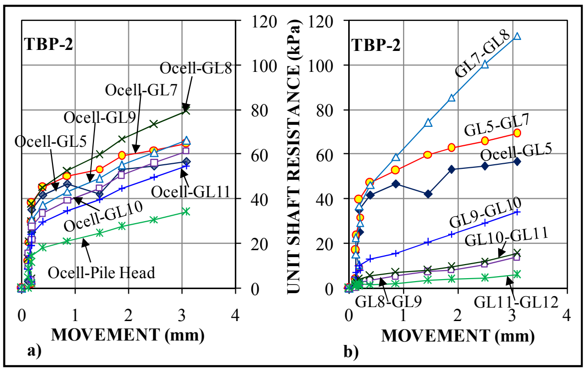

Figure 7. The shaft resistances above O-cell versus movements of Pile TBP-2

Can be seen in Figure 6b) and Figure 7b), the unit shaft resistance curves from O-cell to GL6 and to GL5 show the peak resistance at about 3 mm through 1 mm movement, respectively, and the tendency to post-peak softening is noticeable in these curves. In addition, the unit shaft resistances at these pile segments are significantly smaller than the pile segments from GL6 through GL7 and GL5 through GL8, respectively. It is surprising that such distinct post-peak softening shaft resistance response would occur in a dense to very dense sand. It is likely that these shaft resistances for both piles are affected by the presence of left-behind slurry filter cake between the pile and the soil. The uppermost gage level loads were too small to produce meaningful curves in both piles and therefore the following analysis of the Menard Pressuremeter test data to address to these not full mobilized shaft resistances.

4.3 *τ_{max} and τ-z curves from Menard pressuremeter test*

For the direct design methods of the Menard pressuremeter test developed for pile, they are assumed that the ultimate bearing capacity is related to the limit pressure, P_{lm}, and the settlement to the pressuremeter modulus, E_m. These relationships have been developed from theoretical studies of foundation behavior and observations of full-scale structures.

A correlation between the ultimate shaft resistance, τ_{max}, and pLM at the same depths can be written in the form of τ_{max} = f (pLM, soil type & pile/shaft type). These are correlations which rapidly started to receive confirmation based on hundreds of observations carried out by the French Government

Laboratories for Bridges and Roads (LPC's) on all types of prototype piles and drilled shafts using strain gauges along the whole pile shaft (Bustamante et al. 1981 - 2009).

The charts in Figure 8 show that the unit ultimate shaft resistances can be directly obtained basing on the type of installed piles and the values of limit pressures from Menard pressuremeter test. For the case subject, the ultimate shaft resistances will be found from curve Q_2 (Bustamante et al. 2009).

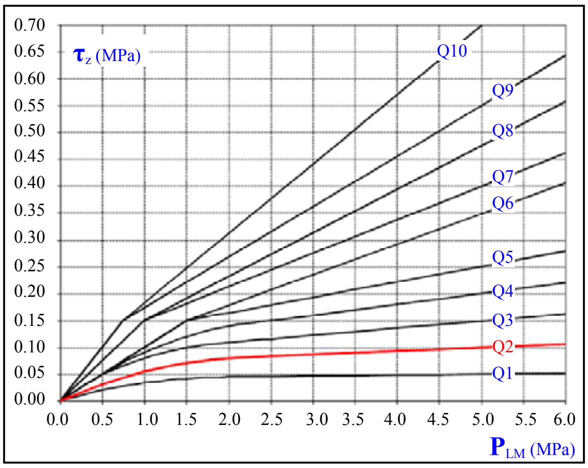

Figure 8. Direct Design using PMT Data. Chart for unit ultimate shaft resistance, τ_z (Bustamante et al. 2009).

Thus, from the limit pressures versus depth as shown in Figure 2, the average of the unit ultimate shaft resistances (τ_{max}) determined is about 20 kPa to 20 m depth, 60 kPa from 18 m through 25 m, 70 kPa between 25 m and 40 m, 75 kPa from 40 m to 45 m, and 80 kPa below 45 m to 52 m depth.

For the settlement calculation for deep foundation, the load transfer approach has been used. A semi-empirical method was developed from elasticity theory using Menard pressuremeter test results to fit experimental data for the prediction of the load-movement curves of pile. As shown in Figure 9, the t-z curve is modeled as bilinear elastic-plastic curve (Frank and Zhao 1982). This method is employed to determine the unit shaft resistance-movements curves of test piles.

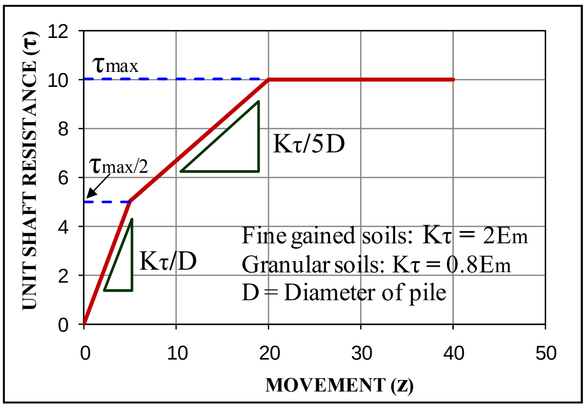

Figure 9. Tri-linear mobilization model of shaft resistances (Frank and Zhao 1982)

As shown in Figure 4, the gage levels of Pile TBP-1 and TBP-2 around 20 m through 52 m depth is GL6 to GL10 and GL7 to GL12, respectively. Figure 10 and 11 represent the unit measured shaft resistance-movement curves and the constructed-tri-linear curves.

For the τ_{max} of 80 kPa, the constructed-tri-linear curves in Figure 9 and 10 is correlative to the Ocell-GL6-and-GL6-GL7 curve of Pile TBP-1 and the GL7-GL8 curve of Pile TBP-2. These curves indicate that the tri-linear curves are stiffer in the initial line and softer in the second line. In addition, the GL6-GL7 curve provides an important evidence of the considerably underestimated capacity from the chart of (Bustamante et al. 2009) in Figure 8.

20 m through 40 m depth and 40 m through 45 m depth, the average of the τ_{max} is about 70 kPa and 75 kPa, respective. The tri-linear curves constructed from these shaft resistance values are compared with the measured shaft resistances-movements curves within the GL7-GL9-GL10 gage levels and the GL8-through-GL12 gage levels of Pile TBP-1 and TBP-2, respectively. Can be observed in Figure 11, the measured GL9-GL10 and GL11-GL12 curves have a good agreement with the initial lines of the 70 kPa and 20 kPa trilinear curves, respectively. Moreover, the GL7-GL8 curve also has re-verified the significantly underestimated capacity of (Bustamante et al. 2009). However, as reported in Figure 4, the GL7 and GL8 gage levels were placed in the two other soil layers and thus this seems to be the effect to the ultimate shaft resistance determination.

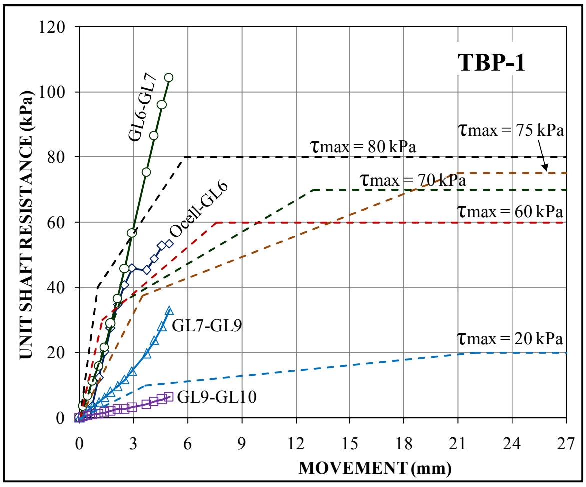

Figure 10. Tri-linear mobilization model of shaft resistances (Frank and Zhao 1982)

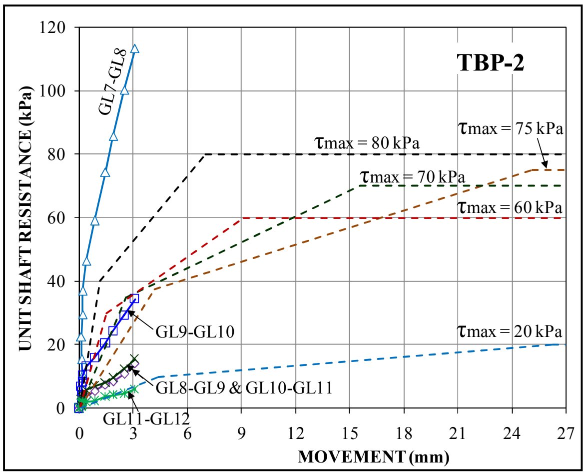

Figure 11. Tri-linear mobilization model of shaft resistances (Frank and Zhao 1982)

5 SUMMRARY AND CONCLUSION

The results of the O-cell tests on two instrumented bored piles and the analysis of the pressuremeter tests were presented.

The O-cell tests for Pile TBP-1 and TBP-2 have failed to meet the required test loads of 32 MN and 50 MN, respectively. The upward and downward maximum movements of Piles TBP-1 and TBP-2 at O-cell levels were 4.98 mm through 3.08 mm and 147.36 mm through 82.53 mm at maximum test loads of 10.25 MN and 15.23 MN, respectively. The maximum upward movement of the pile heads were 0.6 and 0.2 mm, respectively, which means that the shaft resistances along the upper pile lengths were only partially mobilized.

The measured pile toe stress-movement curves indicate the pile toe stiffness is very low and not display a capacity condition for the pile toes placed in compact to dense sandy silt.

The analysis of the unit shaft resistance curves adjacent to O-cell show some peak resistances and the tendency to post-peak softening. These unit shaft resistances are significantly small and it is likely that these shaft resistances for both piles are affected by the presence of left-behind slurry filter cake between the pile and the soil.

The pile capacity analysis from the pressuremeter tests shows the average of the unit ultimate shaft resistances about 20 kPa to 20 m depth, 60 kPa from 18 m through 25 m, 70 kPa between 25 m and 40 m, 75 kPa from 40 m to 45 m, and 80 kPa below 45 m to 52 m depth.

The t-z curve analysis modeled as a bilinear elastic-plastic curve (Frank and Zhao 1982) indicates most of the initial and second lines stiffer and softer than the actual measured curves, respectively. In addition, some measured curves provide an important evidence of the significantly underestimated capacity from the chart of (Bustamante et al. 2009).

The measured GL9-GL10 and GL11-GL12 curves of Pile TBP-2 have a good agreement with the initial lines of the 70 kPa and 20 kPa tri-linear curves, respectively.

.

6 REFERENCES

Bustamante, M., Gambin M. and Gianeselli L. 2009. Pile Design at Failure Using the Ménard Pressuremeter: an Update, *Contemporary Topics in In Situ Testing, Analysis, and Reliability of Foundations*, *Proc Int Foundation Conrgess and Equipment Expo'09* (IFCEE'09), Orlando, Florida, 15-19 March, Iskander M., Debra F. Laefer D. F. and Hussein M.H. (Eds), ASCE, Geotechnical Special Publication No. 186: 127-134.

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,* Volume 2, Issue 3, 196-207.

Frank, R. and Zhao S.R. 1982. Estimation par les paramètres pressiométriques de l'enfoncement sous charge axiale de pieux forés dans des sols fins (Assessment of the settlement under axial load of bored piles in fine-grained soils by means of pressuremeter parameters). *Bull. Liaison Labo.P. et Ch.,* 119(May-June, Réf. 2712): 17-24.

Fugro Loadtest 2010. "Report to Bauer Vietnam Ltd. on bored pile load testing (Osterberg method). Sunrise City Project, HoChiMinh City, Vietnam". Projects LTI 2875-1 and 2875-2, Singapore, 155 p.

Gambin, M. and Frank R. 2009. Direct Design Rules For Piles Using Ménard Pressuremeter Test. *Contemporary Topics in In Situ Testing, Analysis, and Reliability of Foundations*, *Proc Int Foundation Conrgess and Equipment Expo'09* (IFCEE'09), Orlando, Florida, 15-19 March, Iskander M., Debra F. Laefer D. F. and Hussein M.H. (Eds), ASCE, Geotechnical Special Publication No. 186: 111-118.

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

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