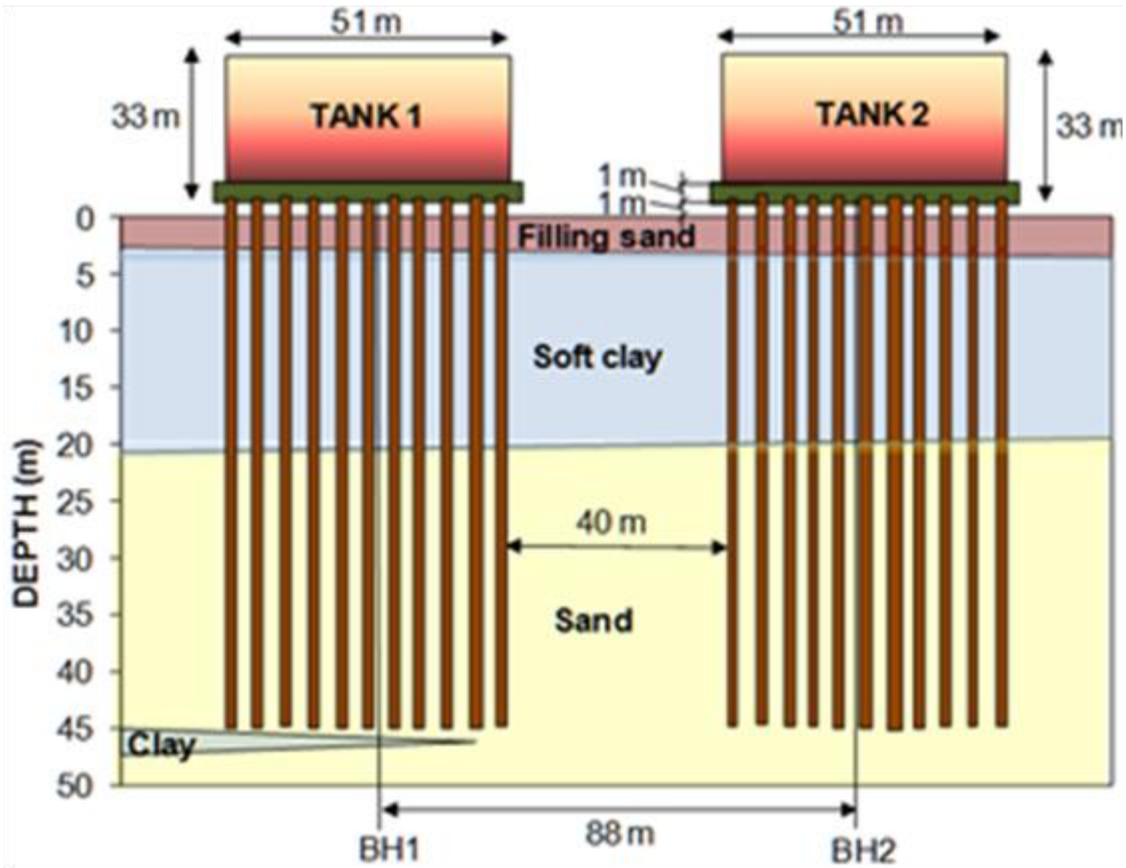

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ORIGINAL PAPER

Results of Static Loading Tests on Single Piles and on Pile-Supported LPG Tanks

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Abstract Full-scale static loading tests on eight single test piles and hydrotests on two 50-m diameter tanks supported on 849 piles intended for storing refrigerated gas were performed at Cai Mep Industrial Park approximately 90 km southeast of Ho Chi Minh City, Vietnam. The test piles were precast concrete piles installed through 21 m of soft clay into dense sand to about 45 m depth below ground surface. Four piles were installed by driving and four by jacking. The maximum pile test loads were 3,000 kN, which was well below any ultimate resistance, but showed that the tank-foundation piles (to be installed by jacking) could expect to shorten about 3 mm for the 750-kN working load. Each tank was hydro-tested to 636-MN maximum service load, which load was held constant for 1 week, while monitoring the tank settlement at benchmarks placed along the tank perimeters. The records showed the tank perimeters to settle about 15 mm in addition to an about 3-mm pile shortening. Back-analysis of the tank foundation modeled as flexible equivalent rafts at the pile toe level showed that the settlement of the tank center was about three times larger than that of the perimeter. Analysis of long-term settlements indicated that, under service loading, the tank perimeters and centers will settle an additional about 100 mm and 300 mm, respectively. Due to a 3 m thick fill placed over the site causing the clay to consolidate, the ground surface is expected to settle more than 1 m over the long-term. The general subsidence will affect the perimeter piles and transfer load to the interior piles. However, because the pile neutral plane is located in the sand below the clay, downdrag is not an issue for the piled foundation. The drag force will be well below the limit of the pile axial structural strength.

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Keywords Single piles · Pile groups · Static loading tests · Hydrotests · Load-movement curves

1 Introduction

In the last decades, many attempts were done to estimate the settlement of pile group under a sustained load, e.g., from the Equivalent Raft Method (Terzaghi and Peck 1967), Methods based on single pile load-

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movement curves (Meyerhof 1976), Equivalent Pier Method (Tomlinson 1986), and Unified Design Method (Fellenius 2018). However, much uncertainty still exists for estimating the response of a wide pile group to load.

In practice, for a piled foundation project, the design involves relating the working load to the pile bearing capacity as, frequently, based on the results of a static loading test performed on one or more single piles. The design for settlement is most often assumed 'automatically' assured if the 'factor of safety' is sufficient for the bearing capacity design. When considered, specifically, it is usually estimated from the results of a static loading test.

The soil profile at the subject site consists of soft, normally consolidated, highly compressible clay on silty sand. The groundwater table lies near the ground surface (Elev. +1.0 m) and rises can seasonally rise to 0.5 m above the ground, at times, to Elev. +1.5 m. This necessitated raising the land to Elev.+3.0 m, 1.5m above the flood level before constructing the LPG storage tanks.

This paper presents assessment of piled foundations for two storage tanks, where settlement was the dominant aspect of the tank long-term performance. The tanks were two refrigerated Propane and Butan Gas, LPG, storage tanks constructed over a 5.3 ha area at Cai Mep Industrial Park approximately 90 km southeast of Ho Chi Minh City, Vietnam (Fig. 1). They were 51 m in diameter and 33 m in height and designed to contain 30,000 tonne of refrigerated Propane or Butane and intended to operate for 30 years. Each tank is supported on 849, 400 mm diameter jacked-in square precast concrete piles installed into dense sand at 45 m depth.

A test programme was performed including static loading tests on single piles, a dynamic pile test, and hydrostatic loading tests on each group to 636-MN total load maintained over a period of 40 through 45 days.

2 Soil Investigation

2.1 General Site Conditions

The soil investigation program of the project consisted of boreholes to 50 m depth, standard penetration tests (SPT), in situ field vane tests (FVT), and cone penetration tests (CPTU) located as shown in Fig. 2, which figure also shows the location of the two tanks, about 50 m edge-to-edge apart. Soil samples obtained with 76-mm diameter fixed piston samplers were used for classification and strength tests per direct shear test (DS), unconfined (UC) and unconsolidated-undrained (UU) compression tests, and consolidated-undrained (CU) triaxial.

The soil investigation indicated that the soil profile at the two tanks consisted of a 3 m thick old sand fill on soft clay to about 20 m depth, underlain about 25 m of silty sand followed to large depth by medium dense to dense sand, becoming very dense below 50 m depth. The clay soils above 20 m depth are normally consolidated and compressible—expressed in Janbu modulus number, the compressibility was about 8. Figure 3 shows the distribution of water content, consistency limits, grain size distribution, and cone stress, q_t. The average natural water content ranged from 100% at 3 m depth to 50% below 3 m depth to 20 m depth and about 20% below this depth to 50 m depth. The soil layers below 50 m depth are mainly dense to very dense sand, but silt and clay lenses or zones are common. The average saturated density of silty clay was about 1,530 kg/m^3 (from water content, w_n = 79%). The average density of sand immediately below the silty clay was 1,950 kg/m^3 (determined from w_{n-} = 21%). An about 1 m thick lens of silty clay was found at 45 m depth with a saturated density of is about 1,730 kg/m^3 (determined from w_n = 41%). The groundwater table was at 1.0 m below the ground surface (November 2009) and the pore pressure distribution was hydrostatic. Figure 4 shows the distribution of SPT N-indices from the four boreholes. Between about 3 m through 20 m depth, the SPT N-index was no more than about 1 blow/ 0.3 m, which signals a very soft condition for the clay. From 20 to 50 m depth, the SPT N-indices indicated compact to dense to very dense conditions.

2.2 Undrained Shear Strength Measurements

Figure 5 shows the distribution of undrained strength, s_u, as determined from UU-tests, VST-results, CPTU-results, and DS-results together with a diagram of the distribution of natural water content at the site. The distribution of s_u with depth is fitted into three equations of s_u as a function of the initial effective overburden stress. The values of s_u/σ'_0 range from 0.1 through 0.3, corresponding to a range of plasticity

Fig. 1 Satellite image of the project area

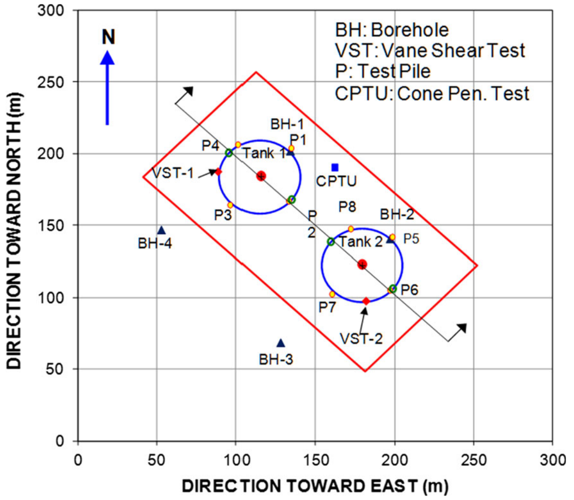


Fig. 2 Locations of LPG refrigerated storage tanks, Tanks 1 and 2, and the field tests

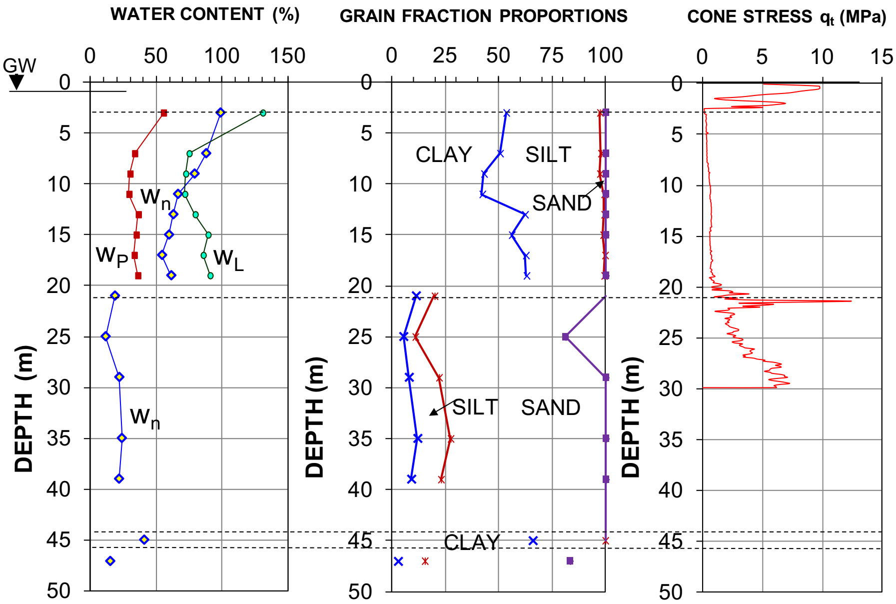
indices from 41 through 75%, which indicates near-normally consolidated condition (Bjerrum and Simons 1960; Bjerrum 1973; Ladd and DeGroot 2003).

As can be seen in Fig. 5, the evaluations of undrained strengths differ considerably. The normal stresses and the initial chamber pressures of direct shear and triaxial compression tests were in range of 25 kPa through 75 kPa and 20 kPa through 440 kPa, respectively. These strengths were about 50 and 30% of the in situ vane strengths of VST1 and VST2, respectively. The normalized cone stress, q_t - σ_{v0}, is usually correlated to the undrained shear strength, s_u, by division with a correlation coefficient, N_{kt}, ranging from 10 through 20 (Aas et al. 1986 and Rad and Lunne 1988). Here, the correlation was obtained by $N_{kt} = 20$ for a fit to the VST-distribution and $N_{kt} = 37$ for a fit to the laboratory test results, respectively. The $N_{kt} = 37$ was significantly greater than the reported values. It is likely that tested soil samples were disturbed. This means that the compressibility of the clay may be larger than indicated by the compressibility tests. However, a modulus number of 8 is in agreement with results obtained from nearby investigations (Fellenius and Nguyen 2013).

3 Design of the Storage Tanks

The two LPG tanks, Tanks 1 and 2, were designed as reinforced concrete tanks to contain 30,000 tonne refrigerated gas and operate for a 30-year life period.

Fig. 3 Consistency limits, grain fraction proportions, and CPTU cone stress



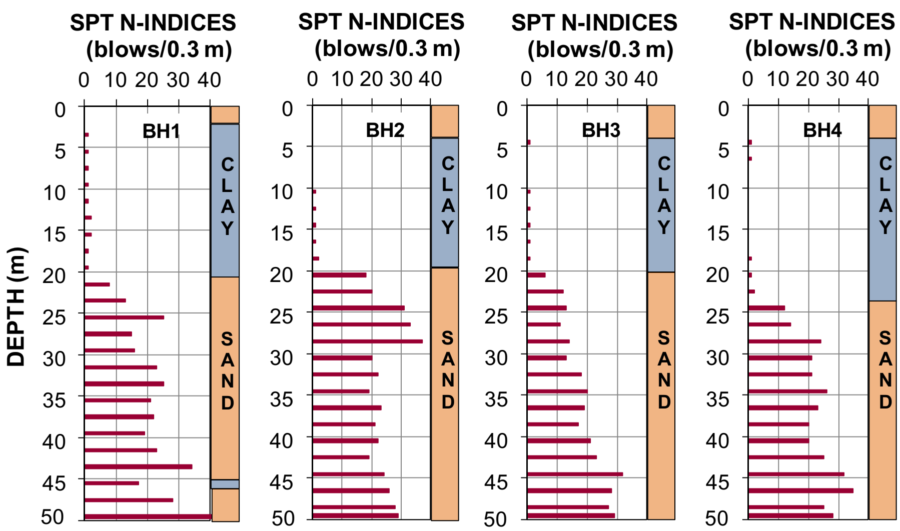
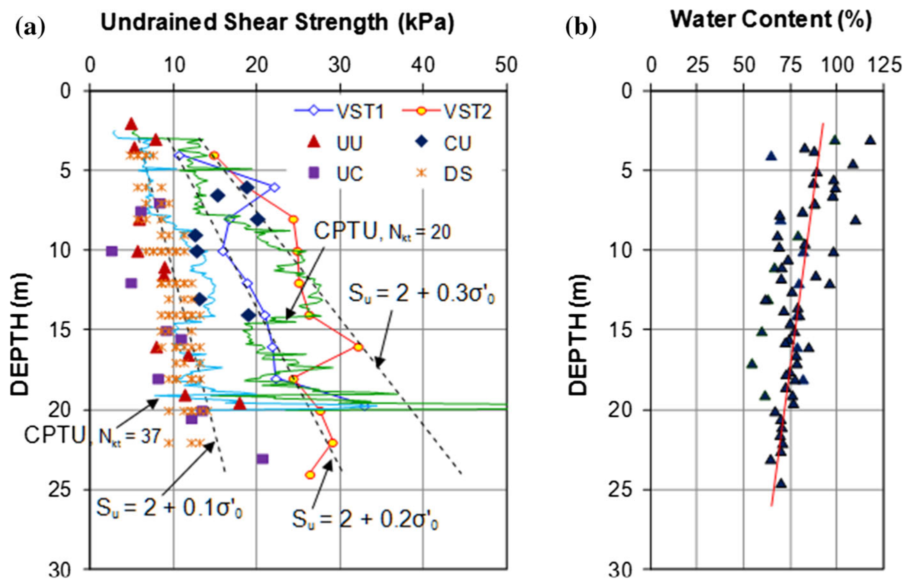


Fig. 4 N-indices of Borehole BH1 through BH4

The internal and external diameters of each tank were 49 m and 51 m, respectively; i.e., the total cross-sectional area was about 2,000 m^2. The height of tanks was 33 m. The maximum liquid level of Propane and Butane tanks were 28.7 m and 27.8 m, respectively. The as-empty and operating weights of each tank were 29 and 636 MN, respectively, corresponding to 15 and 320 kPa stress over the tank area as-empty and as-operating, respectively. Each tank was supported on 849 piles installed by jacking to about 45 m depth. The piles were 400 mm diameter, square, precast concrete piles cast in 15 m long segments, which were spliced in the field by welding end plates together. The center-to-center distance between the piles was 1.60 m (4 times the pile face-to-face diameter), but for the two outer rows that had a spacing of about 1.0 m (2.5 times the pile diameter). The pile cap was constructed as a 1.0 m thick reinforced-concrete slab placed 1 m above the ground surface. Figure 6 shows a vertical section of tanks, piles, and soil profile and Fig. 7 shows the

Fig. 5 Distribution of undrained shear strength and water content



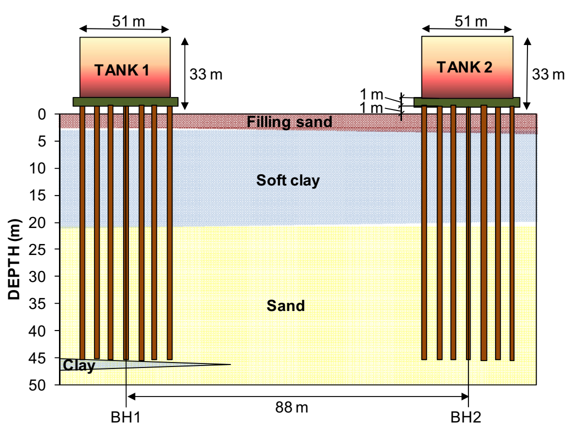


Fig. 6 Generalized soil profiles of the twin refrigerated LPG storage tanks

pile layout and the location of the settlement measuring points.

The average sustained working load was about 750 kN/pile. The sustained working load on the two perimeter pile rows was about 1000 kN/pile. To confirm the design, eight static loading tests were performed: Piles P2 and P4 in Tank 1, and P6 and P8 in Tank 2 were tested to a 2000-kN maximum load and Piles P1 and P3 in Tank 1 and P5 and P7 in Tank 2, located 2.0 m outside the tank footprint were tested to 3000-kN maximum load, respectively (see Table 1). After construction, the tanks were "hydrotested" to 100% of working load, 636 MN. Ten settlement monitoring benchmarks were placed along the perimeter of the tanks. No efforts to measure also the settlement across the tanks, say, along a diameter, were made.

4 Single-Pile Test Programme

4.1 Construction of the Test Piles

The eight test piles were installed during the last week of December 2010. Piles P1 and P2 in Tank 1 and Piles P7 and P8 in Tank 2 were driven by a Kobe KB45 diesel hammer with 4,500-kg ram weight and 124-kJ nominal energy. Piles P3 and P4 in Tank 1 and Piles P5 and P6 in Tank 2 were installed by jacking. The jacking installation was by a hydraulic jack system with 6-MN weight, but limited to a 4,000-kN maximum jacking force. When jacking Pile P5, the pile encountered a hard soil layer at 34.2 m depth preventing further penetration. The other seven piles were installed to about 45 m depth, as designed. See photographs in Fig. 8a and b.

The driving and jacking of the piles was recorded in terms of penetration resistance, PRES (blows/m) and the jacking was recorded as jacking force (kN) versus depth, respectively, as shown in Fig. 9 for the driven piles P1 and P7, and for the jacked piles P3 and P5. (Because, as will be shown below, the static loading tests did not add much information, the results of Piles P2, P4, P6, and P7 are not included). The compilation

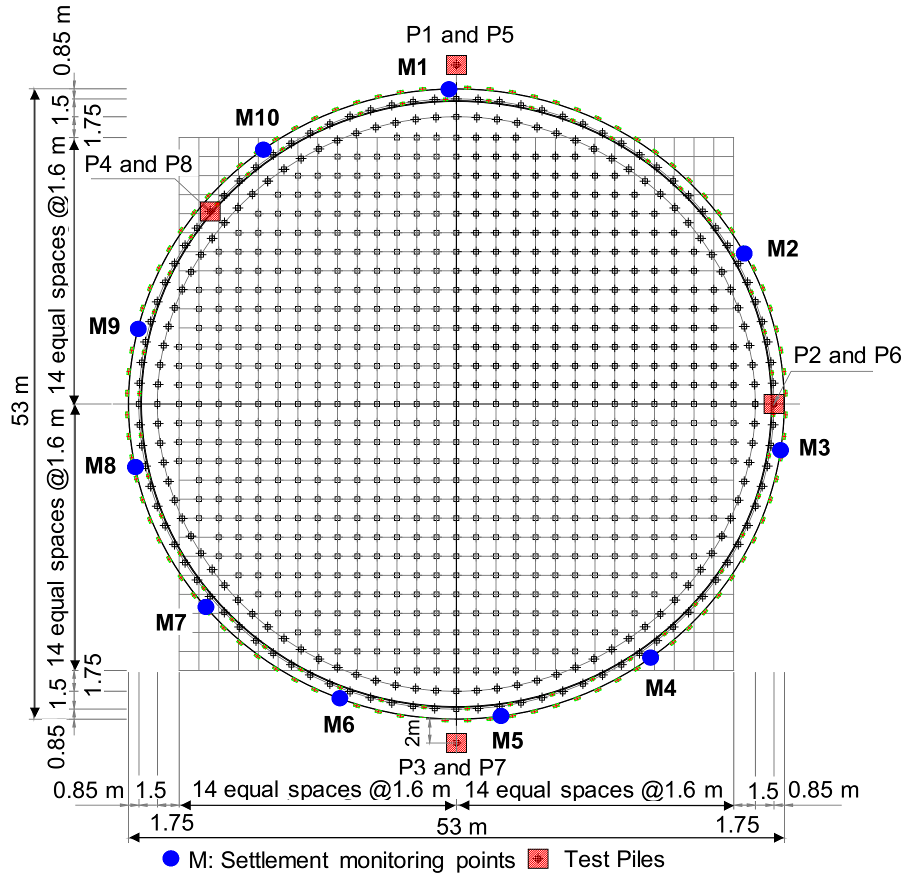


Fig. 7 Pile layout, location of test piles, and settlement measuring points (same for both tanks)

Table 1 Details of the piles and maximum test loads

Pile name pile	Embedded depth (m)	Installation method	Max test load (kN)
Tank 1			
P1	43.6	Driving	3,000
P2	45.6	Driving	2,000
P3	44.0	Jacking	3,000
P4	45.0	Jacking	2,000
Tank 2			
P5	34.2	Jacking	3,000
P6	45.5	Jacking	2,000
P7	46.2	Driving	3,000
P8	45.2	Driving	2,000

shows that the installation resistance for the pile in terms of PRES (blows/m) and jacking force was similar for the piles.

The jacking installation is equivalent to performing a rapid static loading test on the pile. The maximum jacking force applied to Piles P5 and P3 at termination of jacking indicated 3,000 and 4,000-kN ultimate resistances, respectively. Therefore, in static loading tests performed some time (7 days, actually) after the installation, it is expected that the static tests would show an ultimate resistance equal to the maximum jacking force with, some addition due to soil set-up. As no relaxation (reduction of capacity after installation) was expected, one would consider such static loading

Fig. 8 **a** Hydraulic jacking system (Authors' photo). **b** Pile driving by KB45 diesel hammer (Authors' photo)

tests to be redundant unless the tests would be brought to a maximum load much in excess of the applied jacking force.

4.2 Results of the Static Loading Tests

The static loading tests were performed on December 28, 2010 through January 07, 2011, about a week after installation the test piles (and before the installation of the construction piles). The tests were carried out using cycles of unloading and reloading. The loading was performed in a first cycle with 250-kN increments to 1,000 kN and then unloaded in two steps. The pile was then reloaded (second cycle) using increments of 250 kN to the maximum load (2,000 kN for P2 and P4 and 3,000 kN for P1 and P3). Unloading was in four and six steps, respectively. Each of the loading and reloading levels was held during 60 min and the maximum load was held for 24 h. Each of the unloading levels was held for 30 min and the zero load after unloading was held for 60 min before reloading.

The results of static loading tests are compiled in Figs. 10 and 11. The results show that (1) even at the maximum load of 3,000 kN, the piles had not reached an ultimate state, (2) at a load equal to the average working load, the pile-head movements of the jacked piles were 3–4 mm and for the driven piles, the movements were about 5–7 mm. The difference between the piles installed by the two installation methods is due the fact that the built-in residual load in a jacked pile is much larger than that in a driven pile (Fellenius 2014), and (3) the test results verified that the response of the single piles met the requirements of the design. The long-term ground subsidence around the tanks will impose negative skin friction the perimeter piles, transferring their load and some drag force to the interior piles. The perimeter piles will be able to accept this load without appreciable additional movement. A back-calculation of the loading test results and the results of the measured resistance to the jacking as well as reference to the CPTU sounding, indicated that the neutral plane for the perimeter piles will be well into the sand layer and, therefore, the settlement in the compressible clay above 21 m depth is not going to cause appreciable downdrag on the piles. The eventual drag force and maximum axial load in the perimeter piles are estimated to be about 1,500 kN and 2500, respectively, which are acceptable for the pile structural axial strength. The interior piles will not experience any significant drag force or downdrag.

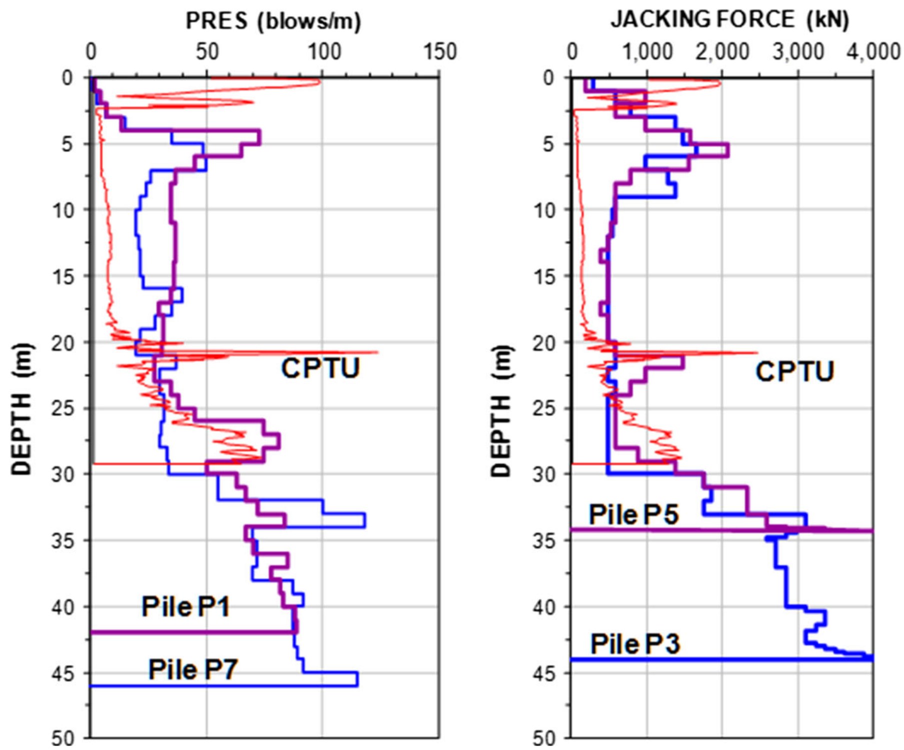


Fig. 9 Pile driving and jacking diagrams for the driven Test Piles (P1 and P7) the jacked Test Piles (P3 and P5). To each, the CPTU cone stress (q_t) diagram is added for reference. A 5-MPa q_t-stress corresponds to scale points of 150 blows/m and 3000 kN in the pile driving and pile jacking diagrams, respectively

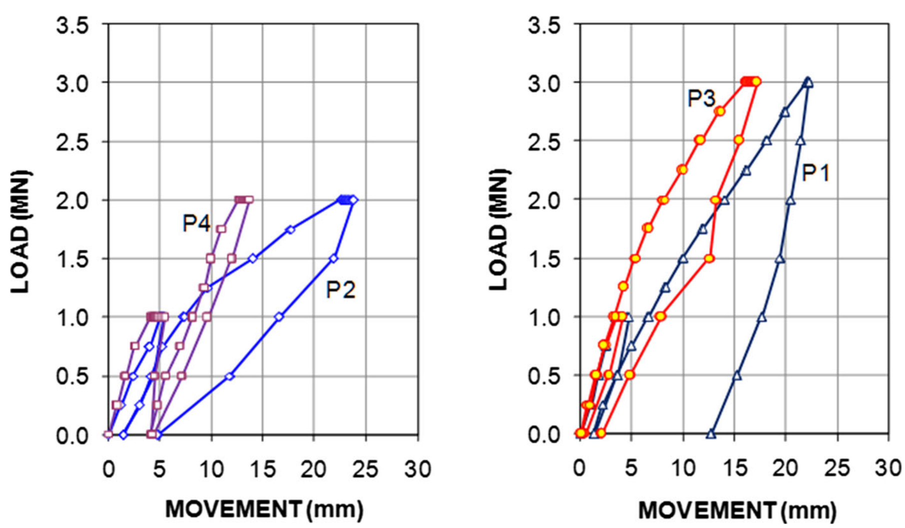


Fig. 10 Load versus movement of Piles P2 (driven) and P4 (jacked) and Piles P1 (driven) Pile 3 (jacked) in Tank 1

4.3 Results of the Dynamic Pile Tests

As the piles installed by jacking method showed a stiffer response to load than the driven piles, the jacking installation method was selected for the project. The pile installation for the two tanks started on January 22, 2011 and was completed on March 10, 2011. Figure 12 shows a photograph of Tank 2 about 8 months after completed construction.

During the installation of the piles, the jacking of one pile was interrupted at two intermediate depths, 27 and 32 m, and subjected to dynamic tests using a light pile driving hammer. After completed installation, the pile was again subjected to a dynamic test. The test at the 27-m depth was after the pile entered the more

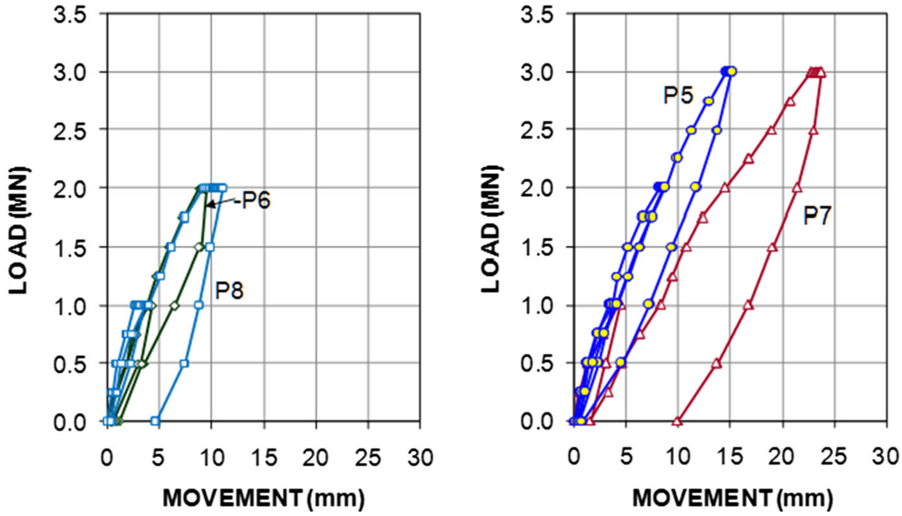


Fig. 11 Load versus movement measured in static loading tests in Tank 2 on Piles P6 (jacked) and P8 (driven) and Piles P5 (jacked) and P7 (driven)

Fig. 12 Tank 2 after construction (Authors' photo)

resistance soil, appearing at about 30 m depth in Fig. 9. The light hammer could only mobilize the full resistance at the 27-m depth. The PRES was 600 blows/m (15 blows/25 mm) and the transferred maximum energy was 7 kJ. The CAPWAP-determined pile capacity was 2,100 kN and the shaft and toe resistances were 1,800 and 300 kN, respectively. The 2,100-kN CAPWAP capacity agrees with the jacking resistance showed for the test piles (Piles P3 and P5). Unfortunately, no records were kept of the jacking force for the three depths. Both the other two dynamic tests showed a PRES exceeding 1,000 bl/m (30 bl/25 mm), a 9-kJ transferred energy and a 2,500-kN mobilized CAPWAP-determined capacity.

5 Pile Group Test Programme

5.1 Hydrostatic Loading Tests

The hydrostatic loading tests on Tanks 1 and 2 were performed on July 1 through 14, 2012, and August 14 through 28, 2012, respectively. The tests were for both tanks carried out in one loading cycle and included 12 through 11 increments to 636 MN maximum load (average 750 kN/pile). Unloading was in 11 through 5 unload levels for Tank 1 and 2, respectively. The loading was by means of raising water level inside tanks by pumping water from the Thi Vai River. The pumping rate aimed to reach each load level within 24 h of start of pumping. The settlement measuring points were surveyed immediately after each completed load level was reached and before the next load increment was started. The last load level was maintained during 6 days and 8 days for Tanks 1 and 2, respectively.

The loads and settlements over time are presented in Figs. 13 and 14 for Tanks 1 and 2, respectively. For Tank 1 on reaching the maximum load, the minimum and maximum settlements (Benchmarks M1 and M4) are 12 and 17 mm, respectively, and the average settlement is 14 mm. For Tank 2, the minimum and maximum settlements (Benchmarks M2 and M4) are 16 and 20 mm, respectively and the average settlement is about 18 mm.

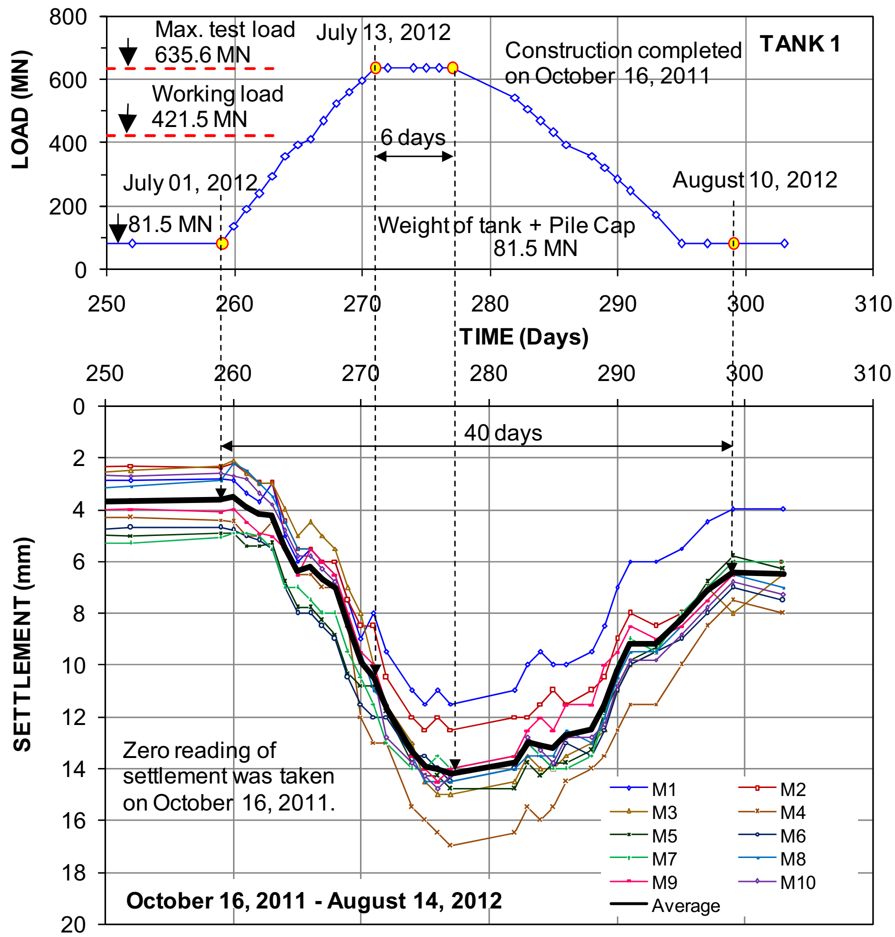


Fig. 13 Load versus settlement of Tank 1 during hydrotesting

The average additional settlements during the 6th and 8th load holding days for Tanks 1 and 2 were 4 and 3 mm, respectively, and the average rebounds were 3 and 8 mm, respectively. Unfortunately, the settlements during construction were not recorded, which means that the settlements due to the tank self-weights are unknown. However, proportioning to the observed settlements from the hydrotest indicates that the self-weight settlements were about 3 mm. Thus, the total average hydrotest perimeter settlement of Tanks 1 and 2 were about 17 and 21 mm, respectively. Assuming that the pile shortenings were about the same as those measured in the static loading tests on the jacked single piles, this suggests that the settlement below the pile toe level around the tank perimeters was about 15 mm.

6 Settlement Analysis and Discussion

Fellenius and Ochoa (2016) and Fellenius (2018) have shown that settlement of a wide piled foundation loaded by a sustained load can be calculated as the settlement of a flexible equivalent raft at the pile toe level plus the "equivalent-pier" shortening of the piles due to the transfer to the soil of the applied load. Thus, the settlement of the foundation depends very much on the compressibility of the soil layers below the pile toe level and the width of the pile group. (In contrast, the load applied to a single pile does not cause settlement in the soil layers below the pile toe, but experiences load transfer movements).

Both the static loading tests and the hydrotests represent short-term conditions. The settlement of the tank perimeter minus the pile shortening for the load can therefore be analyzed at the immediate settlement of an equivalent raft placed at the pile toe depth for the

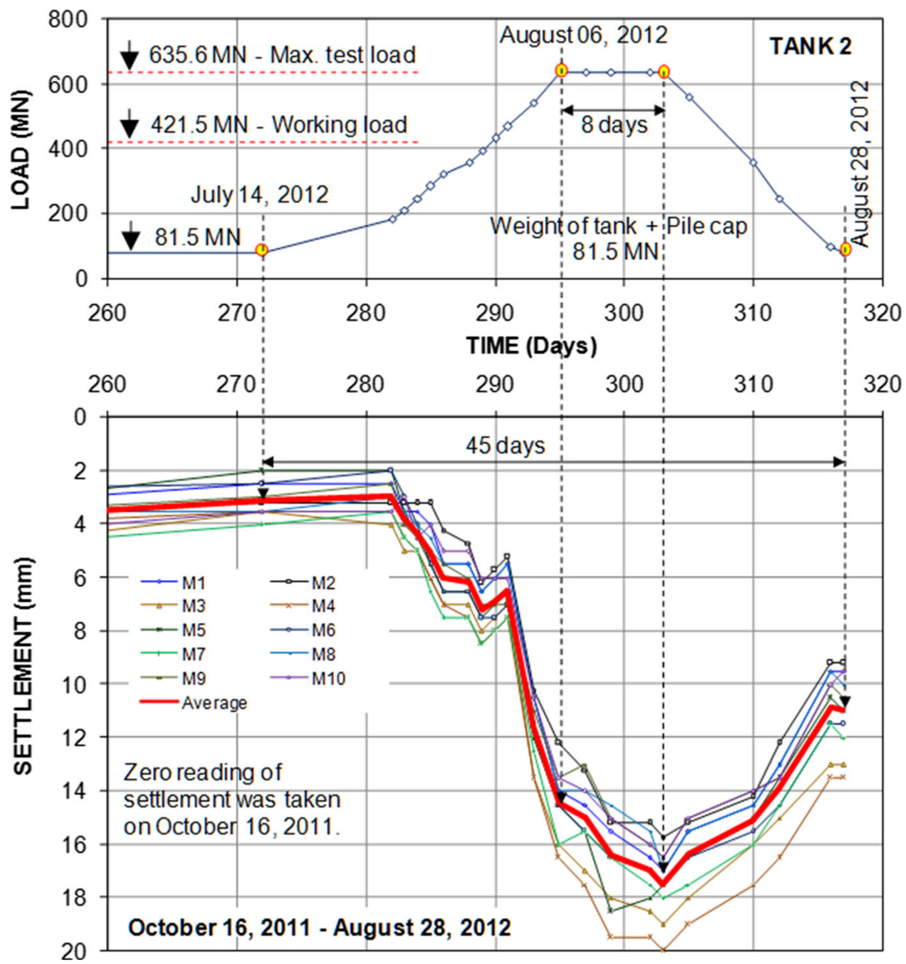


Fig. 14 Load versus settlement of Tank 2 during hydrotesting

appropriate soil compressibilities and thicknesses of the soil layers below the pile toe. We assume that a calculation input matched to the observed perimeter settlement will be representative for the settlement of the tank center. For any and all combinations of compressibilities and soil layers that indicate a 15-mm settlement of the perimeter, the calculated settlement of the tank center is about three times larger, i.e., 45 mm.

The soil layer below the pile toe depth will be affected by consolidation settlement. Even if assuming a Janbu modulus number as large as in the 200–600 range, the long-term tank settlements will likely be close to 100 mm at the tank perimeter and 300 mm at the tank center.

Moreover, the fill will cause significant consolidation settlement of the clay, across the site. In time, the consolidation settlement will likely be in excess of 1 m. The tanks will, therefore, appear to rise above the ground. Before their 30-year life span is over, it is probable that additional fill will have to be brought into avoid flooding of the area around the piled foundations. This will cause additional settlement for the tanks.

7 Summary and Conclusion

Full-scale static loading tests on eight single test piles and hydrotests on two 50-m diameter tanks supported on 849 piles were performed. The test results and analysis were presented. The conclusions drawn from this study are as follows:

- The static test results verified that the response of the single piles met the requirements of the design and the test piles had not reached an ultimate state.

The bearing resistance of the jacked piles is 1.5 times greater than that of the driven piles at maximum movements of jacked piles.

- Back-calculation indicated that the neutral plane of the perimeter piles will be well into the sand layer and the settlement in the compressible clay above 21 m depth is not going to cause appreciable downdrag on the perimeter piles. The drag force will be well below the limit of the pile axial structural strength.
- The dynamic test results showed that the 2,100-kN CAPWAP capacity agrees with the jacking resistance for the test piles.
- The hydrotests showed the tank perimeters to settle about 15 mm in addition to pile shortening.
- Analysis of the tank foundations modeled as flexible equivalent rafts showed that the settlement of the tank center was about three times larger than that of the perimeter.
- Analysis of long-term settlements indicated that, under service loading, the tank perimeters and centers will settle 100 and 300 mm, respectively.

References

Aas G, Lacasse S, Lunne T, Hoeg K (1986) Use of in situ tests for foundation design on clay. In: Proceedings of the ASCE special Conference In Situ'86: Use of In Situ Tests in Geotechnical Engineering, American Society of Engineers, Blacksburg, pp. 1–30.

Bjerrum L (1973) Problems of soil mechanics and construction on soft clays. In: Proceedings of 8th ICSMFE, Moscow, Vol. 3, pp. 111–159.

Bjerrum L, Simons NE (1960) Comparison of shear strength characteristics of normally consolidated clays. In: Proceedings of the ASCE special Conference on Shear Strength of Cohesive Soils, pp 711–726.

Fellenius BH (2014) Analysis of results from routine static loading tests with emphasis on the bidirectional test. In: Proceedings of the 17th Congress of the Brasiliero de Mecanica dos Solos e Egenharia, Comramseg, Goiania, Brazil, September 10–13, p 22.

Fellenius BH (2018) Basics of foundation design, a text book. Revised Electronic Edition, [www.Fellenius.net], p 468

Fellenius BH, Nguyen MH (2013) Wick drains and piling for Cai Mep container port, Vietnam. In: Armin WS, Barry RC (eds) Sound Geotechnical Research To Practice, ASCE GeoInstitute Geo-Congress San Diego, ASCE, Reston, GSP 230, pp 445–462.

Fellenius BH, Ochoa M (2016) Wide storage tanks on piled foundations. Geotech Eng J South-East Asian Geotech SocAssoc Geotech Soc South-East Asia 47(1) 50–62.

Ladd CC, DeGroot DJ (2003) Recommended practice for soft ground site characterization. In: Arthur Casagrande Lecture, 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering, Cambridge, Massachusetts, June 22–25, 2003.

Meyerhof GG (1976) Bearing capacity and settlement of pile foundations. ASCE Geotechn Eng 102(GT3) 195–228.

Rad NS, Lunne T (1988) Direct correlations between piezocone test results and undrained shear strength of clay. In: Proceedings of the International Symposium on Penetration Testing, ISOPT-1, Orlando, Balkema Pub., Rottterdam, 2, pp. 911–917.

Terzaghi K, Peck RB (1967) Soil Mechanics in Engineering Practice. Wiley, New York, p 729

Tomlinson MJ (1986) Foundation design and construction, 5th edn. Pitman Books Ltd., London, p 856.