Piling Foundation Design & Construction Problems of Tank Farm in Reclaimed Land over Untreated Soft Marine Clay in Malaysia

Liew Shaw-Shong[1] , Ting Deng-Ing[2] and Low Yew-Hup[3]
[1]*Director, G&P Geotechnics Sdn. Bhd., Kuala Lumpur, Malaysia*
[2]*Senior Engineer, G&P Geotechnics Sdn. Bhd., Kuala Lumpur, Malaysia*
[3]*Engineer, G&P Geotechnics Sdn. Bhd., Kuala Lumpur, Malaysia*
E-mail: [1]ssliew@gnpgroup.com.my, [2]diting@gnpgeo.com.my

ABSTRACT: This paper presents the engineering challenges in the pile foundation design and construction of a tank farm project at a reclaimed platform over untreated soft marine deposits. Load transfer behaviour observed in the instrumented pile testing programme and tank settlement performance during the hydrostatic load testing scheme were both far better than the design prediction. Problems of installing large displacement pile inducing the soil heave causing excessive tensile stress at the pile joints and lateral soil displacement causing additional flexural stress to the adjacent piles have become a major construction problem in a very tight construction programme. Lesson learnt from the foundation design aspects with the consideration of construction problems and hydrostatic load test results for the tanks are discussed in this paper.

1. INTRODUCTION

This paper presents a tank farm project implemented over a soft swampy ground near to a river mouth close to coastal line of southern region of Malaysia. The site platform was reclaimed with cut residual materials from the adjacent areas to a fill thickness of 5m to as high as 10m. There was no ground improvement work carried out to the weak foundation soils beneath the reclaimed fill.

In Phase I development, there were seven numbers of steel tank structures constructed for storage of petro-carbon products with specific gravity (SG) of about 0.8. Six of the tanks with dimension of about 38m diameter and 20m in height have a net storage capacity of 20,000m^3 meanwhile only one tank with smaller dimension of 33m diameter and 15m in height has a net storage capacity of 10,000m^3. The Phase I tank farm layout consists of two rows of tanks with three numbers of ∅38m tanks arranged in a row whereas the smallest tank is located next to the first row tank.

Due to tight project delivery schedule, ground treatment by the means of pre-loading method was not practical for the overall tank farm platform. As such, conventional pile foundation system consisting of reinforced concrete piles and structural raft slab was adopted to support the tanks. During the construction period, installation of large displacement pile in group had brought up problems of ground heave and lateral soil displacement, causing detrimental effects onto the installed piles. Instrumented pile testing and hydrostatic testing programmes were executed respectively during the course of the works and after completion of tank structure in order to monitor and validate the actual performance of pile and piled raft.

2. SUBSOIL CONDITIONS

The site is underlain by two geological formations, namely superficial deposits and Gunung Pulai Volcanic member. The superficial deposits mainly consist of upper recent alluvium and lower older alluvium. The recent alluvium is constituted of unconsolidated deposits of sand and clay meanwhile the older alluvium is semi-consolidated formation of sand, clay and boulder beds of fluviatile and shallow marine origin. Gunung Pulai Volcanic member is mainly composed of tuff or rhyodacidic composition.

A total of twenty exploratory boreholes and fourteen piezocones were carried out over the entire tank farm. Based on the borelog profiles, the subsoil strata are made of three distinct strata, namely Recent Fill, Alluvium and Residual Soils weathered from sedimentary formation. The undrained shear strength of the soft alluvium layer is interpreted to be 18kN/m^2. Figure 1 shows the brief description of the established geotechnical model and relevant interpreted engineering parameters.

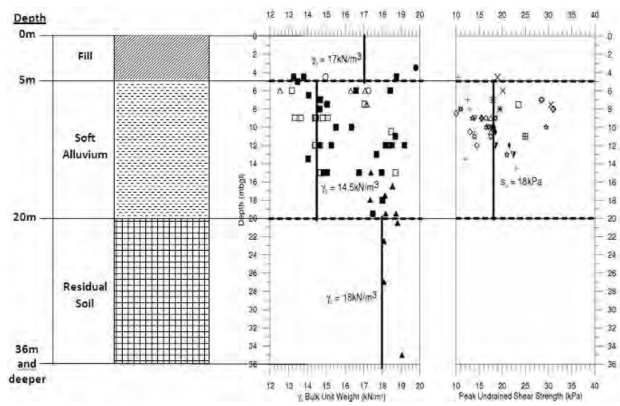

Figure 1 Established Geotechnical Model for Analysis

Dissipation tests conducted at various penetration depths during the piezocone testing show that the measured pore water pressures at end of the full dissipation tests were still in excess of about 20kPa to 35kPa compared to the estimated hydrostatic water pressure. It could possibly be the residual excess pore water pressure generated from the recent filling over the underlying soft compressive alluvial deposits. In other words, the soft fine soils are still undergoing process of consolidation.

3. PILE FOUNDATION DESIGN

Reinforced concrete (RC) square piles of 300mmx300mm or 400mmx400mm with concrete strength of 45MPa had been designed to support the tanks with RC raft. Typically, the RC piles were arranged in square grid within a circular RC raft. More importantly, RC piles near to tank peripheral had been positioned directly underneath the tank shell around the tank circumference to ensure minimal differential settlement circumferentially. Total numbers of piles for each tank vary from 265 to 317 depending on the subsoil condition at respective tank footprints. These piles were mostly designed to be installed until the end-bearing stratum in order to ensure consistency of pile stiffness in controlling raft distortion under the hydrotest loading. The end-bearing stratum is expected at the depth varying from about 33m to 46m for Phase I development.

There were two types of RC raft adopted, i.e. solely raft slab and raft slab with up-stand ring beam beneath the tank shell. The up-stand ring beam was also served as confinement to the sand bedding for the seating of tank base plates. All the tanks with cone-down base slope of 1/300 were seated either directly on the RC raft slab or on the sand bed contained within the RC ring beam.

The following criteria were used to verify the serviceability limit state of the tank raft structure on piles during hydrotest:

a. Total incremental pile settlement : 100mm
b. Raft distortion : 1/200

4. INSTRUMENTED PILE TESTING

At the beginning of the construction work, two sacrificial instrumented test piles using hollow prestressed spun pile were installed at two designated locations to reveal the load transfer behaviour of single pile during static load test. The reason of using the circular prestressed spun pile as the test pile was to allow installation of the proprietary Glostrext strain gauges system as part of the instrumentation scheme.

Figures 2 and 3 reveal the levels of instrumentation in the hollow spun pile of 350mm and 450mm diameter and the corresponding subsoil profiles of nearest boreholes ABH8 and ABH7 respectively.

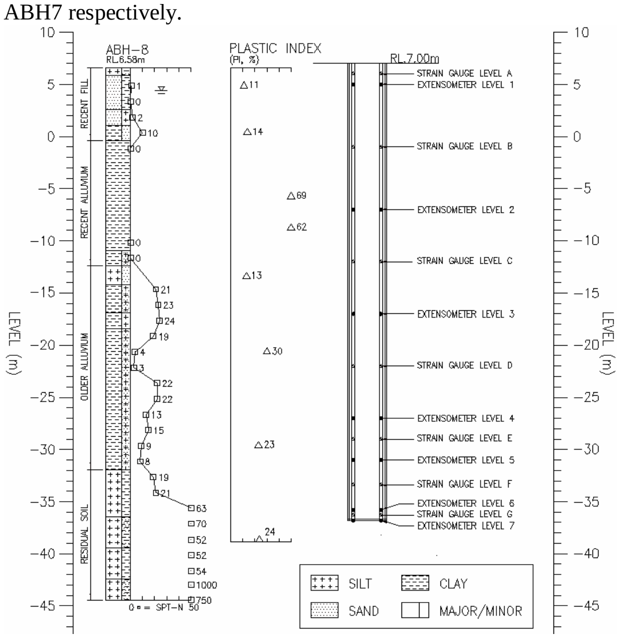

Figure 2 Instrumentation Levels for 350mm Diameter Spun Pile and Profile of Nearest Borehole ABH-8

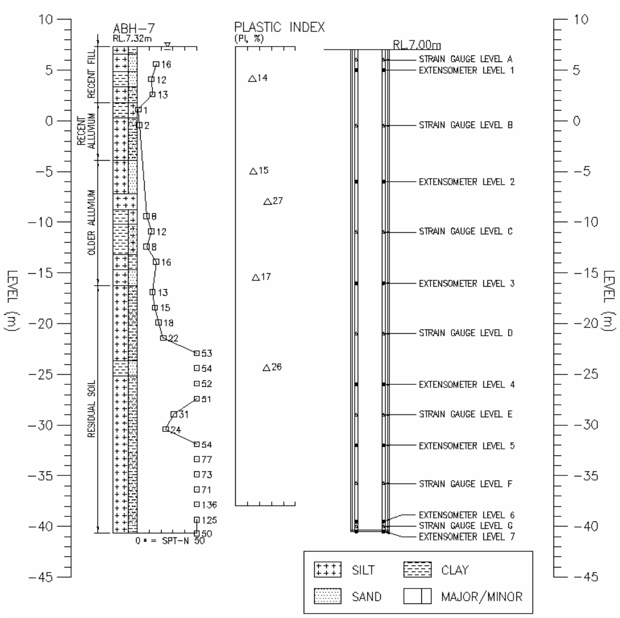

Figure 3 Instrumentation Levels for 450mm Diameter Spun Pile and Profile of Nearest Borehole ABH-7

The sacrificial test piles of 350mm and 450mm diameter were installed until the depth of 44m and 48m respectively. These pile sizes were selected based on its equivalent resemblance of sectional surface area and axial working load to the 300mmx300mm and 400mmx400mm RC square piles respectively. Strain gauges had been installed within the annulus of the hollow spun piles at multi levels. The strain gauge at the top level of the instrumented test piles was free from interaction of soil resistance and was used as calibrating strain gauge level to the load cell during static load test. The calibration is important for interpreting the axial load at other strain gauges at deeper levels in the same test piles.

Figures 4 and 5 show the axial compressive load applied on the pile top and its corresponding pile top settlement.

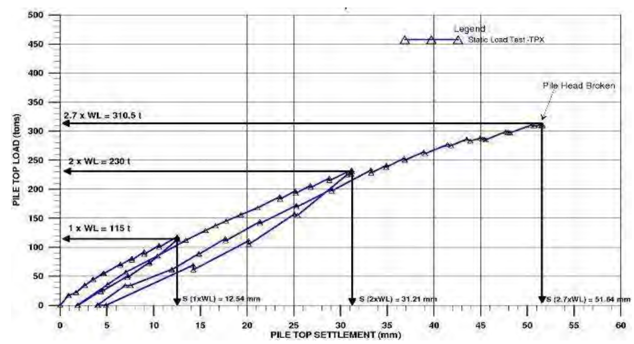

Figure 4 Load Settlement Curve for 350mm Diameter Spun Pile

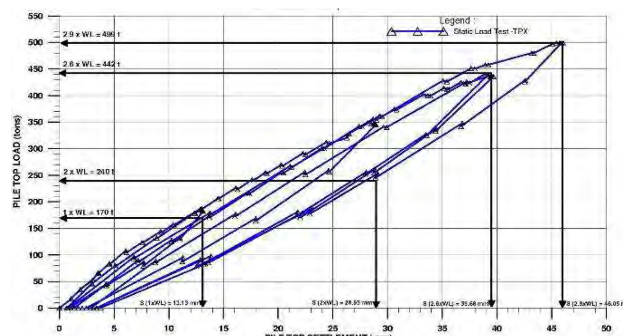

Figure 5 Load Settlement Curve for 450mm Diameter Spun Pile

From the instrumentation results, it was observed that there was insignificant shaft resistance transfer from the pile top level to the depth of 17 to 20m which corresponds to the soft alluvium stratum. After the soft alluvium, significant pile shaft resistance transfer had been registered over the balance of lower pile shaft and was increasing with depth. The maximum test load applied to the test piles was about 3 times the pile design axial load, but was still insufficient to fully mobilise the pile base resistance. Tables 1 and 2 present the back-calculated shaft friction factors ($f_{s,mob}$/N) and base factors ($f_{b,mob}$/N) by Meyerhof (1976) for the subsoil strata in this project site.

Table 1 Summary of Back-calculated Factors for 350mm Spun Pile at Maximum Test Load of 3105kN

Levels (mRL)	**Average SPT'N**	**Mobilised Pile Unit Resistances ($f_{s,mob}$ or $f_{b,mob}$)**	**Back-calculated Factors ($f_{s,mob}$/N or $f_{b,mob}$/N)**
-13 to -19	22	$f_{s,mob}$ = 88 kN/m^2	4.0
-19 to -23	3	$f_{s,mob}$ = 100 kN/m^2	33.3
-23 to -28	17	$f_{s,mob}$ = 111 kN/m^2	6.5^{++}
-28 to -31	8	$f_{s,mob}$ = 109 kN/m^2	13.6^{++}
-31 to -35	19	$f_{s,mob}$ = 80 kN/m^2	4.2^{++}
-37	63	$f_{b,mob}$ = 1671 kN/m^2	26.5^{++}

$^{++}$ denotes yet to be fully mobilised

Table 2 Summary of Back-calculated Factors for 450mm Spun Pile at Maximum Test Load of 4995kN

Levels (mRL)	**Average SPT'N**	**Mobilised Pile Unit Resistances ($f_{s,mob}$ or $f_{b,mob}$)**	**Back-calculated Factors ($f_{s,mob}$/N or $f_{b,mob}$/N)**
-10 to -16	8	$f_{s,mob}$ = 120 kN/m^2	15.0
-16 to -19	14	$f_{s,mob}$ = 144 kN/m^2	10.3
-19 to -23	19	$f_{s,mob}$ = 127 kN/m^2	6.7
-23 to -28	50	$f_{s,mob}$ = 112 kN/m^2	2.2^{++}
-28 to -34	25	$f_{s,mob}$ = 90 kN/m^2	3.6^{++}
-40	50	$f_{b,mob}$ = 1153 kN/m^2	23.1^{++}

$^{++}$ denotes yet to be fully mobilised

In comparison, the mobilised pile unit shaft resistance ($f_{s,mob}$) for the weathered sedimentary derivatives was relatively higher than the computed pile unit shaft resistance (f_{su}) based on the common Meyerhof-type empirical correlation of 2 to 3 times the SPT'N value, which is widely used in Malaysia.

5. HYDROSTATIC TESTING PROGRAMME

Upon completion of tank structure, all tanks had undergone the hydrotest in accordance with API 653: 2001. Basically, a series of settlement points would be set out around the outer face of tank shell circumference while the tank is still empty. Subsequent settlement readings are taken during the progressive filling of water until reaching 100 percent of test level to obtain the relative settlements.

In this project, there were total of 16 numbers of settlement points uniformly spaced around tank circumference as shown in Figure 6. At each of these settlement points, survey was taken at the top of RC raft and angle bars welded on the tank shell plate. The settlements measured at the 16 monitoring points when the water was at the highest level for all the tanks are summarised in Table 3.

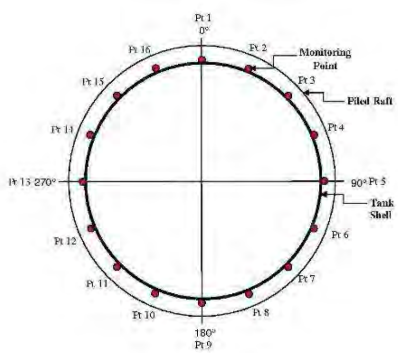

Figure 6 Layout of Settlement Points

Table 3 Settlement Measured on Top of RC Raft and Shell Plate around Tank Circumference During Hydrostatic Test

Tank (Height)	**Pile Size (Grid Spacing)**	**Measured Settlement (mm)**		**Predicted Settlement (mm)**
		RC Raft	**Shell Plate**	
A (20m)	400x400mm (1.9m)	-7 to 35	2 to 17	28
B (20m)	400x400mm (2.0m)	-4 to 39	3 to 23	36
C (20m)	400x400mm (2.0m)	-4 to 17	0 to 17	34
D (20m)	400x400mm (2.0m)	-1 to 27	4 to 28	40
E (20m)	400x400mm (1.9m)	-1 to 20	2 to 17	26
F (20m)	400x400mm (1.9m)	-3 to 8	6 to 24	35
G (15m)	300x300mm (1.8m)	-2 to 15	3 to 17	26

It was observed that, for the settlement measurement on top of RC raft, the settlement readings at some locations actually show negative values implying heaving deformation. The heave measured at the RC raft could be attributed to pivoting effect of RC slab under surface loading in which settlement is shown at loaded area meanwhile heave is observed at relatively unloaded area.

From the two sets of settlement readings whether on shell plate or RC raft, the settlements vary among markers despite the predicted settlement around the tank circumference, which is uniform. To explain such variation in the measured settlement, it would be likely due to the inherent uncertainty of pile support stiffness in reality, which could not be practically modelled in the design analysis.

6. PROBLEMS OF PILE INSTALLATION

The piling construction works for this project involve installation of large displacement piles with sizeable pile group in soft marine clay area. Moreover, due to the tight construction programme, there were 2 to 3 piling rigs working at every tank with total daily production of 10 to 17 pile points. This had induced significant ground surface heave and lateral soil displacement, which had negative effect to the installed piles.

Basically, the mechanism of ground heave and lateral soil displacement are that when displacement pile is driven into the soft marine clay of low permeability, excess pore water pressure would be instantly generated and the uncompressible cohesive soil would be displaced in every direction, i.e. vertically and radially. In particular, the soil near to the ground surface would tend to move upwards more than moving laterally or downwards as there is minimum confinement pressure at shallow depth. This scenario will lead to predominantly ground heave. Meanwhile, the lateral soil displacement could simply denote the scenario where soil moves radially.

When the soil moves upwards while the adjacent pile is in place, there is an interaction of soil and pile shaft inducing uplift force to the installed pile, and may cause the pile to move upward as well. With more piles being driven at the surrounding area, the initial installed pile would have gained and accumulated remarkable heaving in magnitude. Similarly, lateral soil displacement due to an on-going pile driving work would push the adjacent installed piles to move laterally. These scenarios can be well illustrated in Figure 7.

In extreme cases, the ground heave and lateral soil displacement could induce excessive tensile stress and flexural stress onto the adjacent piles. Being the weakest point at the pile joint, the pile joint could have broken apart or dislodged.

However, the occurrence of ground heave and lateral soil movement would be reduced in effect by the increasing in-situ overburden stress of the subsoil with the depth. In short, when a driven pile penetrates deeper into the subsoil, the heaving and lateral movement of soil would become gradually less significant.

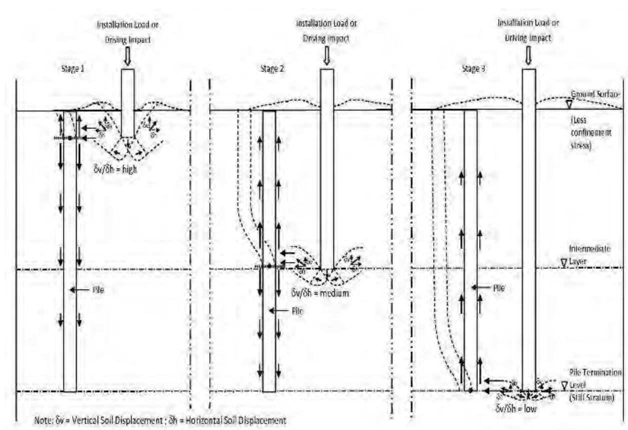

Figure 7 Illustration of Potential Ground Surface Heave and Lateral Soil Displacement Due To Driving of Displacement Pile

6.1 Ground/Pile Heave

A small-scale monitoring programme had been commissioned to reveal the heaving behaviour of the installed piles due to installation of adjacent large displacement pile. Twelve numbers of 400mmx400mm square RC working piles were installed with a pre-determined sequence and were monitored in a systematic way. The centre-to-centre spacing among the driven piles was about five times the pile size.

In brief, Pile No.1 was first installed until the required set and the initial base reading on the pile was taken by using optical levelling instrument, and the same work procedure was applied to Pile No.2 but after completion of driving for Pile No.2, second reading was taken on the first pile to determine the relative magnitude of heave and, at the same time, Pile No. 2 was surveyed for base reading. When Pile No.3 was installed in place, the survey readings were taken on the first pile again and second pile as well. The similar procedural steps were repeated for the rest of the selected piles until all 12 piles were driven. The layout of the driving and monitoring scheme is shown in Figure 8.

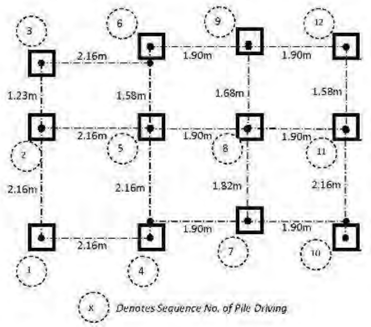

Figure 8 Layout of Driving and Monitoring Scheme

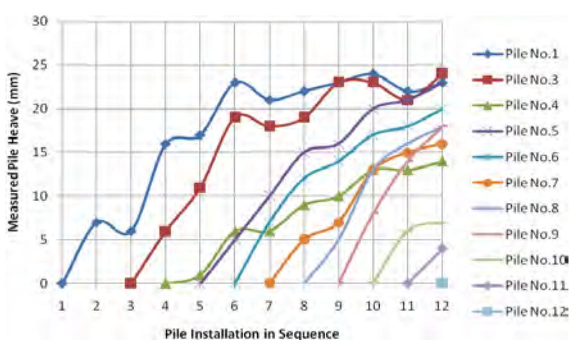

Figure 9 Plot of Measured Pile Heave in relation to Sequential Pile Installation

The measured pile heave in relation to driving of adjacent piles is shown in Figure 9. From the controlled monitoring, it was evident that pile heaving is real as a result of driving displacement pile at adjacent area, especially on the earliest installed pile. Unfortunately, heave reading for Pile No.2 was incorrectly taken at the time of monitoring and thus is not presented here.

6.2 Radial Soil Displacement

During the course of piling works for one of the tanks, the contractor had carried out the driving sequence started from the southern side towards the northern side of the tank due to site logistic consideration. From the piling records, the ground at the southern portion of the tank appears to be softer. Subsequently, there was a rare occurrence on site where the pile head of few earliest installed piles were found to be displaced rather exceptionally large in a range of about 700mm to 1050mm from its original positions. The group of the largely displaced piles consistently moved towards one main direction, i.e. southern side.

Theoretically, the total radial displaced soil volume and ground heave within the tank area shall not be more than the embedded volume of installed piles if the soils are considered incompressible. Hence, if ground heave and pile flexural stiffness are ignored for simplicity, the total area of outwards soil displacement at pile head level is about $51m^2$, which is equivalent to a radial soil displacement of 1350mm as a conservative estimate. Site observations evidenced that the resultant pile movements are primarily induced by instant displacement of uncompressible cohesive soil due to rapid pile driving and are mainly governed by the piling sequence. Unfortunately, no instrumentation was installed for verification of the radial soil displacement as well as the pile movement.

As earlier mentioned in Section 6.0, there could be possibility of pile joint dislodgement due to excessive flexural and tensile stresses induced by ground heave and radial soil displacement. As a matter of fact, five working piles within a tank were detected with dislodged joints using High Strain Dynamic Pile Test (HSDPT) after completion of piling works. During the HSDPT testing, the piles had suffered sudden settlement of about 80 to 125mm when a 7 tonne hydraulic hammer was just placed on the piles prior to hammer impact. Subsequently, the initial blows detected major discontinuity at the first joint and showed an intact length of only 12m. The first joint discontinuity was closed up after few hammer blows and another major discontinuity at the second pile joint was detected. Subsequent driving had made the second joint discontinuity to disappear but velocity reflections were still clearly observable at the first and second joints. Total cumulative pile top settlement after the restriking was recorded between 200 and 500mm. CAPWAP analysis indicated that all tested piles had achieved the designated ultimate static capacity based on the final blow after all pile joints closed-up. Although the condition of the pile joints was not visually inspected, it was suspected that the pile joints were dislodged due to the flexural and/or tensile stress induced by the ground heave and soil displacement.

To overcome the above construction problems, a series of mitigation measures were suggested: symmetrical pile installation sequence from tank centre outwards, use of open-ended prestressed spun pile to allow forming of soil plug in the inner annulus resulting in less soil displacement and better flexural resistance comparing to commercial square pile, and re-driving of earlier installed piles to minimise differential settlement, especially those piles supporting the tank shell plates. With the implementation of the suggested measures, lesser construction problems were reported.

7. CONCLUSIONS

The following conclusions can be drawn for this project:

a. From the instrumentation results, the mobilised pile shaft resistance is larger than the typical correlated factor of 2 to 3 times the SPT'N in the pile design formula by Meyerhof (1976). Back-calculation shows the correlated ($f_{s,mob}$/N) factors in the range of 4 to 33.
b. During the hydrotest, the measured settlements around the tank circumference vary in a range of 0 to 28mm. Such variation of settlement could be due to the inherent uncertainty of pile support stiffness in reality.
c. Installing large displacement pile group at close spacing in soft marine clay area would inevitably cause ground surface heave and radial soil displacement, and subsequently resulting in excessive tensile and flexural stresses to the adjacent piles and potentially dislodge the pile joints. Further research and engineering assessment methodology are required to determine the significance and effect of the current practices of piling construction.

8. REFERENCES

[1] Meyerhof, G.G. (1976). "Bearing Capacity and Settlement of Pile Foundation,". Journal of the Geotechnical Eng. Div., ASCE, Vol. 102, No. GT3, pp. 195~227.