Behavior of Single Pile and Pile Group Foundation for High Rise Buildings on Expansive Soils

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ABSTRACT: High rise buildings supported by piles are now increasingly constructed in Indonesia especially in West Java (at Cikarang and Cibitung area) and West Surabaya. The cases in West Surabaya is interesting because the depth of the expansive clays is more than 80 m. The characteristics of expansive clays are very specific due to its capacity of swelling and becoming soft upon absorbing water from their current unsaturated conditions, and yet as clay material, most designers worry about the long term compression. This paper discusses the results of observation of the single pile and group piles from design, construction and performance in short term and long term settlement of a complex of high rise buildings with three towers of 51 stories. The results of soil laboratory tests were disturbed except for water content and index properties, hence the design has rely on the results of insitu testing including SPT, CPT and Pressuremeter Test (PMT). The estimated settelement of pile groups of 14-15 cm were in fact much less (only 30%) and the short term settlement dominate almost 90% of the total settlement. This fact may be related to the swelling characteristics and unsaturated soil condition
Keywords : pile foundation, expansive soils, pile group, settlement

1. INTRODUCTION

Behavior of single pile and pile groups to support high rise buildings is always of interest to the engineers, and more specific is because the buildings are founded on expansive soils. The main objective of this paper is to review the behavior of a single pile and pile group under three towers. A case study is presented to illustrate the actual behavior of single pile and measured settlement of group piles. This project is a mixed used building that include Apartment, Hotel, Office and Mall in west Surabaya with 51 stories of Tower and 7 layers of Podium. Three towers and the podiums were connected to each other as shown on figure 1, i.e. Tower Anderson, Tower Benson and tower Lavis (tower 7) which was constructed the last. The authors have been involved in the design and following construction and performance of the foundation construction as well as the subsequent settlement

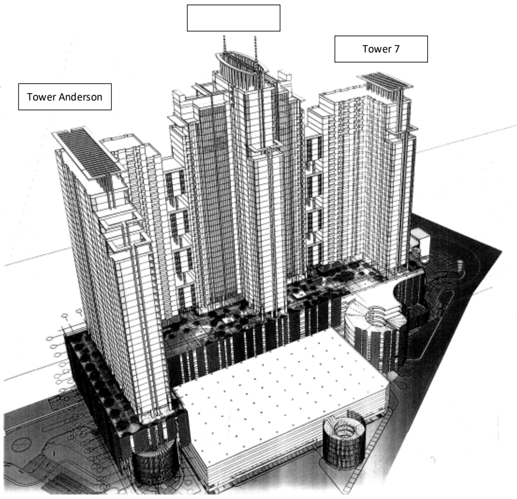

Figure 1 : Description of the project site

2. GEOLOGICAL AND SOIL CONDITION

2.1 Geological Condition

According to Surabaya Geological Map, Location of this study is a part of Lajur Kendeng and occupy Lidah Formation (Tpl) which consist of blue clay stone, black spot, loamy, solid, and harden when dry and soften and swell when exposed to water.

Figure 1 shows project location in the geological map. The sediment rock in the study location is from Pliosen age and Tersiary period that rich in volcanic material. Boring results in this area obtained brownish grey clay, which is welknown as expansive material.

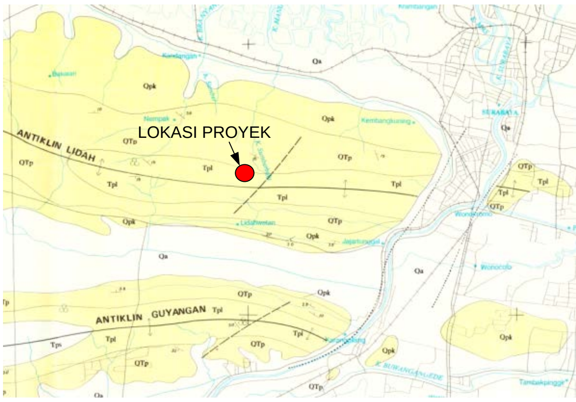

Figure 2 Geological Condition (Badan Geologi Indonesia)

2.2. Soil Condition

Soil investigation was conducted by PT. Data Persada at the project site in September 2015 consisting of 3 boreholes @ 50 m (BH-02, BH-04 dan BH-05), 3 boreholes @60 m (BH-01, BH-03 dan BH-06) with N_{SPT} values at interval 2 m and undisturbed samples for laboratory tests. Pressuremeter tests were also conducted to investigate the at rest soil pressure and stress strain behavior in cylindrical expansion. Description of soil stratification and engineering properties are described in this paper

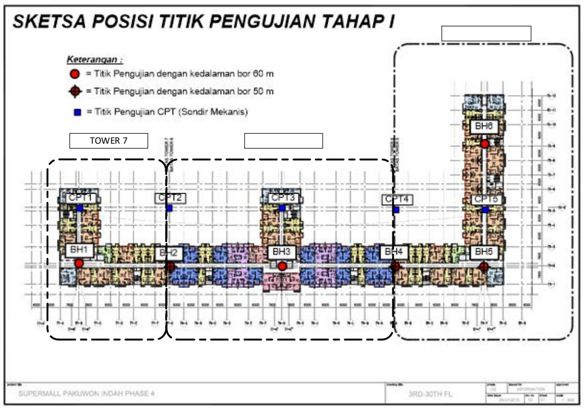

Figure 3 Location of Soil Investigation (phase 1)

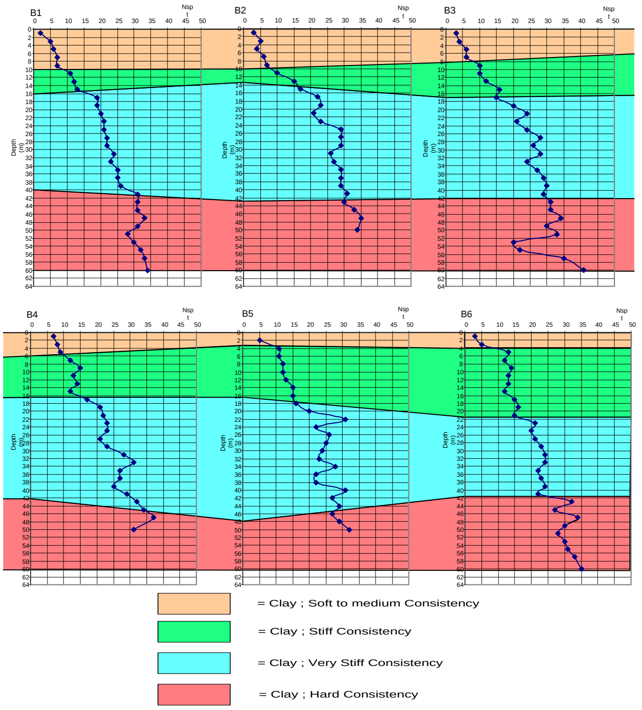

Figure 4 Soil Stratification from 6 boreholes

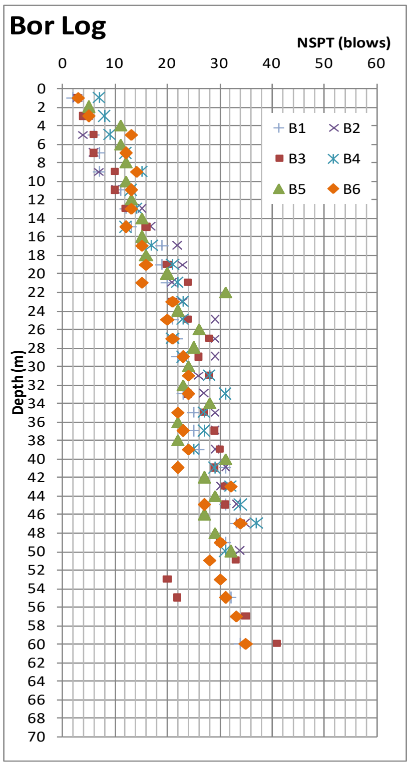

Figure 5 Profiles of N-SPT values of soil

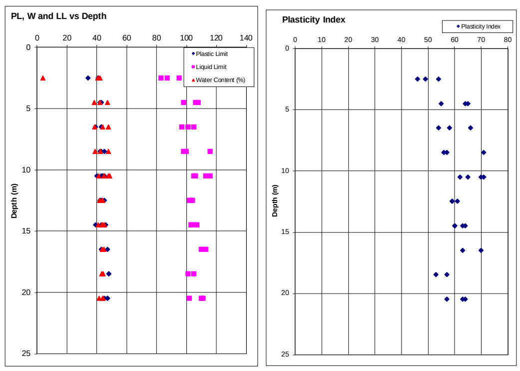

Figure 6 : Water content and Index Properties

Figure 4 shows soil stratification from 6 boreholes; in general the strata may be described as follows :

- The surface layer or first layer is soft to medium clay of 10-12 m thickness with N_{SPT} varies from 2 – 10. This layer may be thin sediment of part of the active zone of the expansive clays that has been influenced by water infiltration causing the clay to softens.
- The second layer may be described as stiff clay found at depth 10-22 m and increasing values of N_{SPT} 13-20
- The third layer is very stiff clay found at depth 20-50 m and increasing values of N_{SPT} 20-30. This layer was the bearing layer of the foundation where the tip was designed at 38-42 m depth
- The fourth layer may be described as hard clay found at depth 50-60 m and increasing values of $N_{SPT} > 30$m.

The distribution and N_{SPT} profile may be shown on figure 5 and we can see that the strength is consistently increasing with depth. Water table is not detected through the whole layer, and if water is found in the borehole, they are trap water or perch water table. This condition is favorable for the case of long term or consolidation settlement since practically no significant pore pressure will be developed.

2.3. Results of Laboratory Test

Laboratory tests conducted for the project include Index Properties, strength by Triaxial UU & CU, consolidation test and swell test. The data show that water content is found around 42% which is closer to plastic limits and even in some depth less that the plastic limit, this and that tells why the soil consistency has fall in the stiff to hard clays.

All data show that the soil is highly plastic clays or clayey silts having liquid limits of 80-130% and plasticity index of 45-85%. Those are very high values compared to general soil (figure 7)

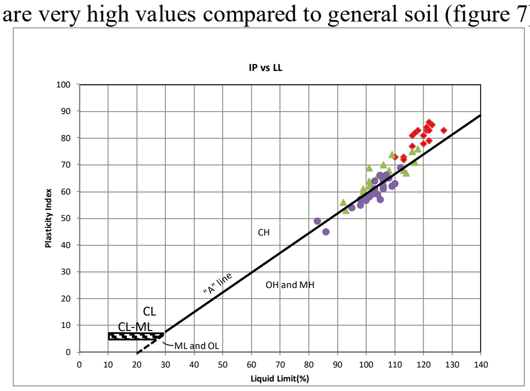

Figure 7 : Plasticity of the soils

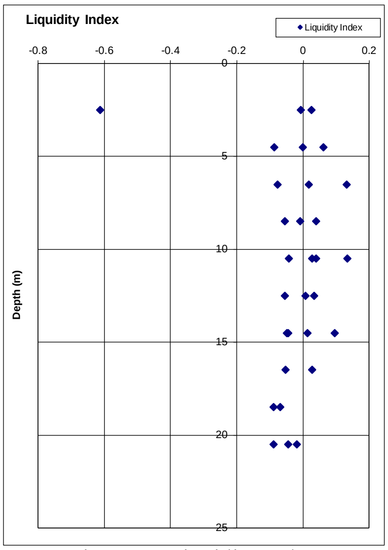

Figure 8 : Liquidity Index

Liquidity index of the soils are mostly very low and in some cases are less than 0. The yield stress ratio or overconsolidation ratio will be very high and those, this phenomena will normally reduce the settlement.

Swelling characteristics of clay

The swelling characteristics of clay is measured with oedometer, where load is applied to the sampel up to the calculated overburden pressure, and then swelling is allowed by addition of water. Under the effective overbudern pressure the soil still capable of exceeding the overburden pressure such as shown on figure 9. This swelling characteristic can bring up low rise building less than five stories and the swelling characteristic can result in reducing the settlement of high rise building. This aspect is seldom measured or investigated but in reality many buildings have been rise up including a four storey buildings nearby

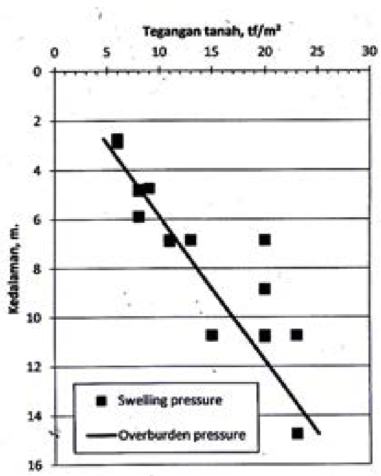

Figure 9 : sweling pressure charareristics

2.4. Pressuremeter Test Results

Problems with Expansive soils are due to the swelling and shrinking characteristics. Once the sampel was retrived to the laboratory, there will be changes in water content or volume of the soils. Furthermore if triaxial CU are conducted, the saturation takes very long time and inclusion of water into the soil may change its behavior, the strength and stiffness drops significantly. To overcome this problem, insitu tests such as SPT, CPT and Pressuremeter Tests are carried out. In practice this insitu tests have been the more reliable data for design and deriving parameters for analysis. The foundation design has been based on SPT values and for the stress history and soil stiffness, the data from pressurementers have been used.

Due to uninspected poor data on strength and stiffness of laboratory tests, limited pressuremeter tests were conducted at 10m, 20m, 35m, 45m in BH-2 and at 12m, 20m, 35m, 45m in BH-4.
Pressuremeter Test (PMT) is the best geotechnical data developed by expansion of cylindrical cavity at insitu soil condition whch gained more popularity due to the fact that the tests are carried out under insitu stress condition, water content, producing insitu stress strain behavior at elastic and plastic condition, fast and economical, also direct use for design and may be done continuously. Parameters obtain by pressuremeter test include

1. Po = ground pressure at rest (kg/cm^2)
2. Py = yield pressure (kg/cm^2)
3. Em = elastic modulus (kg/cm^2)
4. Gm = shear modulus (kg/cm^2)
5. Cu = undrained shear strength of ground materials (kg/cm^2) (after Gibson & Anderson, 1961)

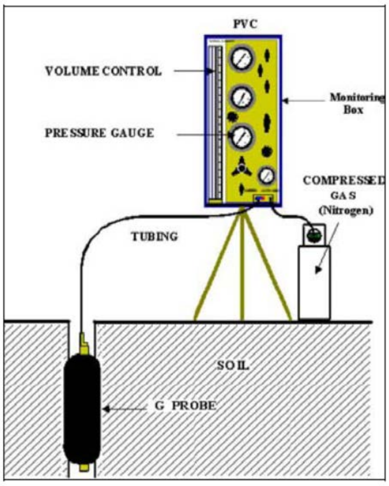

Figure 10 : Pressuremeter Test used for the project

Tabel 1 : Summary Pressuremeter data obtained at BH-2 and BH-4

Borehole No.	Depth (m)	P_0 (kg/cm^2)	P_y (kg/cm^2)	P_1 (kg/cm^2)	km (kg/cm^3)	Em (kg/cm^2)	rm (cm)	SPT value
BH-2	10	0.58	7.27	8.4	63.5	354.66	3.72	7
	20	1.02	10.64	15.26	58.82	310.49	3.52	23
	35	2.29	18.98	26.02	42.01	238.14	3.78	29
	45	2.52	13.6	25.82	68.53	364.27	3.54	33
BH-4	12	0.56	6.66	12.15	10.97	61.28	3.72	13
	20	1.84	6.43	10.83	17.54	109.81	4.17	21
	35	2.66	15.8	25.8	38.23	204.51	3.57	27
	45	3.77	12.73	24.78	57.73	327.36	3.78	34

Although only limited data obtained from PMT, the data has been spread at different depth (figure 11) and may be used to estimate the variation of the test results along the depth.

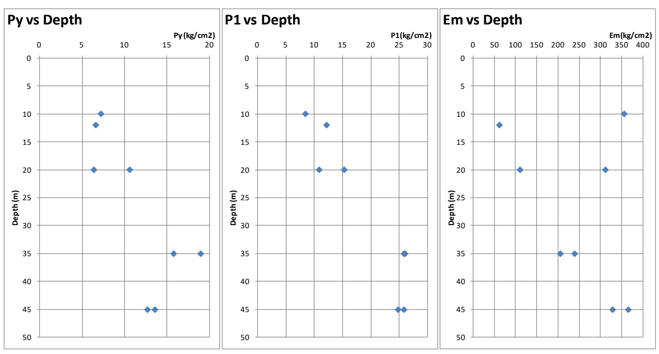

Figure 11 : Variation of PMT parameters with depth

The SPT and Pressuremeter modulus corelations has been well developed such as shown on figure 12. The pressuremeter modulus can be used for long term settlement since practically the modulus is similar to drained modulus tested by triaxial test (Briaud, 1996 and Roger Frank, 2013).

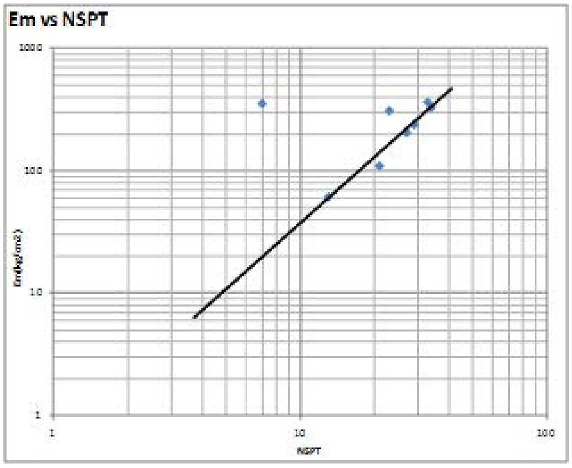

Fig 12 Correlation of pressuremeter moduli (Em) and N

3. THE DESIGN OF PILE FOUNDATION

Based on the soil condition (medium to very stiff clay) and economical consideration, pile foundation has been selected to carry the structure loads. However since the buildings are 51 storeys, the authors have considered that the foundation shall be deep enough to avoid excessive settlement. The main foundation system is spun precast pile with 600 mm and 500 mm diameters. The installation method is by used of push-in/jack-in pile. This jack-in method is to avoid noise and vibration disturbance to the surrounding neighborhood with additional benefit that we had known injection force which is chosen as high as 250% of the work load or allowable bearing capacity.

However, the development is very close to the surrounding neighbour buildings, hence on the pheriphery or at the boundary, the foundation selected is bored piles of ϕ800 mm diameter. In order to balance between the driven pile and the boredpiles, both foundation system has been design to carry the same axial stiffness at their allowable load. The allowable load for driven piles is 200 tons and based on the bearing capacity analysis the required length of the driven spun pile is 34-37 m. While for boredpile ϕ800 mm the same axial stiffness is 35 m with allowable capacity of 300 ton. In some area higher capacity of 400 tons for bored pile is also required and calculated to be 43 m length

For cohesive soils, the general formula use for tip resistance is:

$$Qp = q \cdot Ap$$

where : q $= \left(\frac{D/B}{4}\right) 9 c_u$ for D/B < 4

q $= 9 c_u$ jika D/B ≥ 4

A_p = cross section area of pile

The friction resistance is calculated using adhesion factor as suggested by Kulhawi and Jackson (1989)

$$Qs = \alpha \cdot Su \cdot p \cdot L$$

Where α = adhesion factor.

Su = undrained shear sttrength

p = pheriphery of piles

L = length of embedded pile

The adhesion factor α have used the recommendation of Kulhawy dan Jackson (1989).

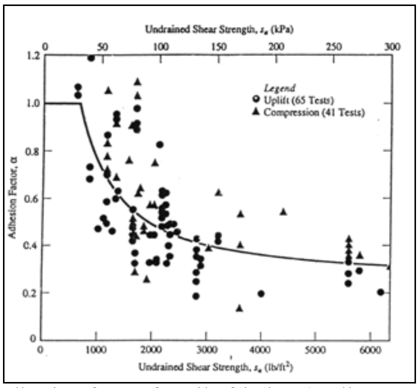

Figure 13 : Adhesion factor for pile friction (Kulhawy and Jackson, 1989)

For bored pile, similar method is done with an average adhesion factor for friction as much as 0.55 (Reese and Wright, 1979)

The results are shown on figure 14

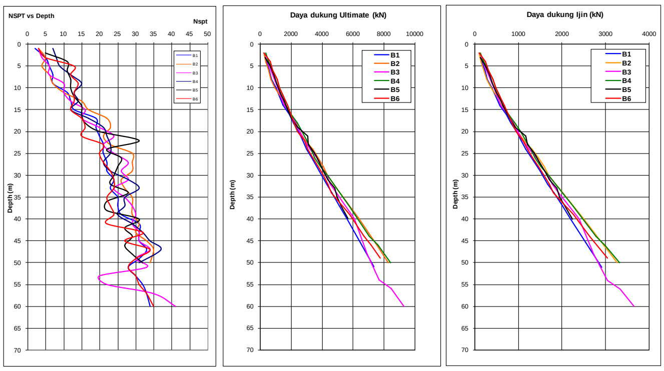

Figure 14 : Calculation for ultimate and allowable capacity of 600 mm diameter of spun pile

4. CONSTRUCTION OF PILE FOUNDATION

Figure 15 shows the installation of pile foundation using push in/jack in method where the pressure reach 250% of the design load and held 3 times 10 second at end of installation. This method has proven to be reliable.

The problems of jack in method (as well as driven hammer) in expansive soils are heaving of the neighbour and the problem of heave of the piles. The second problem was overcome by redriving, but we have to make sure that all length of piles can be exactly pushed into the soils because the movement of the machine requires that no portion of the pile shall be on ground level.

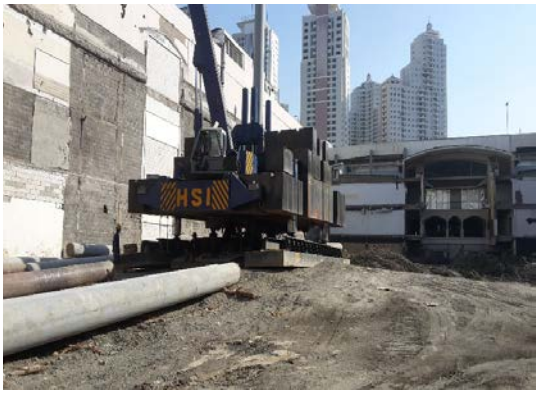

Figure 15 : Pile installation method

Figure 16 : Preparation for Pile Injection

Figure 17 : Installation of Boredpile

Figure 18 : Use of casing for bored pile

Figure 19 : Installation of rebar for bored pile

Figure 20 : Concreting of boredpile with tremie

6. BEHAVIOR OF SINGLE PILE UNDER LOAD

The behavior of single pile is well predicted by the results of pile load test. However this behavior only represent short term condition of the piles and the effect of the settlement is only on a small scale. The real condition has wider area and the impact will also be different. However to a certain degree the single pile behavior may be reflected in the group behavior

The folowing figure is typical results of pile load test in expansive soils. For driven piles, the results may not be consistent due to the condition during driving (such as heave or water penetration into the gap between the soils and the piles). Three different results are presented in figure 21. However for boredpiles, the results of pile load test are more consistent (figure 22).

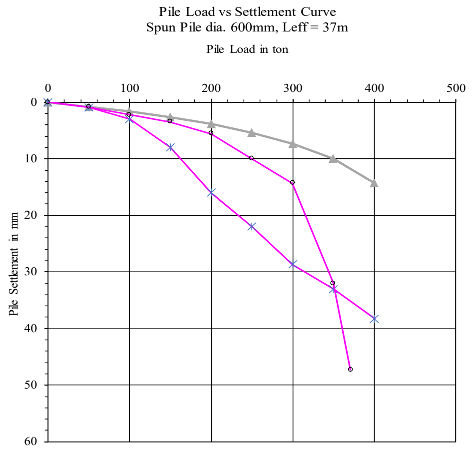

Figure 21 : Typical results of pile load test for driven piles

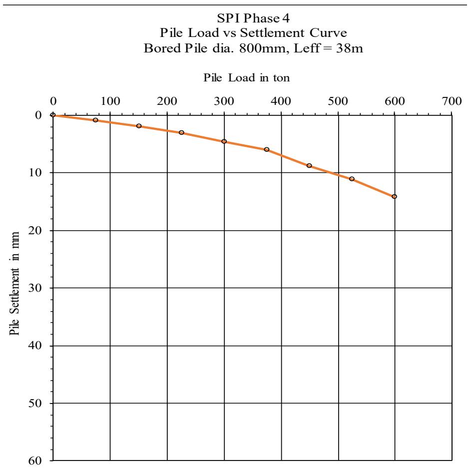

Figure 22 : Typical pile test results for boredpile in expansive soils

7. SETTLEMENT ANALYSIS AND SETTLEMENT MEASUREMENT DURING CONSTRUCTION

Settlement Analysis

Settlement analysis for this project have used two methods, the first one is based on Poulos and Davis (1980) and the second analysis is by Finite Element Method (computer software GTS Midas). The method suggested by Poulos is based on the interaction between piles and computed as follows :

$$Si = x_i \cdot \sum \left(P_j \cdot \alpha_{ij}\right) + x_1 \cdot P_i$$

where :

Si= settlement of pile i due to own load and other piles
xi= settlement of single pile due to unit load (mm/ton)
Pi= load at pile no -i (ton)
Pj = load on pile no-j (ton)
αij= interaction factor between piles

Interaction factor for friction pile and tip bearing piles are different. Poulos derived curves for these interaction by assuming Poisson ratio equal to 0.5. These interaction factors are for particular piles with different length/diameter. Poulos dan Mattes (1971) stated that αF are function of s/D, L/D, and K, where K is the ratio of the pile modulus and elastic modulus of the soils. The results of Poulos and Davis method is shown as settlement of each pile and contour of settlement was plotted on figure 23

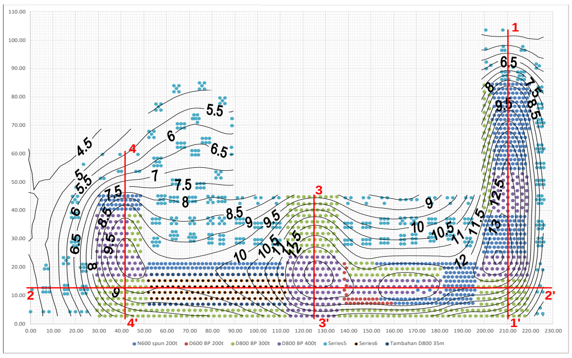

Figure 23 : Results of Settlement Calculation by Poulos and Davis (1980) method

The settlement profiles from Poulos and Davis Method are shown by long section 2-2 and cross section 1-1, 3-3 and 4-4 as illustrated in figure 24 – 27. Based on the cross section and longitudinal section of the settlement profile, the slope may be presented. The differential settlement must be limited to 1/300, and all the results of calculation comply with this requirement.

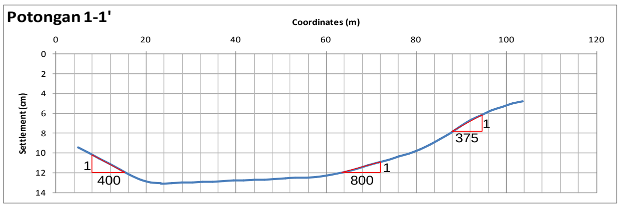

Figure 24 Settlement profiles cross section 1-1

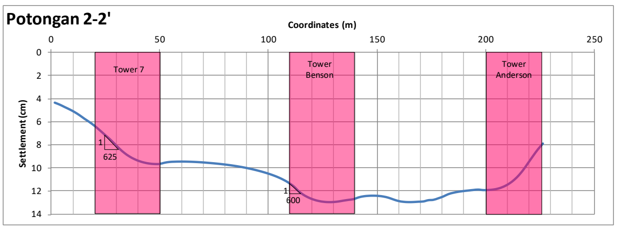

Figure 25 Settlement profiles long section 2-2

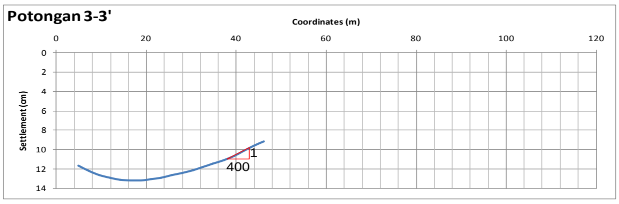

Figure 26 Settlement profiles cross section 3-3

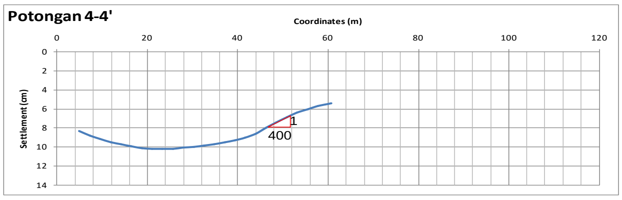

Figure 27 Settlement profiles cross section 4-4

The settlement analysis is also carried out by finite element modelling, where the piles and raft foundation are modeled and soil structure interaction are also introduced.

The model is shown on figure 28, where the towers are supported by the pile and raft foundation and the podium is supported by pile caps. The 3D model allow the interaction between pile, pile caps, raft and also the soil. The difference with Poulos and Davis Method is the load carried out by pile cap and raft are not considered. If one wants to exclude the raft caryiing load, then the upper soil layer may be soften to reduce the portion of load to the raft. Figure 29 shows distribution of settlement under the structures and the induced settlement to the surrounding. Maximum settlement of the system is about 15 cm.

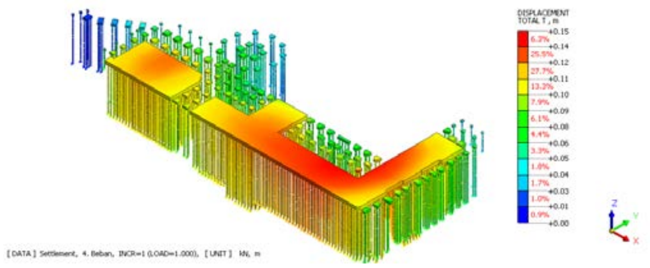

Figure 28 : Model of pile and raft on this project

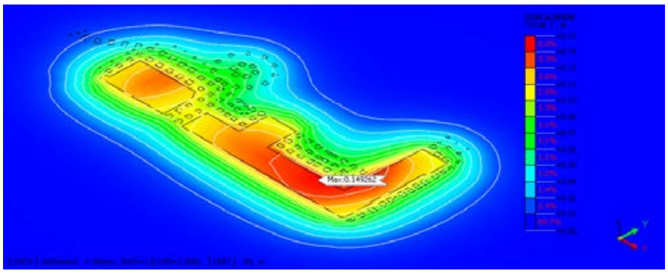

Figure 29 : Countour of settlement of the structure and the surrounding

The settlement profile can then be evaluated by looking at cross of settlement profiles as shown on figure 30. It is shown that the results of finite element analysis are in line with the results of approach using Poulos and Davis method.

The effect of the adjacent towers to the settlement is also shown on the plot, and differential settlement are more pronounced at the location of the tower and the podium

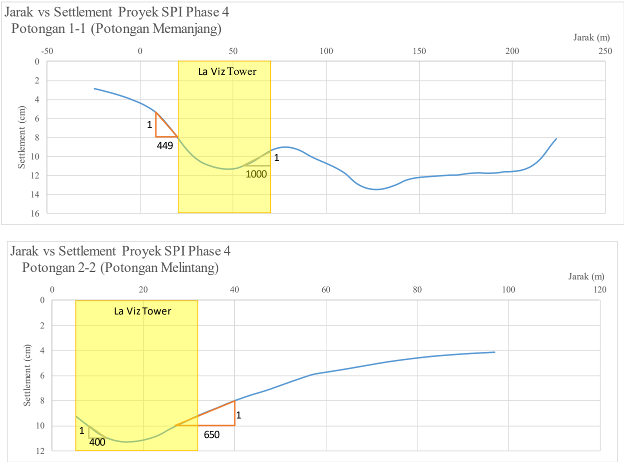

Figure 30 : Settlement profiles in Long Section and Cross Section

After all piles are installed, and pile cap completed, a number of points are decided as points for settlement measurement. The settlements were measured every week and the load of the structures are indicated by the number of storeys constructed. The results are plotted from time to time and the following figures are the settlement of the structures.

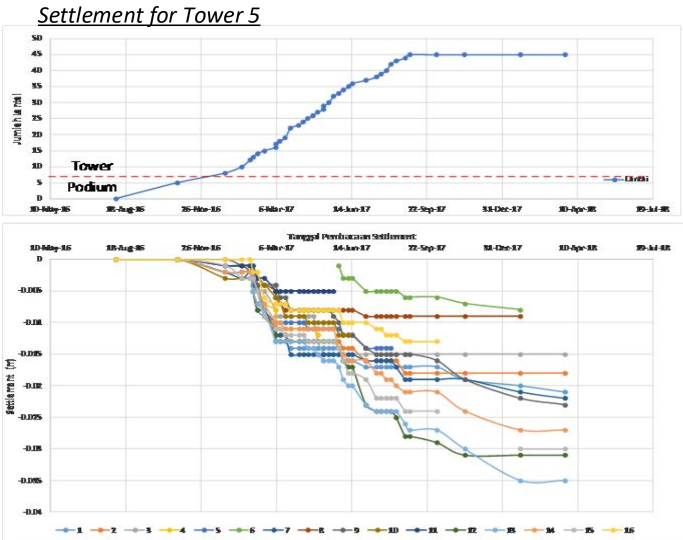

Figure 31 : Settlement of tower 5 (Benson)

This is very interesting topic since the settlement measurement give much less settlement compared to the results of analysis by a factor of more than 3. The short term settlement dominate the total settlement by about 60-70% and long term settlement take very short time in less than 7 months.

8. CONCLUSION SUMMARY

The study of the behavior of piles in expansive soils show that the expansive characteristics influence the behavior of single pile and group piles, in terms of there may be inconsistency on the single pile behavior due to the construction histories. The predicted settlement is much less than calculated which may be due to the heave and the unsaturated soil condition. In general the results of insitu testings are more reliable for design and analysis.

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