BEARING CAPACITY CHARACTERISTICS OF A CAST IN PLACE CONCRETE PILE IN BANGKOK －FIELD DATA AND NUMERICAL SIMULATION－

バンコクで施工した場所打ちコンクリート杭の支持力特性 －押込み試験と 3 次元 FEM 解析－

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バンコクにおいてアースドリル工法による場所打ちコンクリート杭を施工し，杭の押込み試験を実施してその支持力特性について検討した。その結果，試験時の最大沈下量は許容値未満であり，支持力の 99%を周面摩擦力が負担していることが明らかとなり押込み試験は支持力を評価するための有効な手段であることが確認された。押込み試験結果を再現するために，降伏基準に Mohr-Coulomb 式を用いて 3 次元弾塑性 FEM 解析を実施した。解析結果では支持力に対する周面摩擦力の負担率が 95%と実験結果よりも小さくなった。これは，地盤を弾塑性としてモデル化したことに要因があるものと推察された。
キーワード：杭，押込み試験，周面摩擦力，支持力，FEM 解析

Bearing capacity characteristics of a cast-in-place concrete pile in Bangkok were studied by conducting an instrumented static load test. It was found that 99% of the allowable bearing capacity of the pile was taken by the skin friction and the settlement of the pile met with the standards. It was understood that the instrumented static load test as an effective method to verify the bearing capacity. In the second part of this study, 3D finite element (FEM) analysis was conducted to simulate the load test by modeling Bangkok soil using Mohr-Coulomb (linear elasto-plastic) model. From the analysis results, it was found that only 95% of the allowable bearing capacity was taken by the skin friction and that value was smaller than the test results. In addition to that, the settlement of the pile was higher than the test results. It was understood that it is difficult to simulate the behavior of Bangkok soils only by using a linear elasto-plastic model.
Key Words: Pile, Instrumented static load test, Skin friction, Bearing capacity, FEM analysis

1. INTRODUCTION

In Bangkok, pile foundation with circular or barrette piles is mainly adopted for medium and high-rise buildings because of the thick soft clay layer which is deposited near the ground surface[1]. The main purpose is to reduce the settlement induced by the soft clay layer which has high compressibility and low shear strength. When considering the bearing capacity characteristic of a bored pile in Bangkok, they are designed as friction piles since there is no bedrock even at 80 m depth. To achieve high skin friction, deep large diameter bored piles with tips extended to 40 to 60m are used in the case of high-rise buildings[2]. On the other hand, in Japan belled piles are widely used and designed as end-bearing piles where the skin friction is hardly accounted as most of them are seated under bedrock[3]. Similarly, a clear difference in the construction and testing methodologies of pile foundations also was observed between Japan and Thailand and there are very limited reports on this topic.

In order to understand the bearing capacity characteristics of a friction pile in Bangkok soil, a circular, cast-in-place concrete pile (hereafter referred to as pile) was constructed by the wet-process construction method and instrumented static load test was conducted. The similarity and differences in the construction and testing methodologies between two countries were discussed in detail. To simulate the field test results, numerical analysis was carried out by using a Japanese FEM software called SoilPlus (ITOCHU Techno-Solutions, ver. 2019 (1)). In this paper, the obtained FEM results were discussed and compared with the field test results.

2. SUBSOIL CONDITION

Bangkok is located on the lower Chao Phraya plain, which is made of a very thick deposit of marine and alluvial soils. Generally, Bangkok subsoils can be introduced into two parts. The upper part is Bangkok soft to medium clay and the lower parts are alternative layers of hard clay and dense sand. **Fig. 1** shows the soil profile at the site. Stiff lean clay crust of 3m is found as the top layer. Then Bangkok clay layer with undrained shear strength (S_u) less than 40kPa is deposited up to GL-3～-14m depth. The layer deeper than that is called the 1st hard clay layer. It has N value of 20-40 and S_u value of 150-200kPa. Then a sandy clay layer with N value of 20-50 is sandwiched under it, and then a sand layer with N value of 50 or more is deposited up to GL-32～-38m. This sand layer is called the 1st sand layer and it is the support layer for low-rise buildings. After this, the 2nd hard clay layer with a N value of about 50 and S_u>200kPa is deposited at GL -38～-45m. From GL -45m to -57m, the 2nd sand layer with N value of 50 or more is deposited and this is used as a support layer for the high-rise buildings taller than 30 stories[1)]. The pore water pressure condition in this site is hydrostatic from the ground surface. **Table 1** shows soil properties at the position of the test pile. In here, cohesion was obtained from the test results of the unconfined compression test (UC) and the pocket penetration test (PP). Direct shear tests were conducted to evaluate the friction angle (φ) of sand layers.

3. ESTIMATION OF PILE BEARING CAPACITY

The bearing capacity of a single bored pile is calculated using static formula as shown in equation 1.

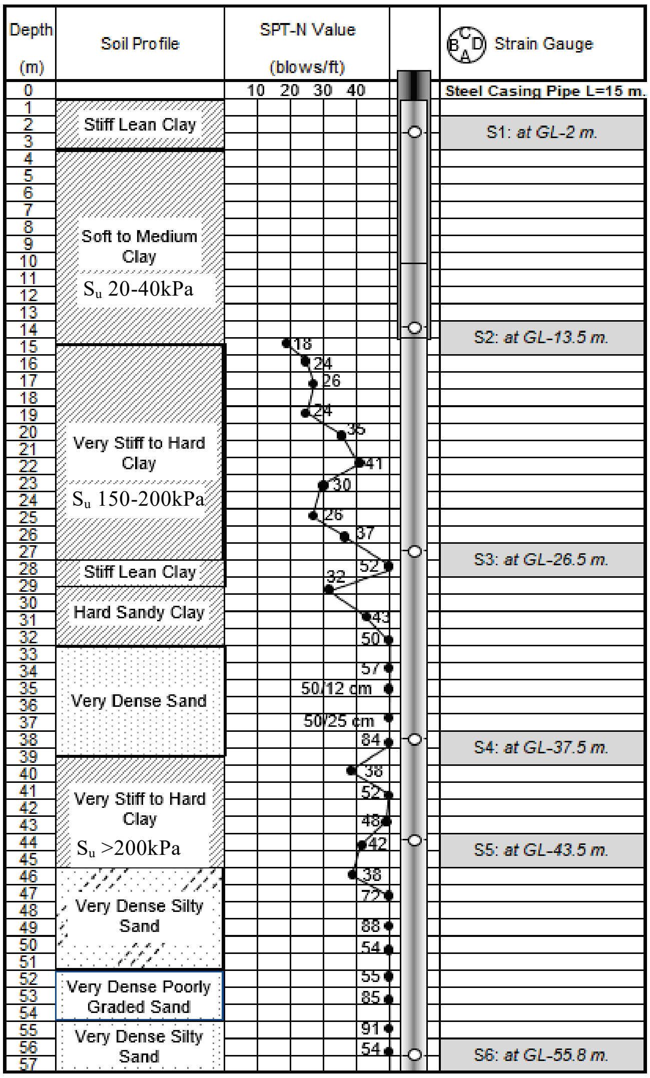


Fig. 1. Soil profile at the site and the location of strain gauges

Table 1. Soil properties

Soil	Depth (m)	Unit weight (kN/m^3)	Poisson ratio	Elastic modulus, E,E[1] (kPa)	Cohesion (kPa)	Friction angle (ϕ)
Stiff lean clay	0-3	18.6	0.45	14250	56	-
Soft to medium clay	3-14	16.7	0.45	13800	28	-
Very stiff to hard clay	14-27	19.6	0.45	206900	180	-
Stiff lean clay	27-29	19.6	0.45	350000	216	-
Hard clay	29-32	19.6	0.45	350000	216	-
Very dense sand	32-38	19.6	0.3	131600	-	37
Very stiff to Hard Clay	38-45	19.6	0.45	380000	244	-
Very dense Silty Sand	45-51	19.6	0.3	132400	-	44
Very dense poorly graded sand	51-54	19.6	0.3	132400	-	44
Very dense Silty sand	54-57	19.6	0.3	132400	-	44

$$Q_a = \frac{(Q_s + Q_b)}{FS} - W_p - NF \tag{1}$$

where,

Q_a : Allowable load capacity

Q_s : Ultimate skin friction

Q_b : Ultimate end bearing capacity

W_p : Weight of pile

NF : Negative skin friction

FS : Factor of safety (2.5 is recommended)

End bearing capacity is

$$Q_b = A_P\,(cN_c + q^1N_q\ ) \tag{2}$$

where A_P is the area of the pile tip, c is the cohesion of soil which is supporting to pile tip, q^1 is the effective vertical stress at the level of the pile tip, and N_c, N_q are the bearing capacity factors.

Skin friction is

$$Q_s = U \sum L_i f_s \tag{3}$$

where U is the perimeter of the pile, L_i is the thickness of the soil layer, and f_s is unit skin friction at relevant depth.

Unit skin friction for clayey layers is evaluated using an empirical formula, where α is the adhesion factor and S_u is the undrained shear strength. Unit skin friction for sandy layers is calculated using the effective stress approach, where σ_v^1 is the effective overburden pressure and β (K_s.tanδ) is the shaft friction factor. K_s is the coefficient of horizontal earth pressure and δ is the coefficient derived from the internal friction angle (φ) of the sand layer.

$$f_s = \alpha . S_u \quad for\ clayey\ soil \tag{4}$$

$$f_s = \beta . \sigma_v^1 \quad for\ sandy\ soil \tag{5}$$

The diameter and the length of the test pile which is reported in this paper were 1.2m and 57m, respectively. From the design, the allowable bearing capacity (Q_a) was obtained as 7848kN (800tf). In here, 85% of Q_a was obtained from skin friction and the remaining 15% was from the end bearing.

4. INSTRUMENTED STATIC LOAD TEST

4.1 CONSTRUCTION METHODOLOGY OF THE TEST PILE

After setting up the location for the pile a temporary

Fig. 2. Drilling of pile using polymer

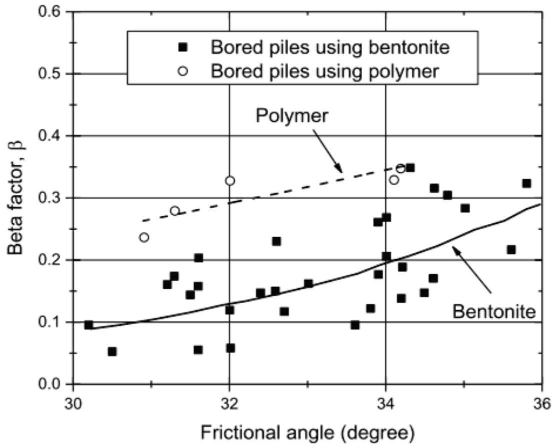


Fig. 3. Relationship between β factor and frictional angle of Bangkok sand [1)]

Fig. 4. Setting for static load test

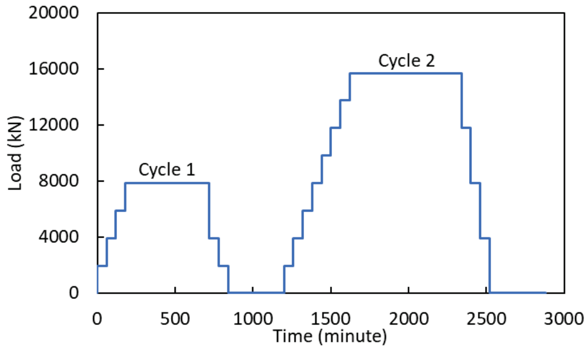


Fig. 5. Time history of loading and unloading

casing with a length of 15m was installed to maintain the hole stability in the upper soft soil layer. The hole was drilled using an auger and continued using the bucket method.

A drilling slurry was used to prevent the borehole instability as the pile is deep-seated. In Japan, bentonite-based and polymer-based drilling slurries are used depending on the soil quality and the groundwater conditions[4]. On the other hand, in Bangkok, a drilling slurry containing only polymer is widely used after the 2000s[5]. **Fig. 2** shows the drilling method of the pile using polymer. The polymer used here is polyacrylamide. The quality control values for the stabilizers were density < 1.02g/ml, viscosity of 40-55 seconds (ASTM D6910/D6910M) and pH of 8-10, respectively. In Thailand, it has been reported that it is possible to achieve higher skin friction and end bearing in sandy layers by using polymer-based drilling slurry than by pure bentonite slurry due to the reduction in thickness of filter cake along the pile shaft and the ease of pile base cleaning[3]. **Fig. 3** shows the relationship between the shaft friction factor β, and the internal friction angle φ of the sand layer. It can be seen that β is larger when a polymer is used than when bentonite is used[1].

After finishing the boring and the cleaning of the hole, Koden test was conducted to verify the verticality. Then the reinforcement cage was installed. In order to calculate the axial force distribution along with the pile depth, strain gauges were attached to the reinforcement bar in several locations to measure the axial strain in each depth. There were 6 measurement positions from S1 to S6 (GL-2m, GL-13.5m, GL-26.5m, GL-37.5m, GL-43.5m, and GL-55.8m) as shown in **Fig. 1**. Strain gauges were installed at 24 locations in 4 directions at each depth. Finally, the concreting was done by using the tremie method. The grade of the concrete was 27.45MPa (280KSC-cubical). After pouring concrete to the desired level, the temporary casing was removed.

4.2 METHODOLOGY OF THE LOAD TEST

Fig. 4 shows the setup for the instrumented static load test. The test pile was in the center and there were four reaction piles. The test was started 92 days after concreting the pile in accordance with ASTM D1143/D1143M-07 item 8.1.3 procedure B. From the design, the allowable bearing capacity (Q_a) for the test pile was obtained as 7848kN (800tf). As this pile was a working pile, the maximum test load was set as 15696kN (1600tf), in other words, two times of allowable load. The time history of the loading and unloading procedure

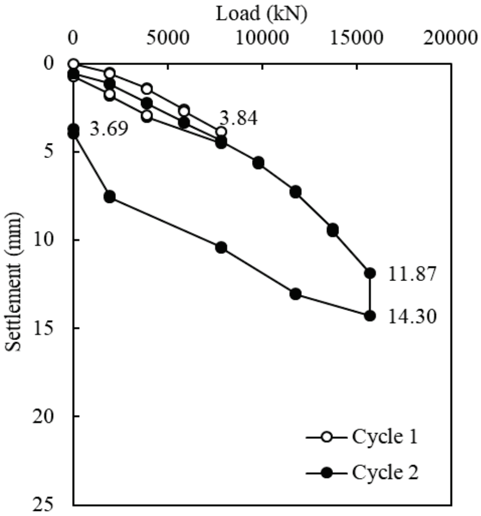


Fig. 6. Load settlement relationship from static load test

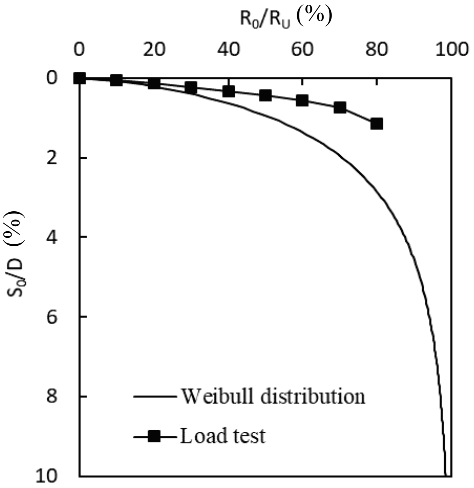


Fig. 7. relationship between R_0/R_U and S_0/D

is shown in **Fig. 5**. Loading was applied in two cycles using the step-loading method, where one step was 1962kN (200tf). The load at the final stage of cycle 1 was 7848kN (800tf), and the maximum load was given in cycle 2. In each cycle, the load was held 12 hours before starting unloading. The loading was applied using six 4905kN (500tf) hydraulic jacks. 4 dial gauges were installed on the test pile head to measure the head movement. The readings of the 4 dial gauges and the 24 strain gauges were recorded in each loading and unloading step.

4.3 TEST RESULTS

Fig. 6 shows the relationship between the pile head load (R_0) and the obtained settlement (S_0) for both cycle 1 and cycle 2. The maximum settlement was 14.3mm and that value was smaller than the allowable settlement of 25mm. In addition to that, the residual settlement was 3.69mm, and that value also smaller than the allowable value of 6mm following the Thailand standards. From these results, it was confirmed that the pile performs well following the standards. From the shape of the load settlement curve, it was found that the pile was behaved in elastic range until around 11772kN (1200tf) load which is equivalent to 1.5 times from the allowable load of the pile.

Fig. 7 shows the relationship between R_0/R_u and S_0/D for test results and the Weibull distribution. Here, R_u is the ultimate bearing capacity and D is the pile diameter. Weibull distribution was obtained from the statistical analysis on the many static load test results which were conducted on friction piles in Japan[6]. From these results, it is understood that the amount of settlement of the friction pile constructed in Bangkok is smaller than the friction piles in Japan.

From the strain gauge measurements, the force distribution along with the pile depth for each loading was obtained as shown in **Fig. 8**. To evaluate the force distribution, the elastic modulus of the pile at each strain gauge location was obtained by Fellenius method[7]. In this method, a linear relationship between the tangent modulus and the strain is considered. Every measured strain value was converted into force via its corresponding strain-dependent secant modulus. A clear linear relationship was obtained only for S2 and S3. The same parameters obtained from S3 were used to evaluate force for S4 to S6. From the test results, it was found that 7750kN (790tf) from the allowable load (Q_a) of 7848kN (800tf) was taken by the skin friction. In other words, it was 99% of Q_a and it was greater than the expected value of 6670kN (85% of Q_a) at the design stage. In addition to that, 15254kN (97%) was taken from skin friction at the maximum loading of 15696kN (1600tf).

By using the obtained force distributions, unit skin friction for each soil layer was evaluated and plotted against the displacement for each soil layer as shown in **Fig. 9**. It should be noted that the direct measurement of pile toe settlement was not obtained using an extensometer (Telltale) in this project. Therefore, negligible negative displacements were obtained due to the limitations of strain measurements from the gauges. The skin friction was considered as fully mobilized when there is no change in the unit skin friction with respect to the pile displacement or softening of the skin friction was occurred[1]. From the test results, it was found that the soil layers up to 27m depth were fully mobilized and the soil layers beyond 27m depth were not mobilized even at 2 times of Q_a load. In addition to that, the maximum unit skin

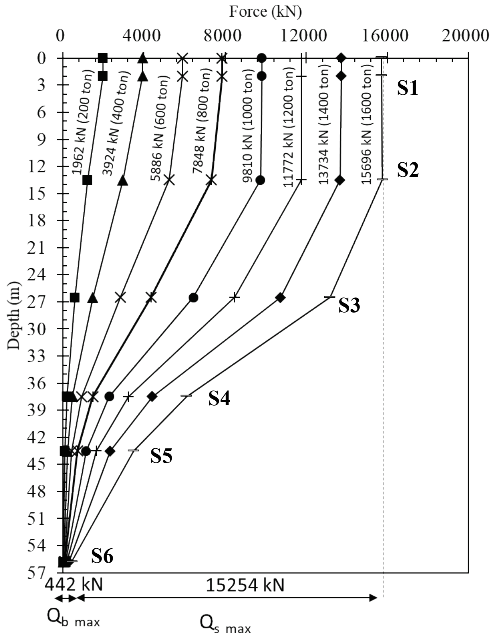


Fig. 8. Force distribution along pile depth

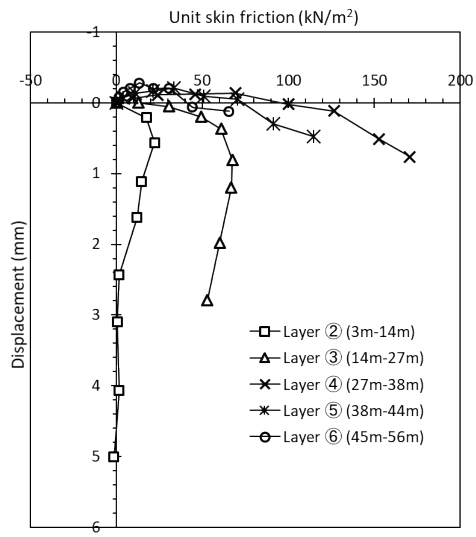


Fig. 9. Relationship between unit skin friction distribution and pile head displacement

friction which can withstand by layer GL -3～-14 m and GL -14～-27m were 20.0kN/m^2 and 73.0kN/m^2, respectively. These values matched with the values which were used at the design stage as of 24.5kN/m^2 and 73.5kN/m^2, respecively. Above results show that the bearing capacity characteristics can be clearly understood and verified using an instrumented static load test.

5. NUMERICAL SIMULATION

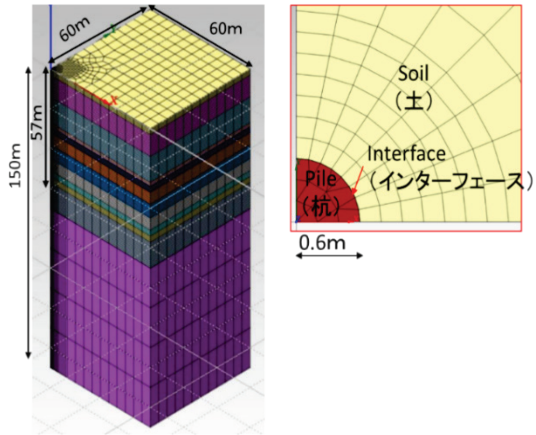
The finite element method (FEM) is reported as a powerful tool to predict the load settlement relationship in the static load test [8]. From a survey in Thailand, it is reported that 10% of the interviewees used FEM to evaluate single pile capacity. Although the instrumented static load test is very common in Bangkok, it is very rare in Japan due to its high cost. In a such case, FEM analysis can be considered as an effective solution. However, there are very limited reports regarding this kind of analysis in the literature. For filling that gap, numerical analysis on the instrumented static load test was carried out by using a Japanese FEM software called SoilPlus to simulate the field test.

5.1 NUMERICAL MODELING AND MATERIAL PROPERTIES

In this analysis, a 3D model was used as shown in **Fig. 10 (a)**. In here 1/4 of the total system was modeled. The length and the width of the model were set 60m × 60m which was approximately equal to the pile length. The height of the model was set by approximately 3 times the pile length. In addition to that, an interface between the pile and the soil was modeled to simulate the relative friction between the pile and the soil as illustrated in **Fig. 10 (b)**.

The subsoil profile in this analysis was referred to the **Fig. 1**. The pore water pressure condition was considered as hydrostatic from the top of the pile. The soil properties used in FEM analysis are summarized in **Table 1**. The elastic modulus of each soil layer was obtained from the designer where they were calculated using empirical relationships with undrained shear strength or N values as reported by several researchers[9]. All the soil layers were modeled using a linear elasto-plastic model named Mohr-Coulomb (MC).

The concrete pile was modeled as a linear elastic material. In here Young`s modulus of 36.6GPa and Poisson ratio of 0.15 were used. Young`s modulus of the pile was evaluated based on the strain gauge readings at S1 following the Japanese method[10]. In SoilPlus software, the interface can be defined as an elastoplastic joint material[11]. The normal stiffness and the shear stiffness of the interface of each soil layer were set as 100 times of its Young`s modulus and shear modulus, respectively. It was considered that there is no skin friction from the soil layers up to 14m depth by setting a cohesion value of 0.1 in the interface for accounting for the disturbance due to the uninstallation of the casing. The shear strength parameters of the interface for each layer were considered equal to the parameters of the respective soil layers beyond the 14m depth. The load was applied in 8 stages with an increment of 1962kN (200tf) in each stage until it



(a) 3D view　　　　(b) Plan view

Fig. 10. Geometry of the model

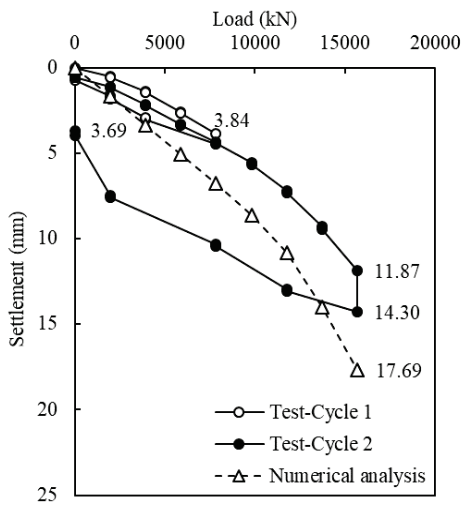


Fig. 11. Load settlement relationship from test and numerical analysis

reaches 15696kN (1600tf). At each stage, the loading was applied in 20 steps.

5.2 ANALYSIS RESULTS

From the FEM analysis, the pile head displacement toward the loading direction was obtained against each loading and plotted with test results as shown in **Fig. 11**. In here the maximum settlement was obtained as 17.69mm at 15696kN (1600tf) loading and it was larger than the obtained settlement of 14.30mm from the test. However, that value was smaller than the allowable settlement of 25mm. In addition to that, both test and FEM results showed that the pile was behaved in elastic range until around 11772kN (1200tf) load which is equivalent to 1.5 times the allowable load of the pile. When comparing the slope of the elastic ranges in both curves, a clear difference was observed. This may be due to the non-linear behavior of the soils in Bangkok.

Similar observations were reported in several papers [9), 12)] and it was suggested to use another model called Hardening soil with small strain stiffness model (HSS model) to simulate the Bangkok soft clay (0-14m) and 1st hard clay (14-32m) layers[9)]. However, all the results reported in this paper were analyzed only using the MC model.

Fig. 12 shows the obtained force distribution along with the pile depth respective to each loading for both test and FEM analysis. Compared to test results, it was found that the end bearing capacity is overestimated in FEM analysis especially at the maximum load. In addition to that, a clear underestimation of skin friction could be observed within GL -14m and GL -38m. When considering the allowable load, 99% of Q_a was taken by skin friction in the test and that was 95% of Q_a (7475kN) in the case of FEM results. Even though both test and FEM results did not perfectly match, both of those values were greater than the expected value of 6670kN (85% of Q_a) at the design stage.

The relationships between displacement and the unit skin friction obtained from the test and the FEM analysis are shown in **Fig. 13**. From the test results, it was found that the soil layers up to 27m depth were fully mobilized and the soil layers beyond 27m depth were not mobilized even at 2 times of Q_a load. On the other hand, in the results of FEM analysis, all the soil layers were fully mobilized and the maximum unit skin friction values were smaller than the test results except for the layer GL -14～-27m. The authors believe this difference is due to the simplification of soil properties for the MC model as explained in the previous section where the non-linearity of soil cannot be properly simulated.

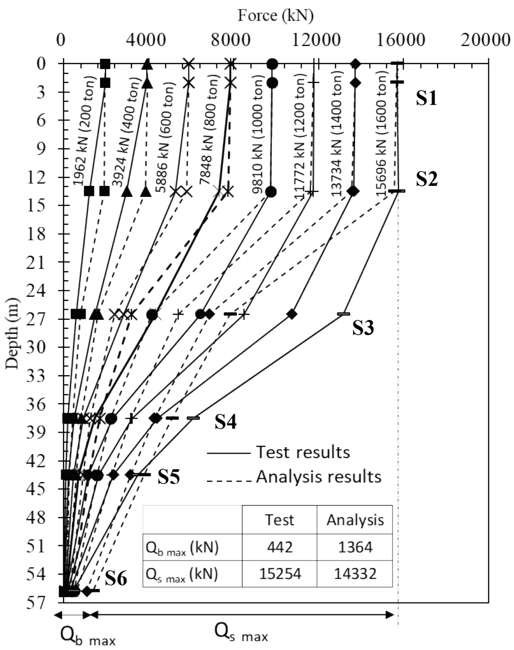


Fig. 12. Force distribution along pile depth from test and numerical analysis

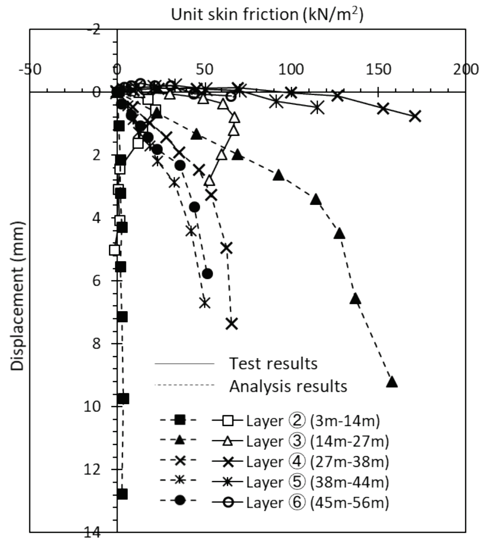


Fig. 13. Relationship between unit skin friction distribution and pile head displacement from test and numerical analysis

6. CONCLUSION

This paper presents the test results of an instrumented static load test conducted on a cast-in-place concrete pile in Bangkok and its comparison with numerical simulation. The main objective of this study was to understand the bearing capacity characteristics at site. Moreover, the similarity and the differences in design, construction and testing of pile foundations between Thailand and Japan were studied and reported. The findings can be summarized as follows.

In Thailand, cast in place concrete piles are designed as friction piles and it was reported that the use of polymer-based drilling slurry helps to increase the skin friction. Furthermore, from the instrumented static load test results, it was confirmed that the performance of the test pile is in accordance with the standard even it is a friction pile. In addition to that, 99% of the allowable bearing capacity was obtained from the skin friction. From the FEM analysis, it was understood that it is difficult to perfectly simulate the nonlinear behavior of Bangkok soils only by using Mohr-Coulomb model and it requires further study for more precise analysis with different soil models. However, the procedure of FEM analysis explained in this report may be useful in Japan to simulate the site condition instead of an expensive instrumented static load test.

The type of the pile, whether it is friction or end bearing is depending on the soil type at the site. However, different construction practices use in other countries can be used to improve the quality of the existing methodologies. The authors tried to exchange the information about the current practices in Thailand on pile foundations and believe that this report marked one more step towards globalization.

Acknowledgment: The authors gratefully acknowledge Professor Teparaksa of Chulalongkorn University for the guidance provided on the design and construction method of cast-in-place concrete piles in Bangkok.

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