Axial Load Response of Long Piles in Deep Alluvial Soil Bangkok Experience

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ABSTRACT: Based on the authors' experiences, this paper is intended to summarize the axial response of the deep-seated long piles in thick multi-layered soil of Bangkok, Thailand. Load transfer characteristics of fully instrumented barrette 1.5x3.0m in size seated about 57m below the ground level is compared with bored pile of diameter 1.5m with the same length constructed in the same subsoil condition. Effectiveness of base grouted piles are discussed in comparison with non-grouted piles. Predicted settlements of large diameter deep-seated bored piles using 3 different analytical methods are compared with measured data. Factors that influenced pile settlement behavior, such as soil type at pile base, shape of pile and drilling slurry were back-analyzed from static pile load tests of 25 fully instrumented bored piles. Pile capacity estimation using normalized load-settlement graphs constructed from back-analyzed data is proposed.

Keywords: deep-seated bored piles, barrettes, polymer-based slurry

1. INTRODUCTION

During the past four decades, owing to the acceleration of development in mega infrastructure and numerous tall building projects, design, construction techniques and practice of cast-in-place deep foundations, particularly wet-processed deep-seated large diameter bored piles and barrettes have experienced enormous development and achievement in Thailand. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of those cast-in-place deep foundations. In the initial stage, there were a number of questions on design and construction aspects of these foundation systems particularly in

Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted through out the 1990s mainly in mega projects, which provided better understanding on behavior of these deep-seated foundations.

Research focused on the design parameters and methods were produced based on these test results. The design parameters became so well established that wet-process cast-in-place foundations became regarded as reliable foundations for practitioners involved in construction industry in Thailand. This paper presents performance of deep-seated long bored piles under axial loads in thick alluvial deposit of Bangkok.

2. SUBSOIL AND EXISTING PIEZOMETRIC PROFILE

Geologically, Bangkok subsoil consisted of alternating clay and sand layers of thick Quaternary deposits extending down to about 550m depth where bedrock generally exists (Balasubramanium, 1991). Soft to very soft, highly compressible dark gray marine clay lies beneath weathered crust layers of 2m thick. Depending on the location, this layer is extended up 12m to 18m.

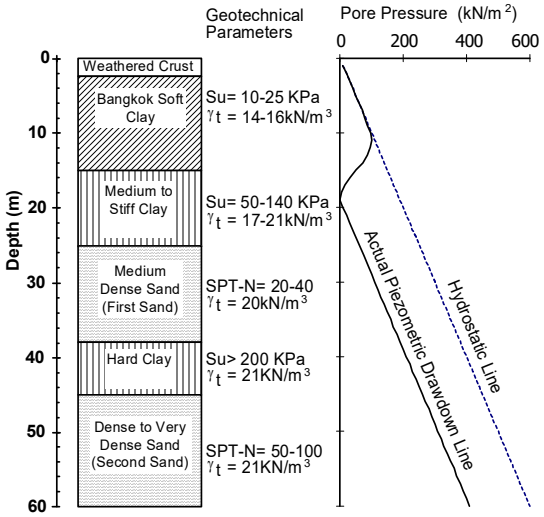


Figure 1. Typical soil profile of Bangkok with piezometric drawdown condition (Thasnanipan, et. al., 2002)

About 2m thick Medium Clay layer can be observed between Soft Clay and underlying Stiff Clay. Generally Stiff Clay layer occurs directly underneath Medium Clay and its depth goes up to 22m. Below Stiff Clay layer, First Sand layer 5-8m in thickness can be found. This First Sand layer, however, is absent in some areas. Stiff to Hard Clay layer underlies First Sand and it is found to be about 5m thick. Second Sand layer generally occurs at depths between 45 to 65m. Existing pore water pressure conditions in upper part of Bangkok Soft Clay are hydrostatic from nearly 1m below the ground level. Then the hydrostatic conditions change to piezometric drawdown near bottom level of Bangkok Soft Clay. Typical Bangkok subsoil and piezometric profile are shown in Figure 1.

3. OVERVIEW OF BORED PILE AND BARRETTE CONSTRUCTION METHOD

Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles and barrettes are constructed by wet-process or slurry displacement method. Rotary drilling method is used for bored pile with support of temporary casing length 12 to 18m, depending on the thickness of soft clay. Depending on the subsoil condition either bentonite or polymer-based slurry is used as supporting fluid.

A mechanical or hydraulic cable-suspended grab is commonly used for barrette construction. Excavation of the trench is carried out under pure bentonite slurry by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables (mechanical grab) or hydraulic action (hydraulic grab).

4. OVERVIEW ON DESIGN CONCEPT AND PARAMETERS SELECTION

In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literature from research carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored piles in Bangkok soft clay. With the passage of time, design method and selection of parameters for local subsoil were improved as a result of

research works carried out in 1980s. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from plain static pile load test results with numbers of assumptions since instrumented pile load test results were limited.

With the peak of construction-boom in Thailand, particularly in Bangkok, large numbers of instrumented full-scale static load tests on bored piles were conducted throughout 1990's which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were published based on these test data. With more confidence on soil parameters selection and better understanding on behavior of these deep foundations, the designer designed higher load capacity bored piles and barrettes in late 1990 than those in 1980s. Thasnanipan et. al., (1999) reported the failure mechanism of long bored piles in layered soil of Bangkok.

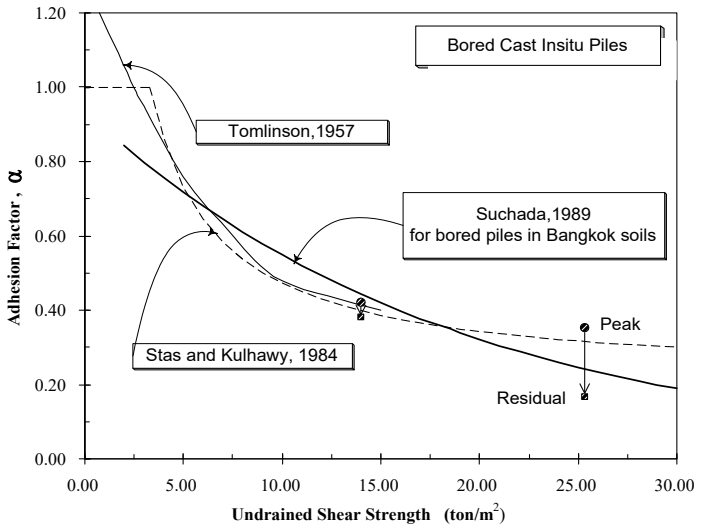


Figure 3. Comparison of adhesion factor α, suggested by different researchers with the actual mobilized in the stiff clay layers (Thansnanipan et. al 1999)

The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle-type failure mechanism of stiff to hard clay layers and the α values selected need to be adjusted accordingly. Peak and residual α values mobilized in the stiff clay

layers analyzed by the authors were plotted as shown in Figure 3 along with the suggested curves by different researchers. It can be seen from the figure that the residual α value of 2nd stiff clay layer (undrained shear strength values of 25 ton/m2), at the maximum test load, drops below the curve suggested by Pimpasugdi (1989). So the α values proposed by the author in Figure 3 for the stiff to hard clay layers overestimate the ultimate shaft friction under these conditions.

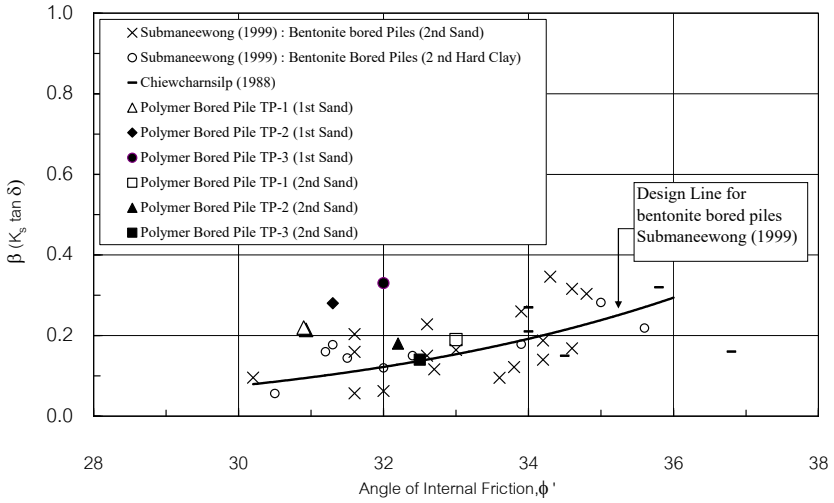


Figure 4. Back-calculated β values of polymer bored piles at maximum test load plotted on design line of bentonite bored piles constructed in Bangkok subsoil (Thansnanipan et. al 2002b)

Introduction of polymer-based slurry for wet-process bored piles marked a major breakthrough for both construction and design engineers. Thasnanipan et al.,(2002b), reported that bored piles constructed with polymer-based slurry have higher capacity than those constructed with bentonite slurry. Figure 4 shows the shaft friction factors β of sand layers for polymer-based bored piles in comparison with the design line of bentonite bored piles. The higher load capacity of bored piles constructed with polymer-based slurry allows the use of a single, large diameter deep-seated bored pile in place of a group of smaller size shallow-seated bored piles or driven piles. Figure 5 depicted the static pile load test result of 1.80m diameter bored pile of 60m depth constructed with polymer-based slurry in urban area of Bangkok. The test was conducted in late 2003. The maximum applied load 48,000 KN in this test is believed to be the highest static pile load test performed on a single bored pile in Thailand. Apparently, from

the test result, the pile (diameter 1.80m x 60m deep) could carry a significantly higher load than the original pre-defined safe design load of 16,000 KN.

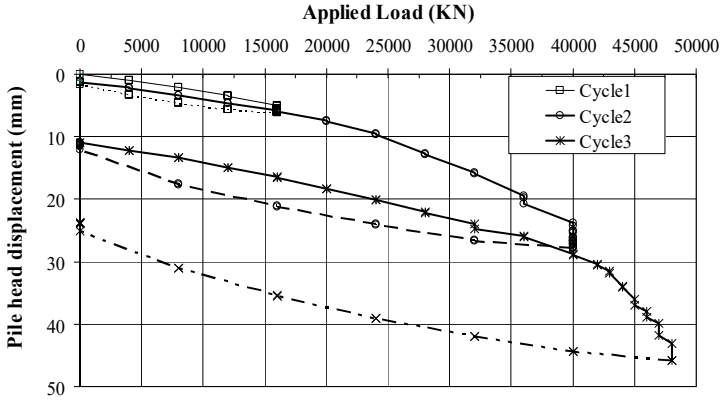


Figure 5. Static pile load test result of 1.80m diameter bored pile of depth 60m constructed by polymer-based slurry in Bangkok

5. AXIAL LOAD RESPONSE OF DEEP-SEATED BARRETTE AND BORED PİLE

Axial load response of instrumented barrette statically tested up to 5290 ton for foundation of a fifty-storey building in Bangkok was reported by Thasnanipan et. al (2002a) . The test results, particularly load transfer characteristics, and shaft friction capacity were compared with those of bored pile diameter 1.50m with the same length located 30m away. Since there are three key parameters difference between barrette and bored piles, such as excavation mechanism, construction time and shape-effect, response on applied axial loads of these two foundation elements would be of useful for geotechnical engineers.

5.1 Project Overview

Foundation of the fifty-storey tower called for 560 bored piles of 1.2m and 1.5m diameter and 24 number of barrettes having cross section size of 1.5m x 3.0m. Bored piles and barrettes were seated at depth of 57m in the second sand layer. Barrettes were designed to support the large load for tower's lift shafts as bored piles were not feasible to utilize in such case. Base grouting was applied for barrettes

and bored piles at the locations of high column load mainly in the central tower area. Instrumented static pile load test was proposed for one barrette and one bored pile of diameter 1.5m. The design safe working loads for the base grouted barrettes and bored piles are 2,300 ton and 1,175 ton respectively.

5.2 Construction Method

Both barrette and bored pile were constructed by wet-processed method under bentonite slurry. Properties of the bentonite slurry used are given in Table 1. Single-stage base grouting was applied 24 hours after concreting for both barrette and bored pile. Time consumed in different activities of two foundation system are shown in Figure 6.

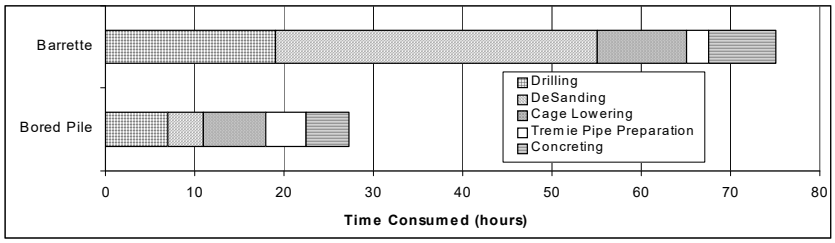


Figure 6 Time consumed in different construction activities of barrette and bored pile (Thasnanipan et. al 1999)

5.3 Test Pile Layout

Testing frame arrangement of barrette load test is shown in Figure 7. With three-layer reaction beams having overall height of 8m, to achieve the maximum capacity of 6000 tons, test frame utilized for static load testing on barrette in this project was claimed to be one of the biggest of its kind in the region. Four barrettes were used as anchoring system. Sixteen numbers of hydraulic jacks each having 500 ton capacity were placed between the test barrette cap and the main beams of the reaction frame.

Test pile layout of bored pile was similar to those of other static bored pile load tests in Thailand. Steel test frame anchored against four bored piles was used in bored pile load test. Different from the test frame of barrette, only two-layer of reaction beams were required for that of bored pile.



Figure 7. General view of the reaction frame for static load test on barrette Load Test

5.4 Monitoring System and Instrumentation

Vibrating wire strain gauges (VWSGs) and Mechanical Extensometers (ME) were fixed at five levels along the shafts at the known interface boundaries of different soil layers. At each level, four sets of VWSGs and one set of ME were installed for the pile whereas six sets of VWSGs and two sets of ME were installed for the barrette.

5.5 Load Test Results and Interpretations

5.5.1 Pile Head Movement at Applied Load

Load vs pile head movement graph of barrette and bored pile is illustrated in Figure 8. Measured pile head movements at design load, double of design load and maximum test load are presented in Table 2. As can be seen in the table, both barrette and bored pile experienced only negligible pile head movement at relevant design load.

Table 1 Predicted and measured pile head movement at specific applied load

Test Pile	Design Load, DL, (ton)	Max. Test Load (ton)	Measured gross pile head movement at applied load		
			At DL	At 2 x DL	At Max. Test Load
Barrette	2300	5290	-5 mm	-12 mm	-61 mm
Bored Pile	1175	2700	-6 mm	-19 mm	-67 mm

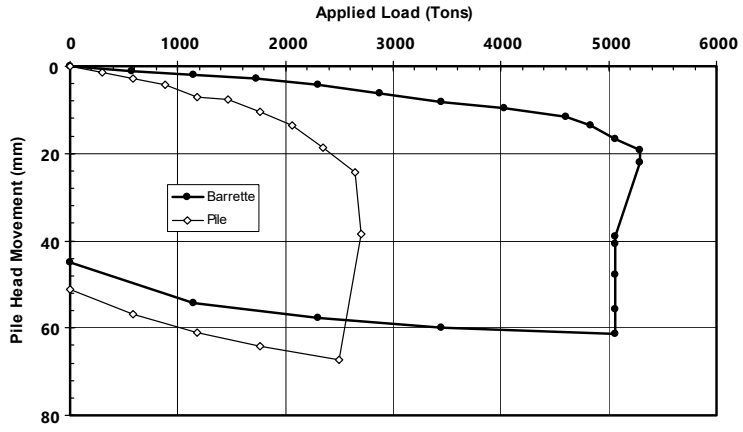
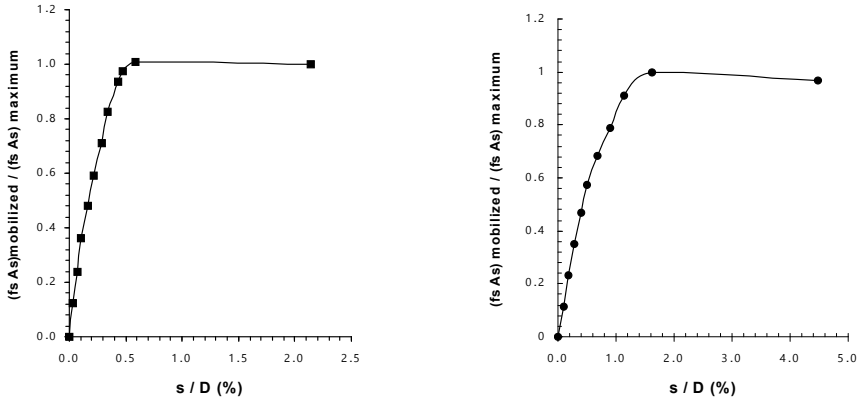


Figure 8. Pile head movement at the last load cycle (maximum loading) of barrette and bored pile

5.5.2 Mobilized Skin Friction vs Pile Head Movement

The graphs showing the ratio of mobilized skin friction to maximum mobilized skin friction against ratio of pile head movement to pile diameter (s/D) are presented in Figure 9a and 9b respectively..



9a 9b

Figure 9. Ratio of mobilized skin friction to mobilized maximum skin friction against ratio of pile head movement to equivalent pile diameter

Symbol 'D' shown in Figure 9a represents the barrette dimension in equivalent diameter. According to the figures, in general, skin friction mobilized to the maximum values at pile head movement of 0.6% and 1.6% of equivalent shaft diameter for barrette and diameter of bored pile respectively

5.5.3 Load Transfer Characteristics of Barrette and Bored Pile

Load transfer curves along the shaft of barrette and bored pile at various applied load are shown in Figure 10. The values of unit skin friction developed at the different soil layers along the shaft of barrette and bored pile in comparison with those of calculated ultimate unit skin friction are demonstrated in Figure 11. This comparison suggests that mobilized skin frictions of barrette and bored pile are higher than those of calculated values using empirical formulas. It proves the findings of various researchers on shaft friction improvement of base grouted piles (eg. Teparaksa et al 1999).

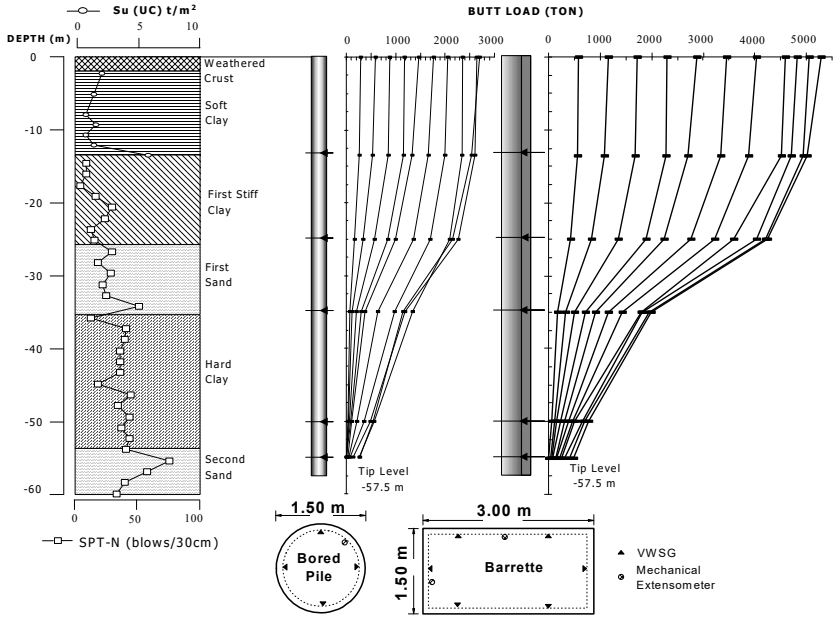


Figure 10. Load transfer curves of test barrette and bored pile with typical soil profile at the site

5.5. 4 Effect of Construction Time on Shaft Capacity Reduction

Excessive construction time is one of the main parameters claimed to be responsible for shaft friction capacity reduction of drilled shaft foundation constructed with wet process under bentonite slurry as reported by various researchers. Total construction time of barrette and pile were 75 hours and 27 hours respectively. Though total construction time consumed for test barrette was almost 3 times more than that of bored pile, there is no significant difference in shaft load transfer between them. The measures adopted in construction of barrette to minimize the excessive filter cake formation along the sand layers by proper desanding (continuous agitation), and retrenching with grab are considered the main reasons contributed to this achievement.

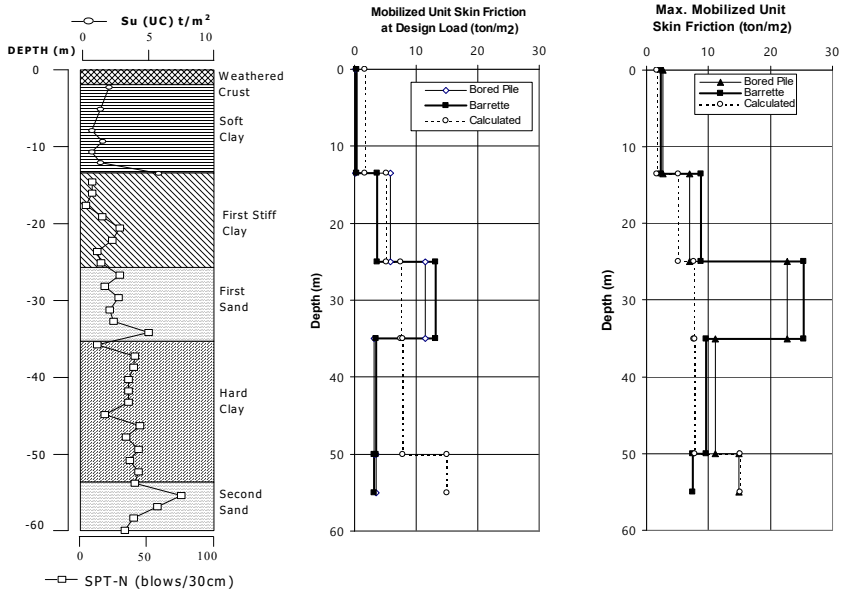


Figure 11. Comparison of calculated and mobilized unit skin friction of barrette and bored pile

5.5.5 Effect of Shape on Shaft Load Transfer

Hosoi et al (1994) concluded, from the results of numerical analysis that earth pressure acting on the flat surface of diaphragm

wall panel is larger than that of circular bored pile. Thasnanipan et al (1999) attempted to assess the effect of different aspect ratios (L/B) on the earth pressure developed around the trenches by using finite element program and reported that no significant difference in earth pressure was observed between the borehole (L/B=1) and the barrette (L/B=2). Further research is necessary to evaluate this finding. No significant difference is found between the maximum mobilized skin friction against displacement of rectangular-shape barrette and circular-shape bored pile according to Figure 6a and 6b respectively. Displacements at maximum mobilized skin frictions of barrette and bored pile fall within the range (0.5 % - 2 % of shaft diameter) reported by Reese (1978).

6. RESPONSE ON AXIAL LOADS OF BASE-GROUNTED BORED PILES

Although the wet-processed method is most suitable and its application has been well established for bored pile construction in Bangkok subsoil, the method itself cannot avoid undesired effects especially loosening and upheaval of soil at pile base as well as accumulated suspension of loose materials or sediments from drilling operation. These effects naturally may create a loose or soft pile toe causing negative impact on the pile capacity or load vs. pile head movement behavior. One of the special measures to improve the soft pile toe is to apply the post-grouting at pile toe. A comprehensive study on the effectiveness of two different toe-grouting methods, known as tube-â-manchette and drill-and-grout, commonly applied for deep-seated bored piles in Bangkok was presented by Thasnanipan et. al. (2004).

Based on the field application and measured data of instrumented pile load tests, the authors made the following conclusions. .

(1) Under design working load for large diameter deep-seated bored piles with toe grouting in Bangkok subsoil, the large portion of load (over 90% of applied load) is carried by the shaft friction. Only the small percentage of the total load is transferred to the pile tip. No significant difference of load-settlement behavior is found for non-grouted piles against TAM grouted and DAG grouted piles at design load and 2 times of design load. Therefore, for deep-seated bored piles in Bangkok subsoil, even with the toe grouting application, the end bearing component of bored piles could be still under-utilized. Thus the remaining

unmobilized end bearing capacity serves only as a reserve which forms the additional factor of safety.

(2) Despite the considerable extra cost, various construction problems encountered and risks involved in successful execution of grouting, drill-and-grout method does not offer particular advantage over tube-â-manchette method as far as pile capacity improvement is concerned.

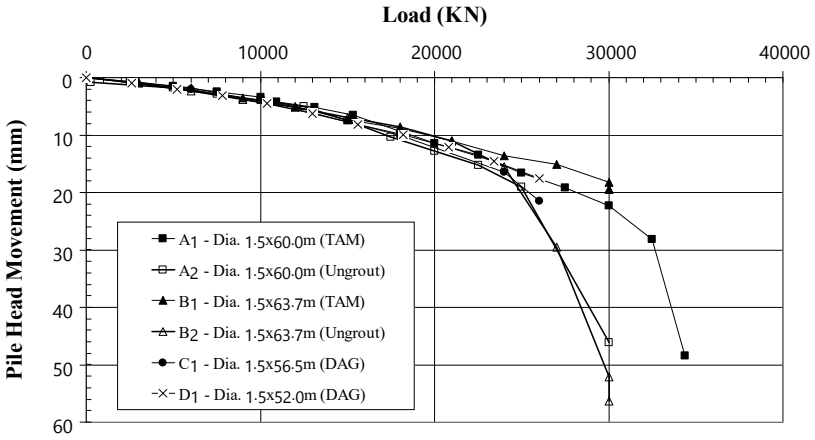


Figure 12. Load vs. pile head movement of ungrouted and toe-grouted piles

Table 2 Summary of the load test results of test piles in Figure 12.

Description		Project A		Project B		Project C Pile No. C1 (DAG Grout)	Project D Pile No. D1 (DAG Grout)
		Pile No. A1 (TAM Grout)	Pile No. A2 (Non-grout)	Pile No. B1 (TAM Grout)	Pile No. B2 (Non-grout)		
Diameter (m)		1500	1500	1500	1500	1500	1500
Length (m)		60.00	60.50	63.70	63.70	56.50	52.00
Q_d (KN)		10,000	10,000	10,000	10,000	13,000	13,000
Q_L (at 20mm total settlement)		26,500	23,200	34,280	21,750	25,500	26,000*
% increase in Q_L		14.20		21.57		N.A	N.A
Total pile head movement at	at Q_d	3.50	3.00	4.20	3.75	6.10	5.80
	1.5 Q_d	6.50	6.50	7.20	6.00	11.90	11.00
	2 Q_d	10.50	12.00	10.00	9.70	21.50	17.50

Note : * At 17.50mm pile head movement

7. PREDICTION OF SETTLEMENT AGAINST AXIAL LOAD OF DEEP-SEATED LARGE-DIAMETER BORED PILES

For deep-seated large-diameter pile design, in addition to analysis of allowable bearing capacity, pile settlement calculation is required to minimize differential and excessive settlements of the buildings. The following section presents prediction of settlement using analytical methods from the work of Boonyarak et. al (2016).

7.1 Settlement Prediction for Deep-seated Bored Piles

For deep foundation, apart from capacity of pile, accurate settlement calculation is also a key factor to be considered. To calculate pile settlement, analytical solution is commonly adopted for simplicity. Tomlinson (1995) proposed an equation to calculate pile settlement as shown in Eq. 1. For brevity of the paper, detail of each parameter is not described. To sum up, the first and second terms refer to elastic shortening of the pile and settlement of soil at pile base, respectively.

$$\delta = \frac{(W_s+2W_b)L}{2A_sE_p} + \frac{\pi W_b B(1-\nu^2)I_p}{4A_bE_s} \quad (1)$$

Bowles (1996) considered pile shortening according to each layer of soil along pile shaft and calculate soil settlement at pile toe (refer to Eq. 2).

$$\delta = \Sigma\left[\frac{P_{av}\Delta L}{A_{av}E_p}\right] + \Delta qD\frac{(1-\nu^2)}{E_s}mI_SI_FF_1 \quad (2)$$

Vesic (1977) adopted three components of pile settlement as stated in Eq. 3. In this equation, the first, second and third terms are pile settlement due to elastic shortening, soil compression at pile base and soil compression along the pile shaft, respectively.

$$\delta = \frac{(Q_{wp}+\xi Q_{ws})L}{A_pE_p} + \frac{Q_{wp}C_p}{Dq_p} + \frac{Q_{ws}D(1-\nu^2)I_{ws}}{pLE_s} \quad (3)$$

7.2 Back-analysis of Pile Load Tests

25 instrumented pile of diameters ranging from 0.80 m to 1.65 m and depth of pile tip are varying between 41 m and 59m load tests

in Bangkok soils were analysed. All piles were tested using slow maintained load test method with maximum test load of 2.5 times the design safe working load. Analyzed data can be categorized into four groups such as circular bored pile with pile tip in sand and clay, barrette with tip in sand and clay. For estimation of pile settlement, Young's modulus of sand and clay are assumed to be 2000N and $800C_u$, respectively (adopted from Aye et al., 2015). Poisson's ratio of soil is assumed to be 0.25. For concrete pile, modulus of elasticity 27 GPa is used.

7.3 Interpretation of results on settlement of circular bored pile

Results of load-settlement relationship of circular bored pile are shown in Figure 13. For comparison of data in each test, applied load is divided by shaft area (P/A_s) and settlement at pile head is normalized by its pile diameter (δ/D). General trend of load-settlement curves can be described by bi-linear relationship. The first part of linear is where normalized settlement is within 2%. After that, load settlement relationship becomes curved or so-called transition zone. The second part of linear is the zone of higher settlement rate than the first part of linear.

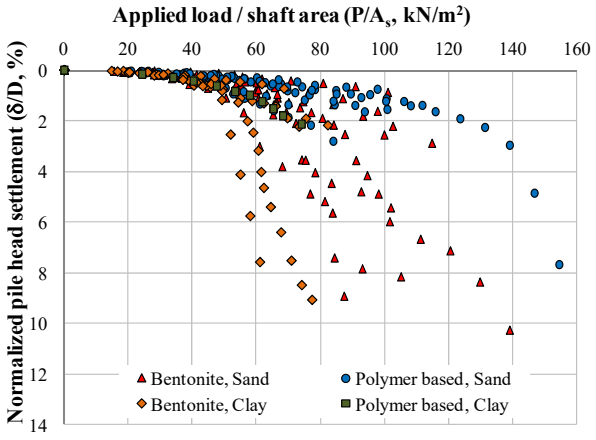


Figure13. Load-settlement curves of circular bored pile with different soil types at pile tip and drilling slurries

For bored pile constructed using bentonite slurry with pile tip in sand, P/As at normalized settlement at 2% is between 70 and 90 kN/m^2. By comparing with pile constructed using polymer slurry

having pile tip in sand, the P/A_s of this type of pile at the same δ/D was ranging from 90 to 120 kN/m^2. It suggests that shaft friction of bored pile using bentonite slurry is smaller than those with polymer slurry. This is because thicker filter cake in the former than in the latter (Thasnanipan et al., 2003), resulting in smaller shaft friction. Note that only one set of data from pile using polymer shows a bi-linear behavior as axial load did not transfer to the toe of most of piles of this type.

The effects of drilling slurry on pile settlement also influenced the piles with tip in clay. At settlement of 2%D, P/A_s for piles constructed using bentonite slurry and polymer slurry are 50-70 kN/m^2 and 70 kN/m^2 (data only from one pile), respectively.

Considering type of soil at pile base, it is found that pile with tip in sand had smaller settlement than those having tip in clay (given the same type of drilling slurry). This is because soil stiffness (e.g. Young's modulus) of sand (at depth of more than 40 m) is substantially larger than clay as stiffness proportionally increases with depth for sand.

7.4 Settlement of barrette

Figure 14 shows the load-settlement relationship of barrettes with tip in sand and clay. All barrettes were constructed using bentonite slurry as polymer slurry was not applicable to stabilize the trench of barrette.

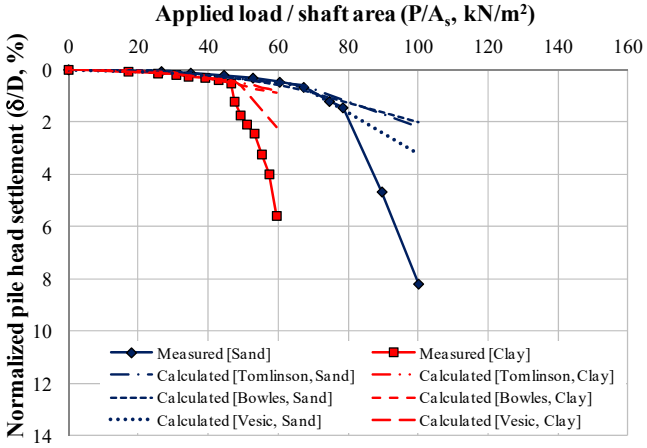


Fig. 14 Comparison of effects of soil type at pile tip on settlement of barrettes

From the comparison, settlement of barrette with pile tip in clay was substantially larger than those in sand. For example, at P/A_s of 60 kN/m^2, settlements of barrette with tip in sand and clay were 0.5%D and 5.5%D, respectively. This observation was consistent with those of circular bored pile (refer to Fig. 13).

For effects of shape on settlement of barrette, at δ/D of 2%, P/A_s of barrette with tip in sand was about 80 kN/m^2, which is in the same range of those for circular bored pile (constructed using bentonite slurry) of from 70 to 90 kN/m^2. When tip of barrette was in clay, P/As was about 50 kN/m^2, which is similar to those of bored pile (50-70 kN/m^2). This finding suggests that shape of barrette does not significantly affect the settlement behavior.

The three approaches for calculating deep foundation settlements were compared with the measured results (refer to Fig. 14). In the region where pile head settlement was between 1%D and 2%D, the three methods reasonably captured the measured results as elastic shortening of the pile was the major factor of pile head settlement. As the load was increased, settlements calculated by Tomlinson (1995) and Bowles (1996) were significantly underestimated. The calculated result obtained from Vesic (1977) had a larger settlement than the other two but still underestimated the measured one. This is because shear modulus of soil considered in the analytical solution was assumed constant, resulting in underestimation of settlement at higher load. This effect of stiffness on pile settlement is explained in details in the work of Boonyarak et. al. (2016).

7.5 Proportion of pile settlement

Pile shortening and toe settlement of the pile predicted by three different methods explained in previous section in comparison with actual measurement are shown in Figure 15. Measured result of normalized pile shortening was 1.4% or equivalent to 0.52 PL_p/A_bE_p; where P is maximum test load, L_p is the pile length, A_b is the cross-sectional area of the pile and E_p is Young's modulus of the pile. Pile shortening calculated from Tomlinson (1995), Bowles (1996) and Vesic (1977) were ranging from 1.8%D to 2.0%D. Similarly, pile shortening obtained from numerical analysis was 1.9%D. It is found that both approaches provided reasonable value of pile shortening compared with the measured result. The normalized toe settlement from measured result was 8.9%D but the calculated results using analytical solution substantially underestimated the measured one.

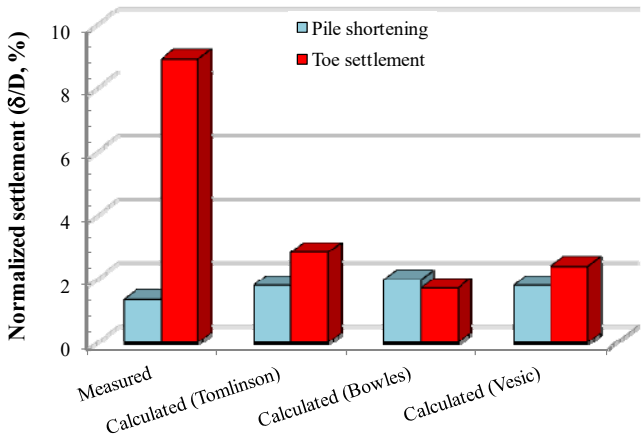


Figure 15 Proportion of pile settlement comparison

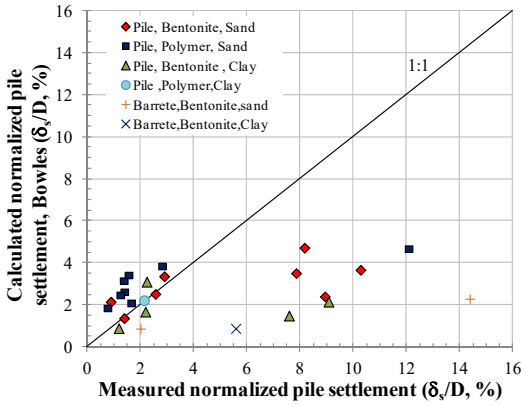
7.6 Accuracy of pile settlement estimation by analytical solution

Comparisons of maximum measured pile head settlement and calculated result from the three analytical solutions are shown in Figure 16.

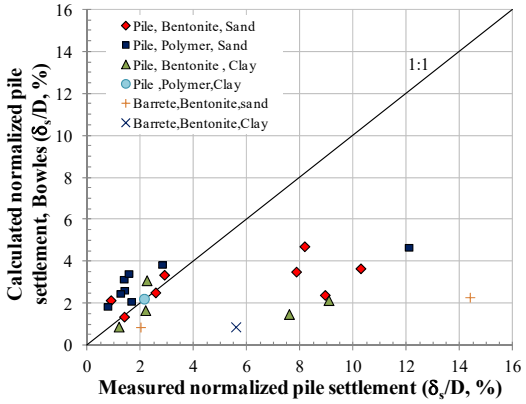
Figure 16a shows comparison of pile settlement estimation using Tomlinson (1995)'s approach with measured results. At pile head settlement not more than 2%D, calculated settlement was slightly larger than the measured one. For this settlement region, the analytical solution was still reasonably estimated the measured settlement with error of from 0.5%D to 1.0%D.

However, for pile with settlement larger than 5%D, the calculated settlement significantly underestimated the measured result by 3%D to 8%D. In addition, estimation of pile settlement constructed by using polymer slurry was more accurate than those constructed with bentonite slurry and barrette. Thus, one should be aware when using the analytical solution to estimate settlement of pile that has a tendency to be larger than 5%.

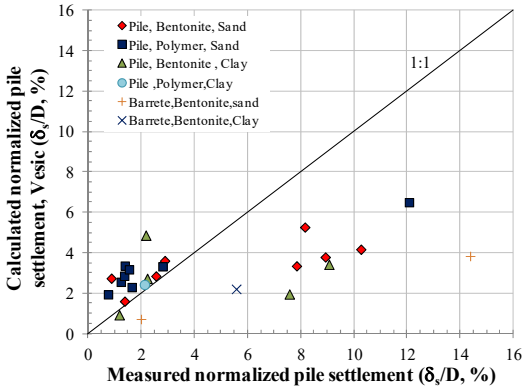
Comparisons were also made between two analytical solutions of pile head settlement using Bowles (1996) and Vesic (1977) and the measured results as shown in Figure 16b and 16c, respectively. These two analytical solutions provided similar trend to those obtained from Tomlinson (1995). Thus, to predict pile settlement, any of the three analytical solutions can be adopted up to pile settlement of 5%D.



(a) Tomlinson, 1995



(b) Bowles, 1996



(c) Vesic, 1977

Fig. 16 Comparison of settlement prediction using following approach: (a) Tomlinson, 1995; (b) Bowles, 1996; (c) Vesic, 1977

7.7 Pile Capacity Estimation from Analyzed Load-Settlement Graphs

Pile capacity may be determined using relationship between applied load per shaft area (P/A_s) and normalized pile head settlement (δ/D) based on Figures 13 and 14, If pile settlement of 5%D is set as an ultimate load bearing capacity settlement, average P/A_s can be obtained for each type of pile. For example, bored pile embedded in sand, bored pile constructed using polymer slurry and bentonite slurry provided P/As of 140 kN/m^2 and 80-100 kN/m^2, respectively. From average load per unit area times total shaft area, ultimate load bearing capacity can be estimated. Table 3 summarizes average load per unit area at normalized settlement of 2% (settlement at fully mobilized shaft friction) and 5% (settlement at ultimate bearing capacity). It is to be noted that pile capacity determination described above is based on the analyzed data of 25 bored piles in Bangkok subsoil and it may be used in preliminary stage of design process for rough estimation.

Table 3 Summary of load-settlement relationship

Pile type	Slurry type	P/A_s , kN/m^2	
		at δ/D of 2 %	at δ/D of 5 %
Bored pile, sand	Bentonite	70 - 90	80 - 100
Bored pile, sand	Polymer	90 - 120	140
Bored pile, clay	Bentonite	50 - 70	60 - 80
Bored pile, clay	Polymer	70	N/A*
Barrette, sand	Bentonite	80	90
Barrette , clay	Bentonite	50	60

Note: * Pile settlement did not reach 5%D

7.7 Discussions

Based on back-analyzed data from 25 numbers of instrumented pile load test, following conclusions may be drawn:

(a) For deep foundation embedded in the same soil type, settlement of bored pile constructed using bentonite slurry is substantially larger than those constructed with polymer slurry. This is because of thicker filter cake in former than the latter, causing reduction in shaft friction.

(b) By comparing bored pile constructed using the same type of slurry, bored piles embedded in clay have larger pile settlement than

those embedded in sand. This is due to smaller soil stiffness in the former than the latter.

(c) There is no significant difference in settlement between barrette and circular bored pile (refer to Figure 13 and 14). Thus, load-settlement relationship of bored pile can be adopted to estimate the settlement of barrette provided that under same category of data range of normalized load-settlement graph in Figure 13 (i.e. bentonite slurry bored pile and same soil type sand or clay at pile toe) is used.

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