

BORED AND DRIVEN PILE TESTING IN BANGKOK SUB-SOILS

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ABSTRACT: As a necessity to support an increasing magnitude of loads from tall buildings and long span bridges, the piling practice in the Bangkok Plain has moved several phases from driven pre-cast reinforced and pre-stressed concrete piles of smaller cross sections to spun piles and large diameter bored piles. The Chao Phraya plain in which the Bangkok city is located is low-lying and consists of a broad basin filled with sedimentary soils which form alternate layers of clay, sand, and clay. The upper clay layer is soft and highly compressible followed by a stiff clay layer extending to about 20 m or so and then followed by a layer of sand. Driven piles are normally taken down to this upper sand layer. However when the demand for a higher capacity arise, these piles cannot be extended in length due to construction problems and as such bored piles are needed to be taken down to as deep as 50 to 60 m. Below the upper clay layer there are eight interconnected aquifers from which ground water is pumped from deep wells. Thus in the design of piled foundations aspects such as the negative skin friction due to pile driving as well as deep well pumping are also needed to be considered. Some of the experiences gained over a period of 30 years in the study of piled foundations in the Bangkok Plain are briefly presented in this paper.

Keywords: Pile foundations, field test, stress analysis

INTRODUCTION

The Bangkok city now has a population of over ten million people. Between 1980 to now the construction activity is very high for tall buildings, expressways, and bridges (Heidengren, 2003). It is interesting to note the development of piling practice in Bangkok. Prior to 1980 the buildings were few stories and driven piles were used. Now bored piles are dominating. The nominal diameter of bored piles range from 0.3 to 2m; but in most cases about 1 to 1.5 m. They extend to depths of even 60 m. The design of these bored piles still has the trace of the original design of driven piles, which extended to depths of 10 to 30 m, but mostly 20 to 30m (Balasubramaniam *et al.*, 1981; Sambhandaraksa and Pitupakorn, 1985; Balasubramaniam, 1991).

The upper soft clay in the Bangkok plain is of low strength and high compressibility. Also the Plain is low lying and is subjected to flooding in the rainy seasons. Additionally continuous subsidence takes place due to deep well pumping from the underlying aquifers. In most housing and industrial projects the land is raised above the flood level by filling and this cause settlement in the compressible soft clay layer. Thus negative skin friction is of importance in piles of low capacity driven to

shallow depths (Phamvan, 1990; Indraratna *et al.*, 1992). Both total and effective stress analysis were carried out on these driven piles. Bored piles have the great advantage that they do not cause soil displacement and remolding effects. In order to increase the capacity of these bored piles, shaft and toe grouting were done. The mobilization of skin friction in the clays, sand and the end bearing were studied by instrumentation of large diameter bored piles. The failure loads of the piles have increased from some 4000kN to more than 28000kN (see Fig. 1). Piled foundations were the subject of research at Asian Institute of Technology (AIT) led by the first author and were investigated by several graduate students as part of their thesis (Adhikari, 1998; Anwar, 1997; Chun, 1992; Fernando, 1992; Oonchittikul, 1990; Phota-Yanuvat, 1979; Roongrujirat, 1983; Soontornsiri, 1995; Wachiraprakarnpong, 1993).

BANGKOK SUBSOILS

Below the upper soft clay layer, there is stiff clay and further down alternating layers of dense sand and stiff clay. Researchers at AIT have done extensive work on the Engineering Geology of these deposits. The

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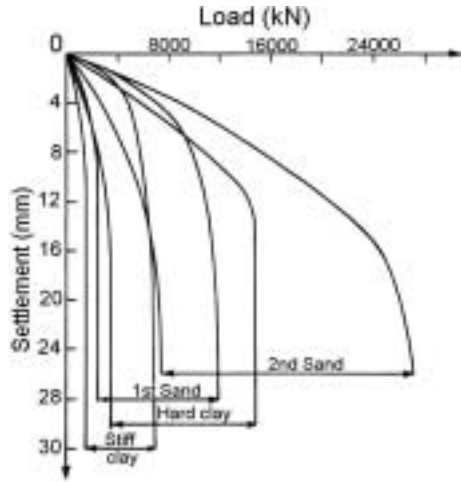


Fig. 1 Range of failure loads of piles with increasing depths of bearing levels

thickness of the sediments ranges from 550 to 1000m or so. Typical deep soil profiles and shallow ones are shown in Fig. 2 and Table 1. For foundation purposes the sub-soil layers of interest are: weathered clay is the uppermost layer, the thickness of which varies from 1 to 4m and undrained shear strength 30 to 50 kN/m². This is followed by soft clay: the thickness is about 10 to 15 m. The soft clay is of high natural water content and average undrained shear strength 10 to 20 kN/m². Below the soft clay is medium stiff-to-stiff clay; the thickness is around 10 to 20 m. The stiff clay has higher shear strength of more than 100kN/m² and compressibility is very low. Below the stiff clay is the first sand layer and is generally found at a depth of 20 to 30m. There is an inter-bedded layer of sandy clay or clayey sand between the stiff clay and the first sand layer. The first sand layer can be classified as silty sand (SM) with a relative density in the range of medium dense to dense. Below this sand layer is hard clay or second stiff clay, which seems normally consolidated with low compressibility and undrained strength over 150kN/m². The second sand layer, which is found at depths of more than 50 m, follows this second stiff clay. It is generally very dense in nature. The piezometric drawdown and the effective stress increase are shown in Fig. 3. The ground water table is about a meter or so below the ground surface.

DRIVEN PILES

Conventional driven piles were used in Bangkok for supporting buildings and other infrastructures for many decades, until 1980, when bored piles became to be popular. These piles generally extend down to the first sand layer (see Fig. 2), which is located around 20 to 30m depth below the ground surface. Driven piles were

Table 1 General properties of Bangkok subsoils

Soil type	Depth (m)	w_n (%)	w_l (%)	w_p (%)	I_p (%)
Weathered clay	0-2	35-70	35-55	-	23-30
Soft clay	1-16	65-90	65-90	30-40	40-63
Stiff clay	10-25	24-34	40-75	20-28	18.50
First sand	14-38	17-25	-	-	-
Hard clay	24-43	24-43	30-35	55-69	31-44
Second sand	30-58	20	-	-	-

		γ (kN/m ³)	e	G_s
Weathered clay	0-2	16-18	1.3-1.4	2.6-2.7
Soft clay	1-16	15-17	1.5-2.6	2.7
Stiff clay	10-25	19-20	0.65-0.95	2.7-2.8
First sand	14-38	18-21	0.7	2.7
Hard clay	24-43	18-20	0.8-0.95	2.7
Second sand	30-58	18-26	0.75	2.7

normally designed using a total stress analysis. The undrained shear strengths in the soft and medium stiff clays were measured using field vane tests and in the stiff clay from laboratory UC, and UU tests. Also, SPT tests were performed in the boreholes and for stiff clay and sand, the SPT values are used to determine the undrained shear strength in clays and the skin friction and end bearing in sand layer. Dutch cone tests were conducted to measure the cone resistance and sleeve friction and these values are used directly to estimate the pile capacities. A scale factor of 1.2 needs to be used to magnify the predictions based on SPT and CPT data, in order to match the test loaded values of the capacity of large diameter spun piles (Balasubramaniam, 1991).

Total Stress Analysis

The α value for the adhesion in the clays is normally used from the adhesion factor undrained shear strength relationship established by Holmberg (1970), Meyerhof, and Peck, Hanson and Thornburn. Sambhandaraksa and Pitupakorn (1985) presented an interesting account of the design of driven piles using SPT data. They also presented the correlation of undrained strength with SPT values depending on the clay type. For CU clays, unconfined compressive strength $q_u = 1.37 N$ and for CL clays, the corresponding relationship is $q_u = 1.04 N$. The variation of α with undrained strength is as shown in Fig. 4. These authors also presented a correlation of $\bar{\phi}$, the angle of friction with SPT values of N as shown in Fig. 5. The unit skin friction in sand is taken as

$$\tau_s = K_s \tan \bar{\phi} (\bar{\sigma}_{v0})_{ave} \quad (1)$$

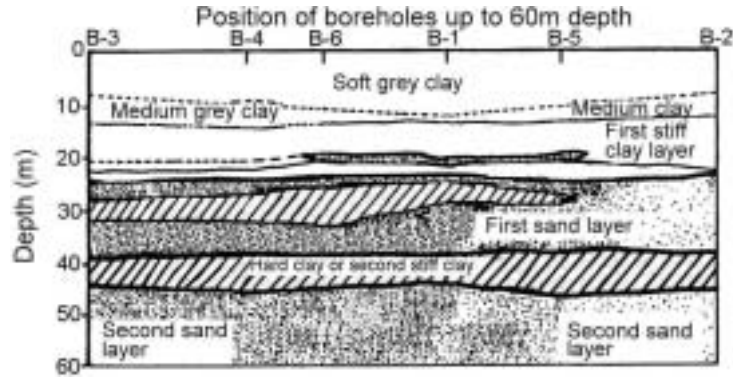


Fig. 2 Typical deep soil profiles (based on Sambhandaraksa and Pitupakorn, 1985).

where K_s , the coefficient of lateral earth pressure is assumed to be $(1 - \sin \bar{\phi})$, and $(\bar{\sigma}_{v0})_{ave}$ is the average vertical effective overburden pressure under in-situ conditions.

The work of Sambhandaraksa and Pitupakorn (1985) is found to be reliable for the estimation of the capacity of small cross section driven piles in the upper clay layer and sand. However, as stated earlier, for spun piles of 600mm and 800 mm diameter, it appeared that the capacity computed from the approach of Sambhandaraksa and Pitupakorn (1985) needs to be enlarged by a factor of 1.2 (Balasubramaniam, 1991). The friction factor and end-bearing factor used with the CPT tests are tabulated in Table 2. Typical cone resistance and skin friction from CPT tests are shown in Figs. 6 and 7.

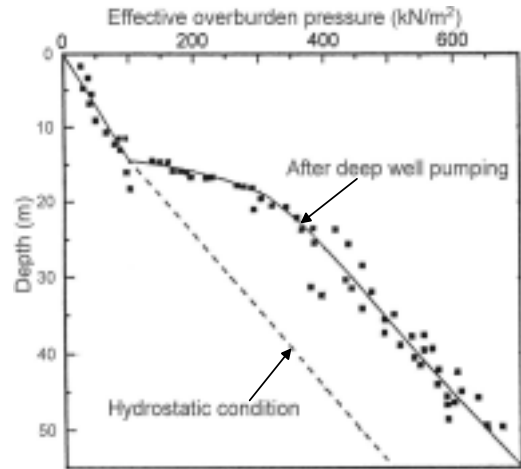


Fig. 3 Variation of effective overburden pressure with depth

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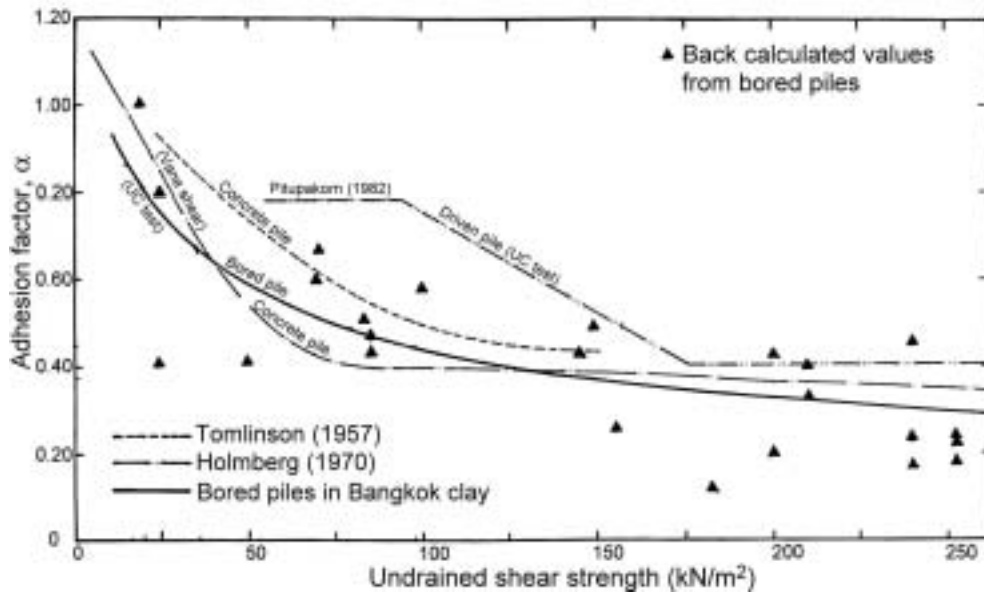
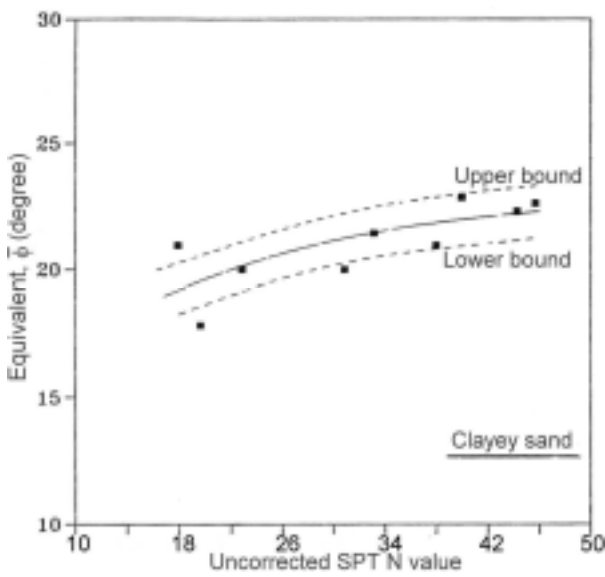


Fig. 4 Adhesion factor for clays (based on Sambhandaraksa and Pitupakorn, 1985)

Table 2 Skin friction and end bearing factors in CPT tests

References	Skin friction factor, α		
	Soft clay	Medium stiff clay	Stiff clay
Chottivittayathanin (1977)	1.1	0.7	0.5
Phota-Yanuvat (1979)	1.0	0.7	0.5
	End bearing factor, λ		
	Sand	Clay	Sand
Chottivittayathanin (1977)	0.5	0.33	0.5
Phota-Yanuvat (1979)	0.8	0.33	0.5

Fig. 5 Relationship between uncorrected SPT N value and equivalent ϕ of clayey sand (based on Sambhandaraksa and Pitupakorn, 1985)

Effective Stress Analysis

The long-term behavior of piles indicates the pore pressure dissipation in the clay around the pile, and, Chandler (1968) suggested a design based on effective stress method. Burland (1973) has also applied a simple effective stress approach to the estimation of the shaft friction. The method of Burland (1973) was also used to calculate the shaft friction of piles in Bangkok clay. The effective stress parameter β is similar to α but is estimated from K_0 and the angle of friction, δ . The effective stress strength parameters were established for Bangkok clays especially in the upper clay layer to a depth of about 16m. These values from CIU tests at depths of 1, 1.5, 2.5, 3.9, 5.3, 5.4, 7.5, 9.3 and 11.5 m indicated zero cohesion in the normally consolidated state and ϕ' values of 20.2, 24.8, 21.9, 20.2, 21.4, 22.6,

21.4, 23 and 22.5 degrees. The corresponding values from CK_0U tests at depths of 3.8, 4.6, 5.3 and 8.1 m gave ϕ' values of 29.9, 27.8, 30.9 and 28.7 degrees. Also, the corresponding values of ϕ' from CID tests at depths of 3.2, 8.9, 15.2 and 16.4 m depths are 24.9, 22.4, 19.2 and 19.3 degrees respectively. The β values estimated using Burland (1973) method ranged from 0.24 to 0.29 at three sites in Bangkok. In this study the value of β was also estimated from full-scale pile load test data in clays by subtracting the end bearing value, to obtain the average value of β denoted as $\bar{\beta}$. Figure 8 shows the plot between average shaft friction ($\bar{\tau}_s$) and the average effective overburden pressure ($\bar{\sigma}_{v0}$). The $\bar{\beta}$ values range from 0.17 to 0.48 with an average value of 0.33. This value of $\bar{\beta}$ seems to predict the capacity of driven piles to first sand layer well (Balasubramaniam, 1978; Balasubramaniam et al., 1981).

Negative Skin Friction

Two full-scale tension piles were instrumented and initially driven in short lengths and then gradually extended in length (Indraratna et al., 1992; Phamvan, 1990). One pile was bitumen coated and the other pile was not. The piles are hollow, pre-stressed, pre-cast and spun concrete type. The outside and the inside diameters are 0.4m and 0.25 m respectively. Each instrumented test pile was divided into six segments, five of which were 4m in length and the last section is 6m long. Load cells were placed at the pile tip as well as at the connection joints of the segments. A tell tale type of system was adopted for the measurement of the pile compression and its movements. LVDT types of gauges were also used to measure the deformations. Figure 9 illustrates the β -

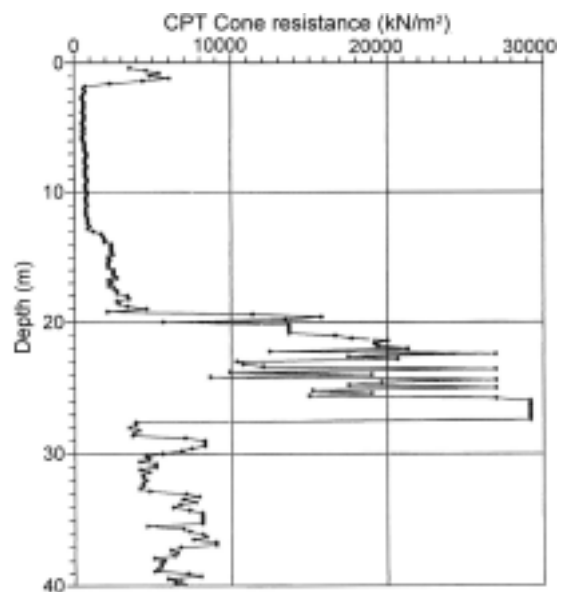


Fig. 6 Variation of CPT cone resistances with depth

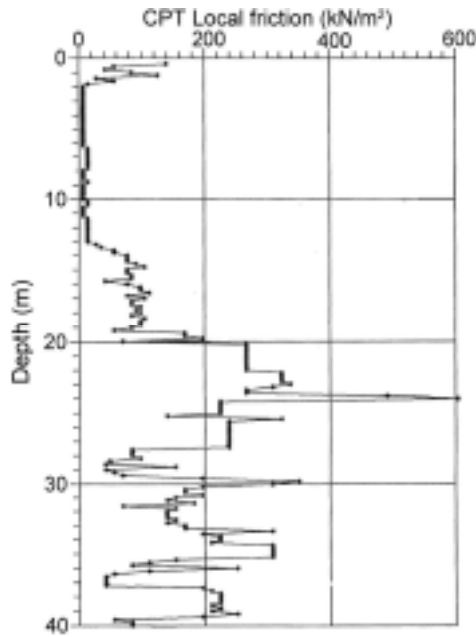


Fig. 7 Variation of CPT local friction with depth

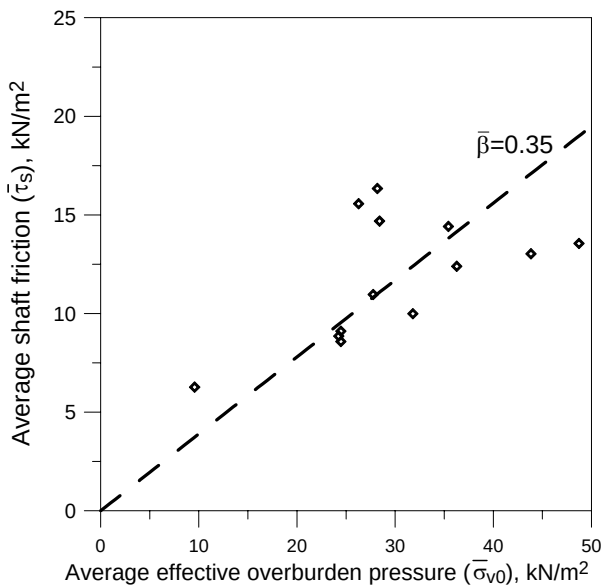


Fig. 8 Estimation of $\bar{\beta}$ from pile load tests

parameter obtained from the negative skin friction measurements under short term in the pull out tests and under long term with an embankment type of loading. The maximum negative skin friction developed was about $0.25 \sigma'_{v0} (\beta = 0.25)$, with an average value of about $0.2 \sigma'_{v0}$.

SPUN PILES

600 mm and 800 mm spun piles are commonly used in elevated expressways as well as in industrial buildings (Balasubramaniam, 1991). The capacity of these piles is

computed from the same expression used for other driven piles using SPT and CPT data. It appears that there is a scale effect when we move from smaller section driven piles to larger section spun piles. The magnifying factor was found to be nearly 1.20. In many instances, the pile load tests are stopped prior to pile failure. Fellenius (1980) studied a number of methods for the estimation of the failure loads of piles load tested to values closer to the failure state. Figure 10 illustrates the use of these methods to compute the failure load. The Mazurkiewicz (1972) method is found to predict the capacity well.

BORED PILES

Over 150 pile load tests on bored piles with the tips in the stiff clay, first sand layer, the hard clay and the second sand layer were analyzed. Some load tests were carried to failure while most others were taken to about 1.5 to 2.0 times the design load. The piles were also instrumented to determine the mobilization of skin

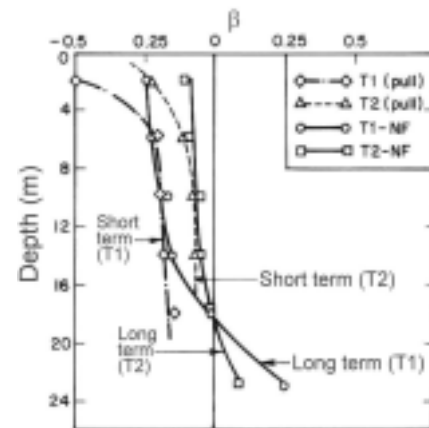


Fig. 9 Variation of β -parameter with depths during negative skin friction

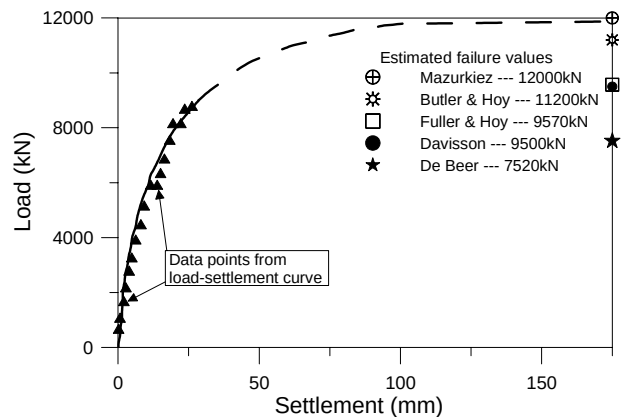


Fig.10 Load settlement characteristic of 0.8m diameter spun pile

friction and end bearing. With more and more emphasis on deformation based design than the limit based ones, it is important to understand the mobilization of skin friction and end bearing with the movement of the piles. Figure 11 to 14 illustrate such mobilization of skin friction and end bearing in the Bangkok sub-soils for the design of bored piles. These data are back calculated from several instrumented bored piled load tests, and can be used in deformation analysis of piled foundations and piled raft foundations (Lin *et al.*, 1999). Due to construction problems of bored piles and the variation in soil properties in sedimentary soils, the back calculated values of soil parameters and the mobilization of skin friction and end bearing from load tests show substantial scatter in the data points.

For the stiff Bangkok clay the undrained shear strength is about 200-250 kN/m² as obtained from unconfined compression tests and SPT correlations.

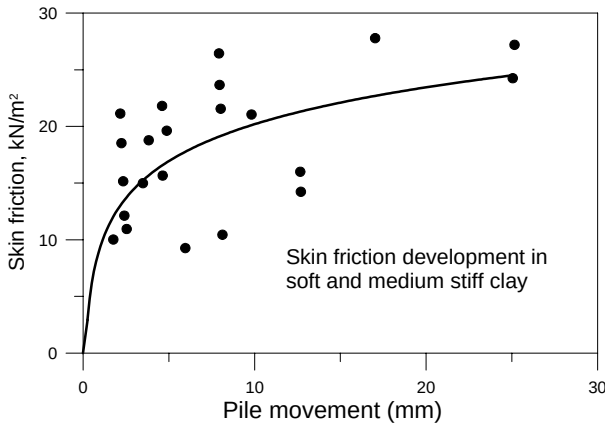


Fig. 11 Development of skin friction in soft and medium stiff clay with pile movement

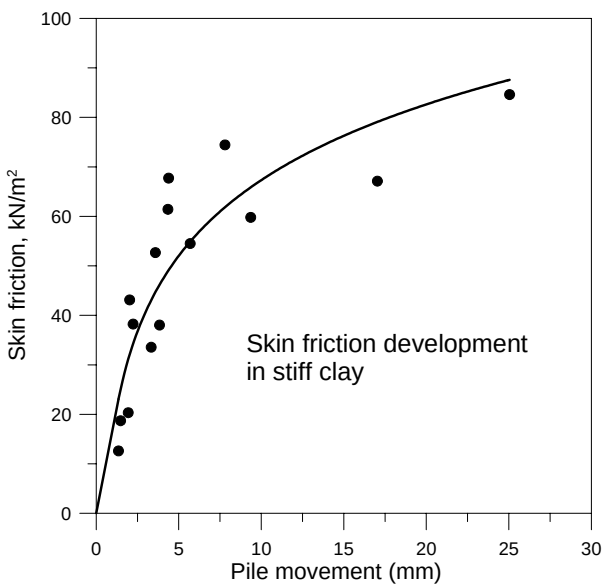


Fig. 12 Development of skin friction in stiff clay with pile movement

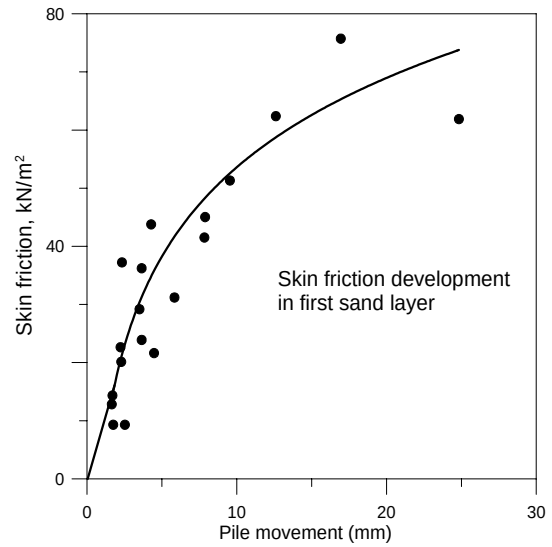


Fig. 13 Development of skin friction in first sand layer with pile movement

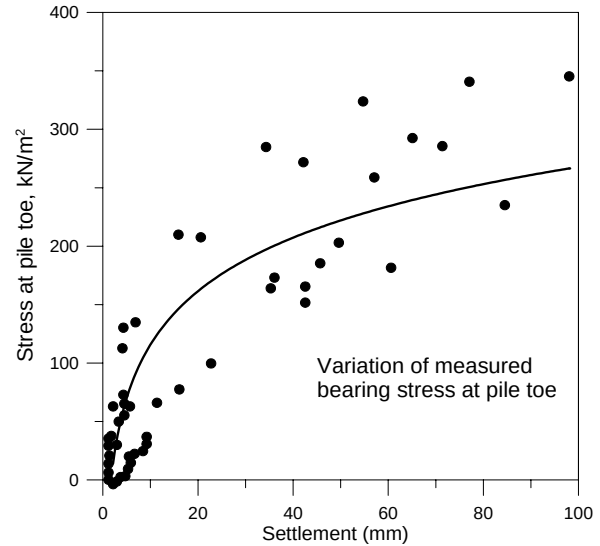


Fig. 14 Variation of measured bearing stress at the pile toe with settlement

The average skin friction is about 100kN/m²; this will then give a α value of about 0.4 to 0.5. It appears both the driven piles and the bored piles develop more or less the same skin friction in the stiff clay. If we assume an effective overburden pressure of 400kN/m² as the average value at the centre of the stiff clay, then the corresponding β value will be about 0.25.

In order to better mobilize the shaft friction and the end bearing in bored piles, both shaft grouting and base grouting are carried out. During shaft grouting, a very irregular bond area is formed and the strength of sand around the pile is increased; also the effective diameter of the pile. Thus the skin friction is increased due to the increase of the radial stresses along the pile-soil interface. Soil-grout adhesion in granular soil increases with depth

until the potential failure occurs at the pile interface. Base grouting is done after completion of the shaft grouting. The back-analyzed parameters indicated that, the adhesion factor (α) is higher for the shaft-grouted piles than for the non-grouted piles (see Fig. 15). Similarly, there is a substantial increase in $K_s \tan \delta$ (where, K_s is the coefficient of lateral earth pressure, and δ is the angle of friction for sand and pile surface) for the shaft-grouted piles in the sand than the non-grouted ones (see Fig. 16). Even though increase in end bearing values is noted for the grouted piles, the magnitude of the increase does not seem to be substantial. In the case of non-grouted piles also, very little mobilization of the load in end bearing is noted.

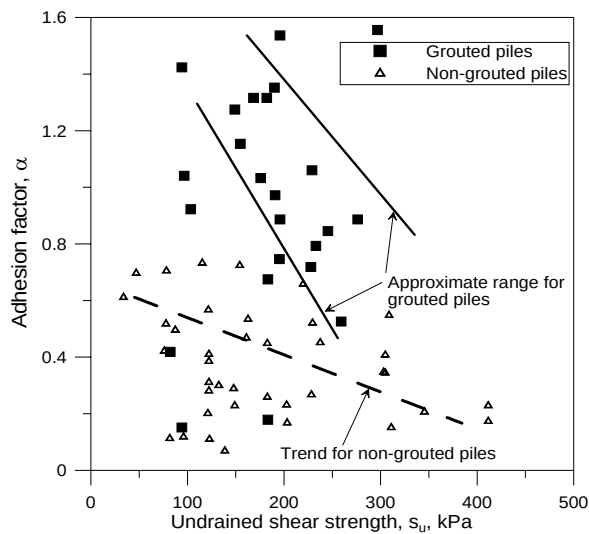


Fig. 15 Relationship between adhesion factor (α) and undrained shear strength of non-grouted and shaft grouted bored piles

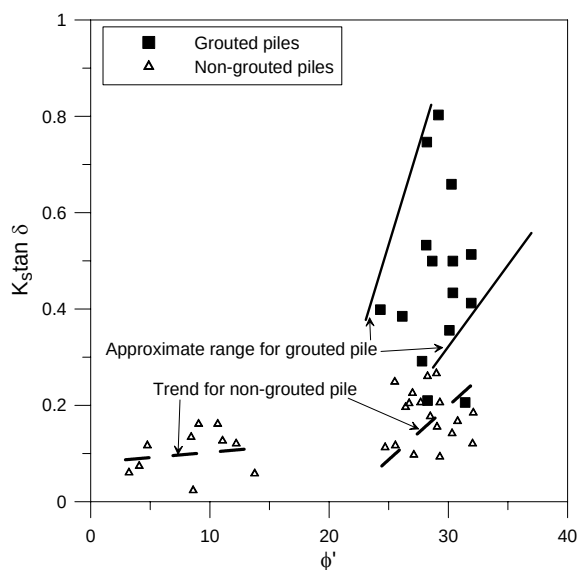


Fig. 16 Relationship between $K_s \tan \delta$ and ϕ' for non-grouted and shaft grouted bored piles

CONCLUSIONS

The paper summarizes the role of back-analysis and interpretations of pile load tests in Bangkok sub-soils and their influence in estimating the capacity of both driven and bored piles. The sedimentary soil conditions of the Bangkok subsoils with an upper soft clay followed by medium stiff and stiff clay, and then sand is ideally suited for driven piles. However the demand for piled foundations to carry an increasing magnitude of load had made a shift of the piling practice from driven piles to bored piles bearing at much deeper levels. Also, the extensive deep well pumping and the associated piezometric draw-down has caused a very large increase in the effective stress and as such the design of piled foundations needs to be based more on the effective stress analysis rather than the traditional total stress analysis. Back calculated values of soil parameters from the total and effective stress analysis as well as the mobilization of skin friction and end bearing with deformation for bored piles as estimated from load tests are also presented.

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