PILED FOUNDATIONS AND BASEMENT EXCAVATIONS FOR TALL BUILDINGS IN BANGKOK SUBSOILS

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ABSTRACT

This special lecture is based on the piling practice and the diaphragm walled excavations used for basements of tall buildings in the Bangkok plain. 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 prestressed concrete piles of smaller cross sections to spun piles and large diameter bored piles. Also included are the details of deep well pumping, the associated piezometric drawdown and the subsidence and their effect in foundation and basement construction practice. Total and effective stress analysis is presented for the piling works and aspects such as negative skin friction are also incorporated. The construction control of deep excavations in Bangkok subsoils is purely based on the measured lateral movements adjacent to the support system used in the excavation methods. Back analysis of the deformations of deep excavations supported by diaphragm walls is performed and the back-analysed soil parameters are also tabulated. Fourteen case studies involving deep excavations are presented here.

Keywords: Piled foundations; Excavations; Deep well pumping; Subsidence;

1. 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). In addition most tall buildings have basements and deep excavations were made in soft Bangkok clay using diaphragm wall techniques. The construction controls of these excavations are mainly based on the monitoring of wall movements. Thus predicted and observed wall movements are presented on a number of deep excavations with diaphragm wall supports.

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 (Balasubramaniam *et al*, 1990; 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 remoulding 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 placing large diameter bored piles. The failure loads of the piles have increased from some 4000kN to more than

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28000kN. Piled foundations and deep excavations were the subject of research at Asian Institute of Technology (AIT) by several graduate students as part of their thesis (Adhikari, 1998; Anwar, 1997; Chia, 1986; Chun, 1992; Fernando, 1992; Gan, 1997; Ng, 1999; Oonchittikul, 1990; Parnpoly, 1990; Phota-Yanuvat, 1979; Pitupakorn, 1982; Pornpot, 1997; Roongrujirat, 1983; Soontornsiri, 1995; Srichaimongkoi, 1991; Teparaksa, 1993; Tuk, 1998; Twarath, 1992; Wachiraprakarnpong, 1993).

Deep excavations are made in Bangkok mainly for basements of buildings. With numerous tall buildings cropping up in the eighties, deep excavations also increased in number and at the same time become deeper and deeper. In the first case for basements of buildings, the excavation reached 10 to 18 m and is always in the upper clay layer. Starting with steel sheet-piled walls, subsequently diaphragm walls and contiguous bored piled walls are also used. Except for few cases of mishaps, the use of concrete diaphragm walls for deep excavations in Bangkok has been relatively problem-free and very successful. The impervious nature of the Bangkok soft clay and stiff clay eliminates potential problems related to ground water inflow. Thanks to the piezometric draw down due to deep well pumping which further minimized the uplift and the boiling of the sand layer Fourteen diaphragm walled excavations were studied and their details are given in Table 1.

Table 1: Details of diaphragm wall excavations

No.	Project	Strut elevations (m)	Preload	Max. un-supported height (m)	Excavation depth (m)
1	Rajawej Hospital	-1.73, -6.8	No	7.57	-14.4
2	China Tower	-1.0	No	4.5	-5.5
3	Lumpini Park	-1.5	Yes	6.5	-8.0
4	President Tower	-1.5, -6.5	Yes	9.6	-16
5	Pratunam Complex	-1.5	Yes	7	-8.5
6	Central Rama III	-2.5	Yes	6.6	-9.1
7	Sathorn Complex	-2.4, -5.1, -9.0, -11.8	Yes	4.0	-15.8
8	IFCT Tower	-1.5, -6.8	Yes	6.9	-13.7
9	Green Tower	-2.0,-7.0	No	3.0	-10
10	Windsor Hotel	+1.0, -2.0, -9.5	No	6.0	-15.5
11	ITC Building	-2.8, -8.3, -13.3	No	5.2	-18.5
12	Oriflame Building	-1.5, -4.0, -7.5, -10.5	Yes	5.4	-14.0
13	TPI Building	-1.5, -4.3, -7.0, -9.8	Yes	5.4	-15.2
14	BUI Building	-0.5, -3.2, -6.1	Yes	4.9	-11.0

2. BANGKOK SUBSOILS AND EFFECTS OF DEEP WELL PUMPING

The city of Bangkok is situated about 40 km from the sea in the Chao Phraya Plain. The plain consists of a deep basin filled with sedimentary soil deposits which form alternate layers of clay, sand, gravel and clay. 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 thickness of the sediments ranges from 550 to 1000m or so. The detailed study carried out at the Asian Institute of Technology by Prof. Prinya Nutalya and his team (AIT, 1981) indicate that the aquifer system beneath the city is very complex (see Figure 1) and the available data suggest the existence of eight aquifers separated by layers of stiff and hard clays. The large scale exploitation of ground water by deep well pumping possible started as early as 1950 have resulted in large piezometric drawdown in the underlying aquifers which in turn have resulted in subsidence. The initial study of Bangkok subsidence was by the Department of Mineral Resources and also Cox (1968), Brand and Paveenchana (1971). However, the most comprehensive study was conducted by Prof. Prinya Nutalaya at the Asian Institute of Technology for a ten year period as a Project for the National Environmental Board (NEB) of Thailand and in co-operation with the Royal Thai survey Department (RTSD) and the Department of mineral Resources (DMR). In this detailed study, 26 observation stations were installed at various important locations in Bangkok, and four observation wells were also installed. Each subsidence measurement station has several settlement plates, compression indicators, benchmarks, automatic subsidence recorders and piezometers at various depths.

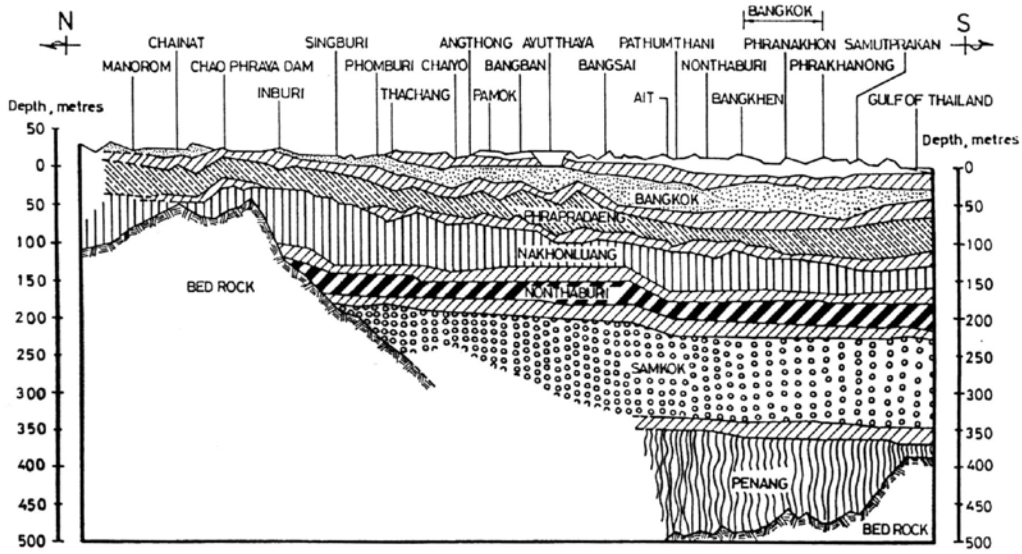

Figure 1: Hydrological profile of Bangkok aquifer system in the north-south direction (after Brand and Arbhabhirama, 1973)

According to Prinya *et al* (1989), the subsidence of Bangkok ranged from 0.20 m to 1.60m for the period 1933 to 1987 (see Figure 2). It was also noted that at certain critical locations, the elevations are already below mean sea level. The total area affected by subsidence is about 5000 square kilometres and the subsidence bowls keep changing depending on the trend of deep well pumping. He upper 10m of soft clay experienced only about 10 percent of the total subsidence possibly due to a combination of effects such as seasonal wetting and drying, self consolidation of the clays, external loads due to construction activities and possible traffic vibration effects. The major subsidence took place at a depth of 10-50 m and 50-200 m and this was related to the piezometric drawdown in the Bangkok, Phra Pradaeng, Nakhon Luang and Nonthaburi aquifers. At the New International Airport site, continuous monitoring of the piezometric draw down was carried out for a long time and these data are compiled by the AIT team of researchers during and prior to the 1995 study (see Figure 3). Consequently, the layers of soils below 6m or so are experiencing increases in effective stress at times at the order of 300 kN/m^2, which in turn cause the subsoil layers to consolidate resulting in subsidence. Figure 4 shows the compressibility characteristics of the clay layers below the Plain at the AIT campus (Jiann, 1977). The upper 10 to 20 m of the sub-soil is highly compressible clay with low shear strength. Since the land is low-

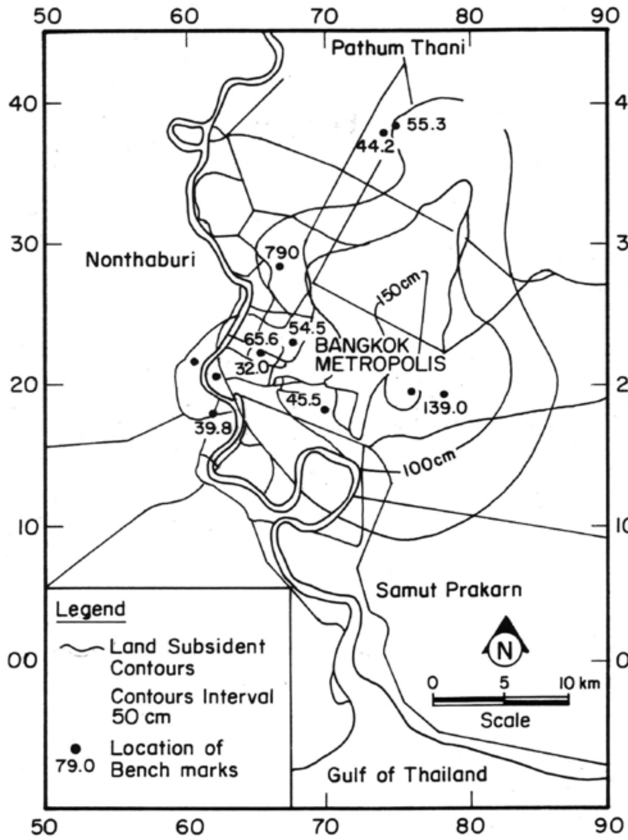

Figure 2: Land subsidence contours of Bangkok metropolitan area during 1978 to 1987

lying, most of the areas are prone to heavy flooding during the rainy season and most development projects are on filled ground to above the flood level.

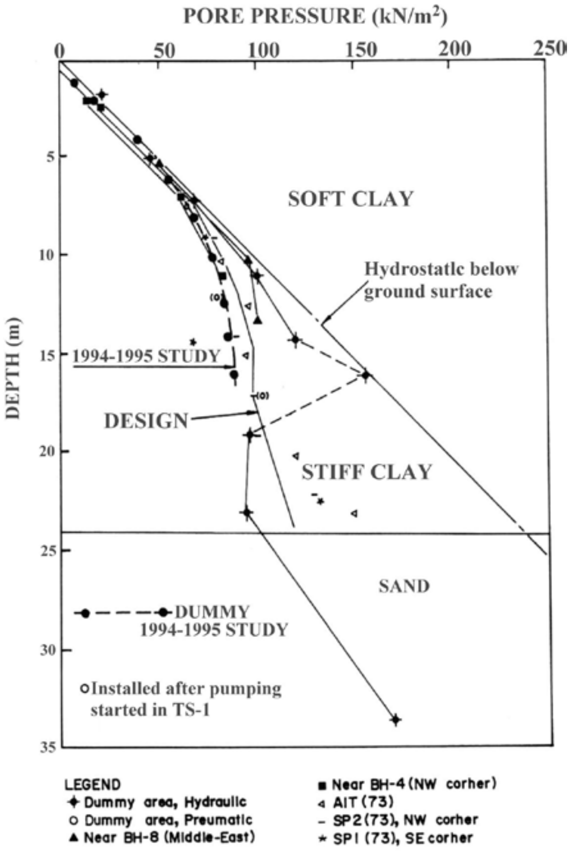

Figure 3: Variation of Piezometric draw down with depth

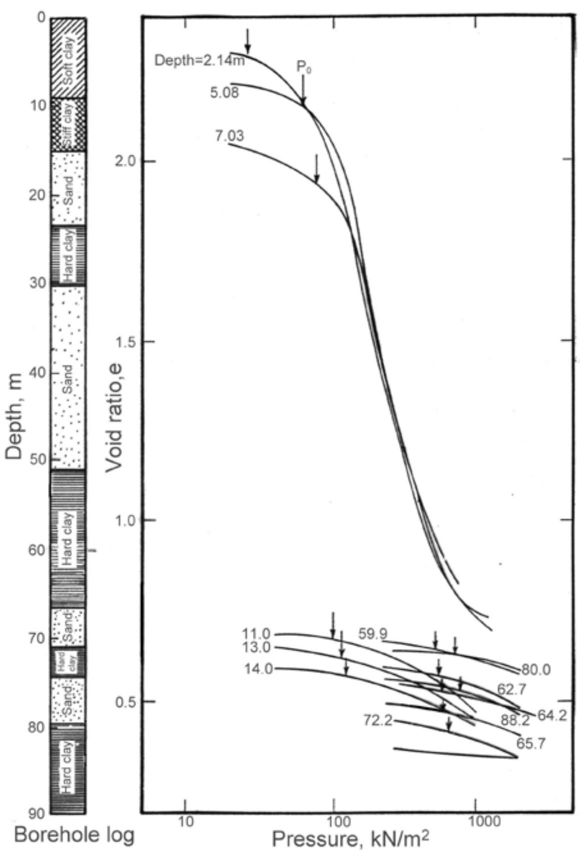

Figure 4: Compressibility characteristics of Bangkok clays (Jiann, 1977)

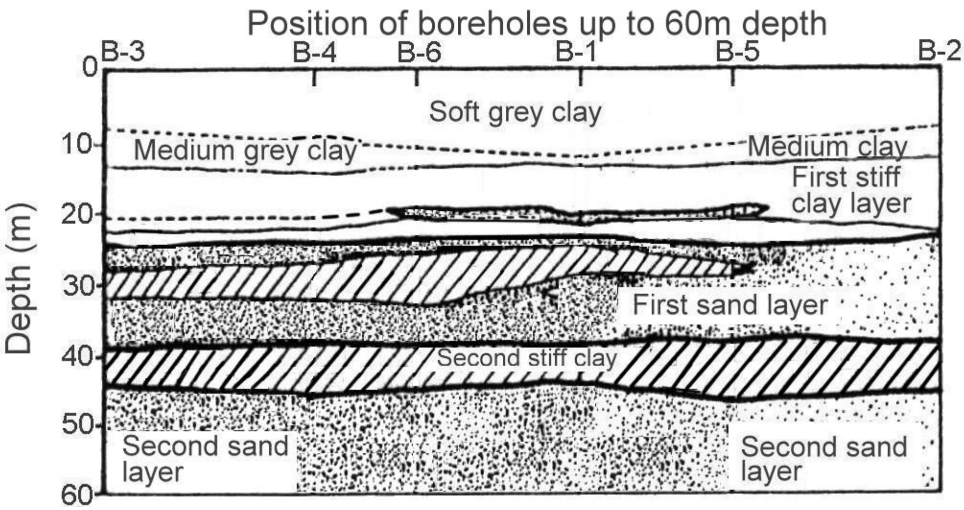

Figure 5: Typical deep soil profiles (based on Sambhandaraksa and Pitupakorn, 1985).

Typical deep soil profiles and shallow ones are shown in Figure 5, and the general soil properties are given in Table 2. For foundation purposes the sub-soil layers of interest are: Weathered clay is the uppermost layer, the thickness of which varies from 1 to 4 m and undrained shear strength 30 to 50 kN/m^2. 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/m^2. 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/m^2 and compressibility is very low. Below the stiff clay is the first sand layer and is generally found at a depth of 20 to 30 m. 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/m^2. 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 Figure 6. The ground water table is about a meter or so below the ground surface.

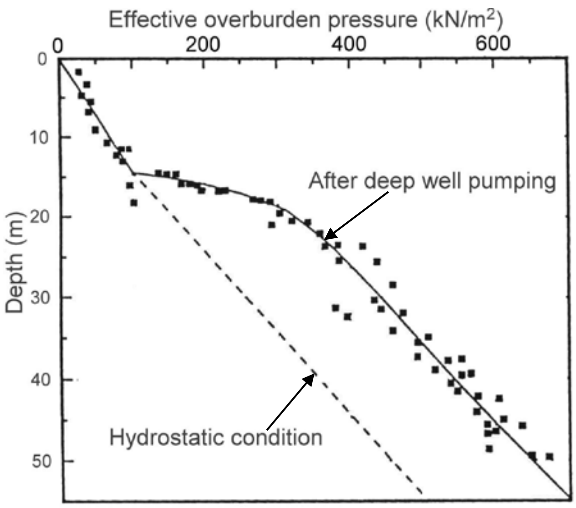

Figure 6: Variation of effective overburden pressure with depth

3. 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 Figure 5), which is located around 20 to 30m depth below the ground surface. Driven piles were normally designed using a total stress analysis. The undrained shear strength in the soft and medium stiff clays was 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).

Table 2: General properties of Bangkok subsoils.

Soil type	Depth (m)	w_n (%)	w_l (%)	w_p (%)	I_p (%)	γ□(kN/m^3)	e	G_s
Weathered clay	0-2	35-70	35-55	-	23-30	16-18	1.3-1.4	2.6-2.7
Soft clay	1-16	65-90	65-90	30-40	40-63	15-17	1.5-2.6	2.7
Stiff clay	10-25	24-34	40-75	20-28	18.50	19-20	0.65-0.95	2.7-2.8
First sand	14-38	17-25	-	-	-	18-21	0.7	2.7
Hard clay	24-43	24-43	30-35	55-69	31-44	18-20	0.8-0.95	2.7
Second sand	30-58	20	-	-	-	18-26	0.75	2.7

3.1 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, 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 Figure 7. These authors also presented a correlation of $\bar{\phi}$, the angle of friction with SPT values of N as shown in Figure 8. The unit skin friction in sand is taken as

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

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 3. Typical cone resistance and skin friction from CPT tests are shown in Figures 9 and 10.

Table 3: Skin friction and end bearing factors in CPT tests

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

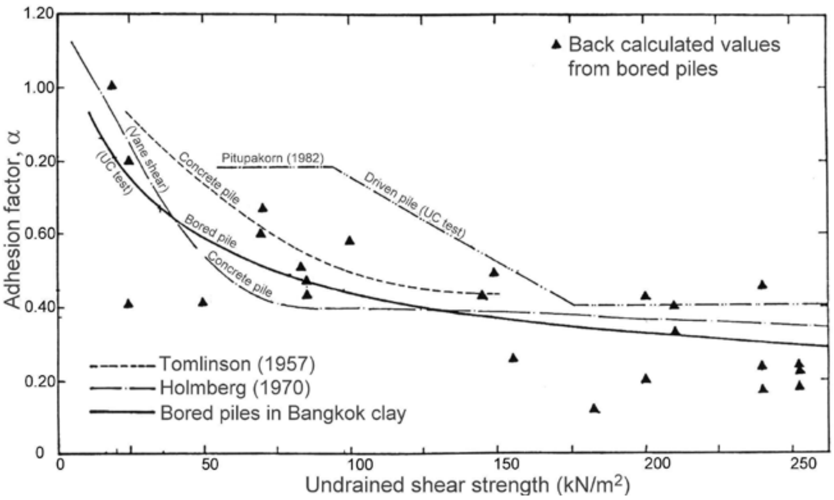

Figure 7: Adhesion factor for clays (based on Sambhandaraksa and Pitupakorn, 1985)

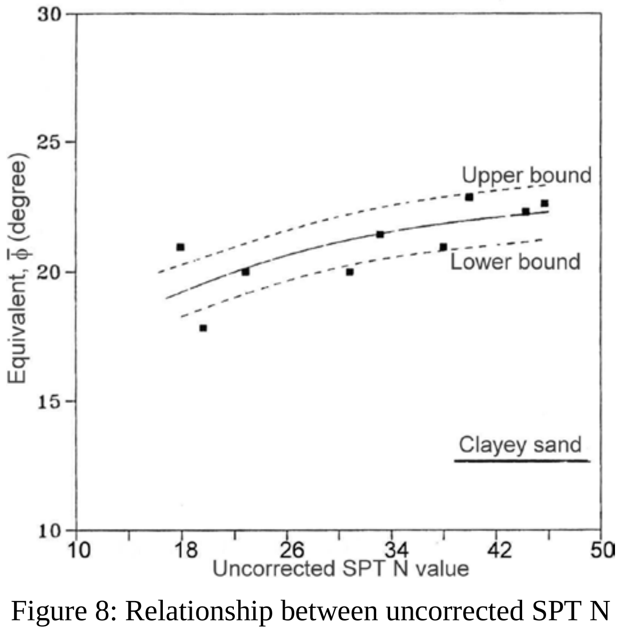

Figure 8: Relationship between uncorrected SPT N value and equivalent $\overline{\phi}$ of clayey sand (based on Sambhandaraksa and Pitupakorn, 1985)

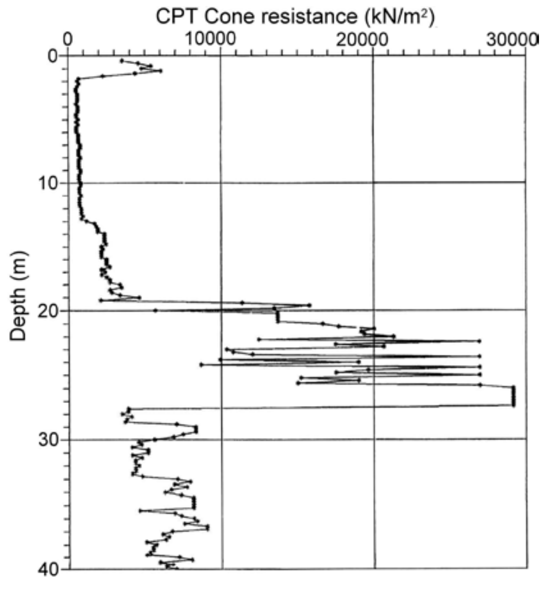

Figure 9: Variation of CPT cone resistance with depth

3.2 Effective stress analysis

The long-term behaviour 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 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 11 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).

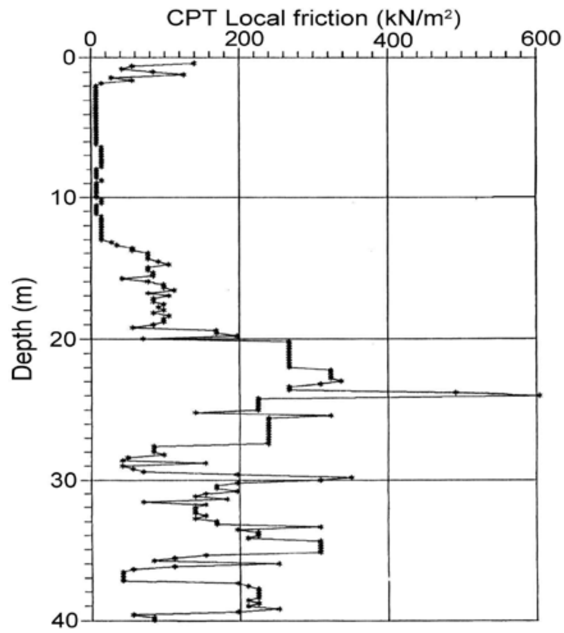

Figure 10: Variation of CPT local frictions with depth

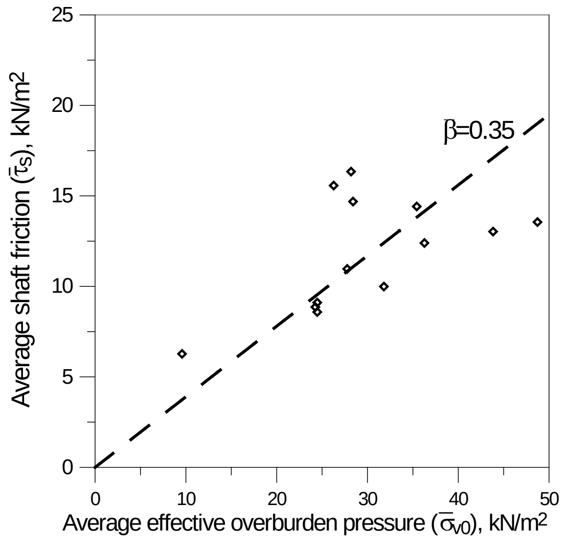

Figure 11: Estimation of $\bar{\beta}$ from pile load tests

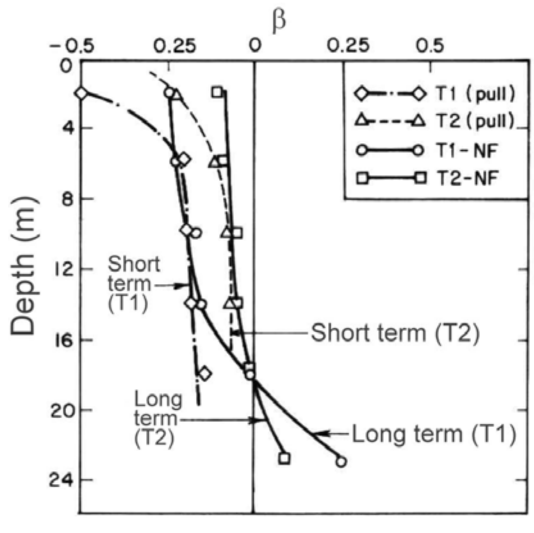

Figure 12: Variation of β-parameter with depths during negative skin friction

3.3 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, precast and spun concrete type. The outside and the inside diameters are 0.4 m and 0.25 m respectively. Each instrumented test pile was divided into six segments, five of which were 4 m 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 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 12 illustrates the β-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 $0.2\,\sigma'_{v0}$.

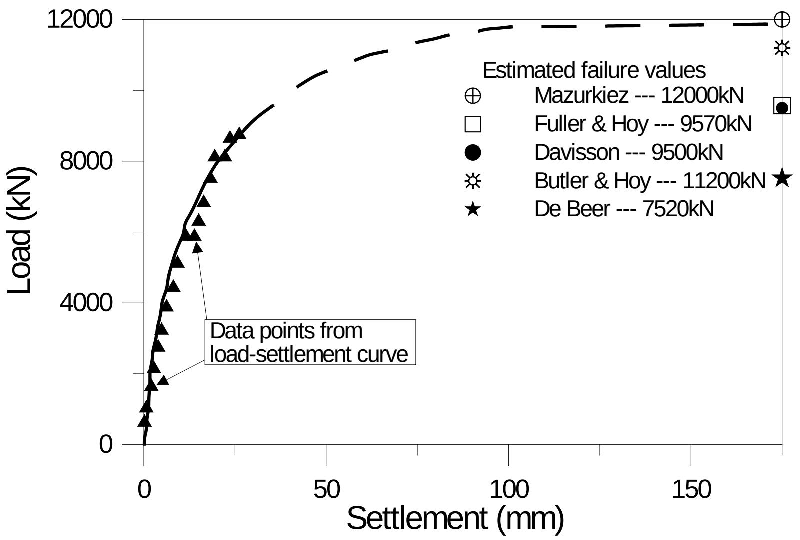

Figure 13: Load settlement characteristic of 0.8m diameter spun pile

4. 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 13 illustrates the use of these methods to compute the failure load. The Mazurkiwicz (1972) method is found to predict the capacity well.

5. 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 analysed. 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 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 14a to 14d 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 mobilisation of skin friction and end bearing from load tests show substantial scatter in the data points.

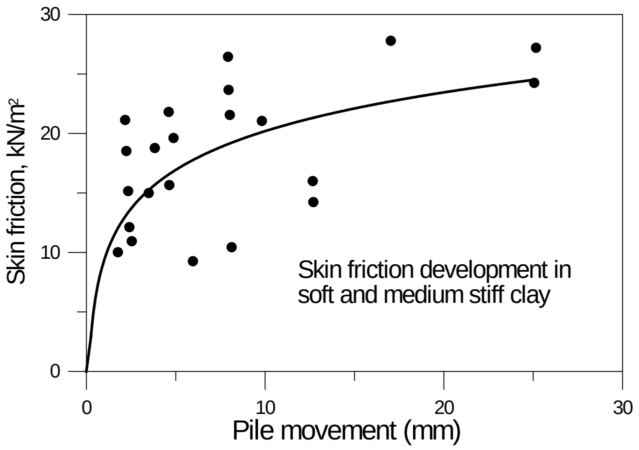

Figure 14a: Development of skin friction in soft and medium stiff clay with pile movement

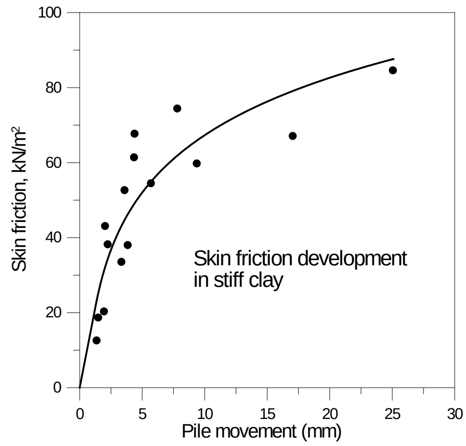

Figure 14b: Development of skin friction in stiff clay with pile movement

For the stiff Bangkok clay the undrained shear strength is about 200-250 kN/m^2 as obtained from unconfined compression tests and SPT correlations. The average skin friction is about 100kN/m^2; 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/m^2 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-analysed parameters indicated that, the adhesion factor (α) is higher for the shaft-grouted piles than for the non-grouted piles (see Figure 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 Figure 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.

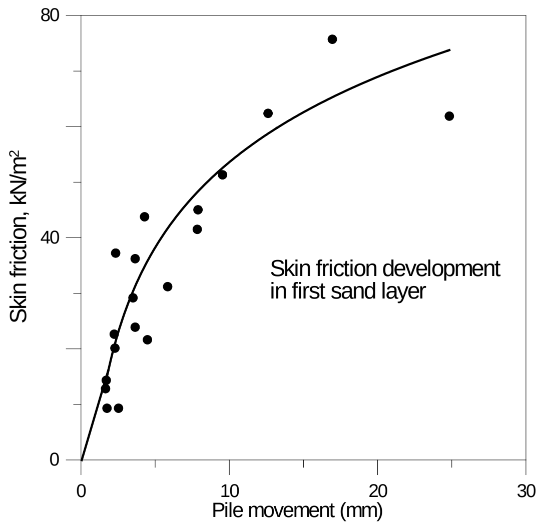

Figure 14c: Development of skin friction in first sand layer with pile movement

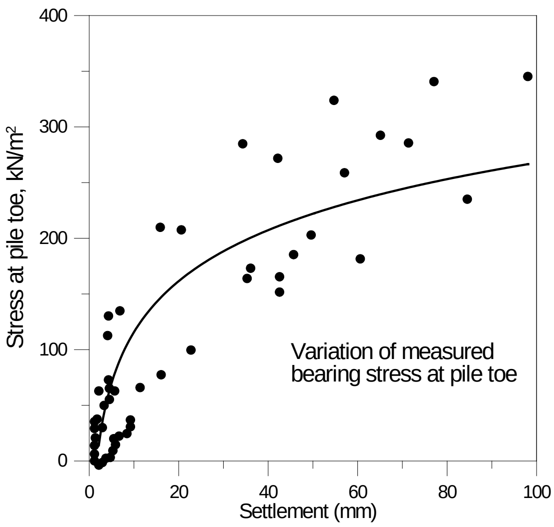

Figure 14d: Variation of measured bearing stress at the pile toe with settlement

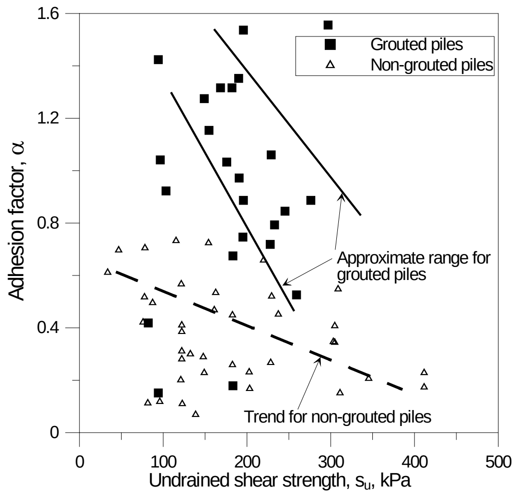

Figure 15: Relationship between adhesion factor (α) and undrained shear strength of non-grouted and shaft grouted bored piles

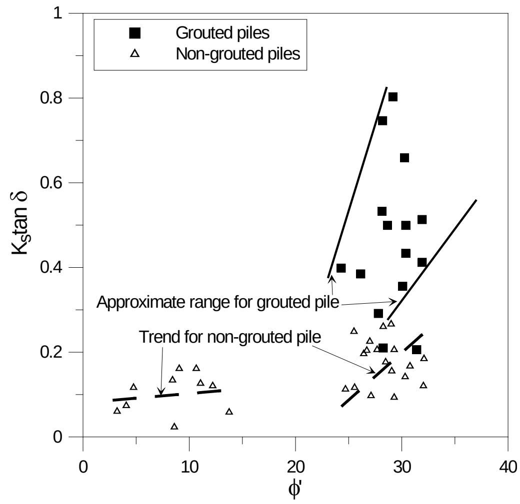

Figure 16: Relationship between $K_s$tan δ and φ' for non-grouted and shaft grouted bored piles

6. DEEP EXCAVATIONS

The construction control of deep excavations in Bangkok subsoils is purely based on the measured lateral movements adjacent to the support system used in the excavation methods. This paper relates to the back analysis of the deformations of deep excavations supported by diaphragm walls in Bangkok subsoils. Fourteen case studies involving deep excavations are presented here. Two types of back analysis are performed. In the first type, the relationship between the ratio of the maximum lateral wall movement to the depth of excavation against the basal heave factor of safety as proposed by Mana and Clough (1981) and the subsequent work of Clough *et al* (1989) was investigated. In the second type of analysis, computer softwares based on the finite element and finite difference methods were used with simple stress strain models for the prediction of the behaviour of one deep excavation. In all cases, back calculated soil parameters were evaluated for the use of each type of computer software for the design and construction control of diaphragm walled braced excavations in Bangkok subsoils.

7. OBSERVED CHARACTERISTICS OF WALL MOVEMENTS

For the diaphragm walls listed in Table 1, the maximum lateral wall movement with excavation depth is shown in Figure 1. The pattern and the shape of wall movement developing with increasing excavation depth are examined. The relationships between the ratios of the maximum lateral movement to the depth of excavation ($\delta_{h\max} / H$), and the factor of safety against basal heave failure (Mana and Clough, 1981), and the subsequent work of Clough *et al* (1989) which incorporates the system stiffness, were examined for their applicability to braced concrete diaphragm walls as used in Bangkok subsoils. Figure 17 shows the trend of the maximum lateral wall movement developing at various stages of excavation depth for all the projects. Generally, the magnitude of maximum lateral movement of the wall does not show a significant increase with increasing excavation depth. In terms of the normalized maximum lateral wall movement with excavation depth, an envelope can be drawn for prediction purposes in future work (as shown in Figure 18). These data indicated that $\delta_{h\max}$ of diaphragm walls can be controlled within 0.2 to 0.6 percent of excavation depth provided that the first stage movement in the cantilever mode is properly minimized. Wall movements in one case (Saxhorn Complex) were extra-ordinarily high because the cantilever mode excavation was made up to 3.5 m deep and there was a change in bracing design to entertain a request for a deeper final excavation depth. Embedment of the diaphragm wall was rather short in this case.

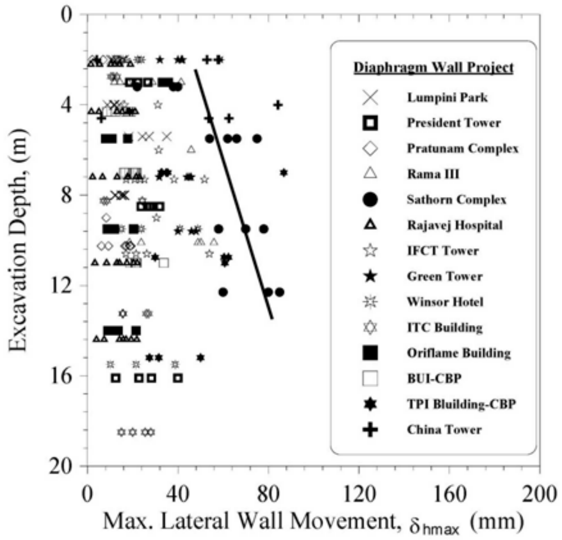

Figure 17: Variation of maximum lateral wall movements with excavation depth

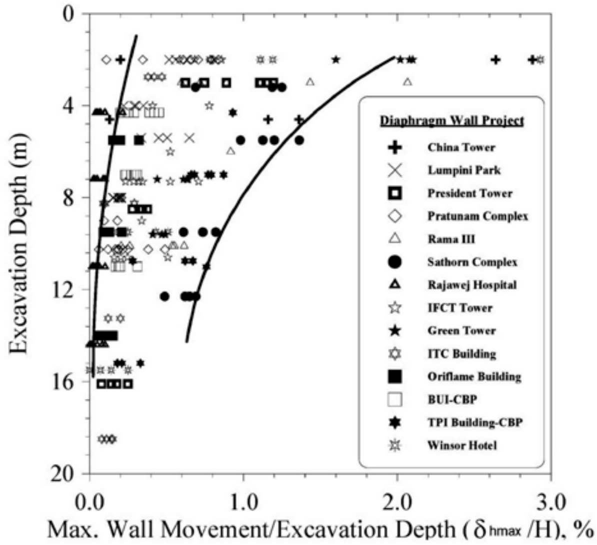

Figure 18: Variation of $\delta_{h\max} / H$ with excavation depth

The relationship between $\delta_{h\max} / H$ and the factor of safety against basal heave failure of the data presented in Figure 19 does not seem to follow the trend of the relationship as proposed by Mana and Clough (1981), which was actually developed based upon data from excavations with steel sheet piles or soldier piled walls (flexible type). In terms of the relationship between $\delta_{h\max} / H$, the system stiffness $\left(EI / \gamma_w I^4\right)$ and the factor of safety against basal heave failure (as proposed by Clough *et al*, 1989), the Bangkok case studies (as presented in Figure 20) do not support the validity of its applicability to the diaphragm walls, either as the data points are quite scattered and do not show any trend. It should be noticed that the vertical spacing of the struts used in the diaphragm walled excavations in Bangkok were rather high (see Table 1), in some case as high as 8-9 m.

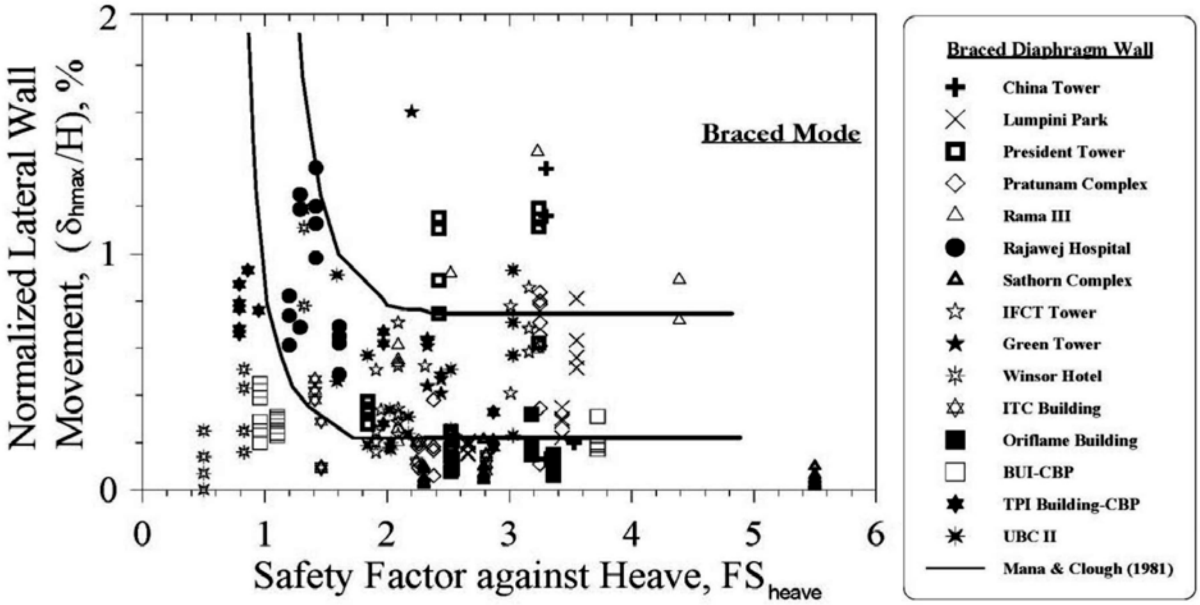

Figure 19: Relationship between $\delta_{h\max} / H$ and factor of safety against basal heave failure

8. NUMERICAL PREDICTION OF WALL MOVEMENTS

Complete information and measurements were available from a diaphragm wall braced excavation project (IFCT). The excavation was for the construction of 2-level basement of the 36-storey IFCT building. The excavation was rectangular in shape with an approximate dimension of 40m x 120m and the excavation depth varied from 8.9m to 13.7m (Figures 21a and 21b). The site was located in New Petchburi Road and was surrounded by multi-storey buildings on the eastern boundary, a single storey boundary on the western boundary, a main canal on the southern boundary and the road on the northern boundary. The excavation was made almost to the full area. The subsoil profile shown in Figure 22 consisted of 18m thick soft to medium stiff clay, underlie by a stiff clay layer. The diaphragm wall was 800mm thick with the tip extending to 20.6m depth. Two levels of pre-load struts (elevation –1.5m and -6.8m) were adopted (see Figure 21b). The ground settlements near the walls were monitored with settlement points. The excavation and the installation of struts were closely monitored and controlled. Strut loads were monitored at one section (close to inclinometer 6) for the verification of the design. Predictions of the wall movement and the strut loads were made prior to the excavation using two well known FEM beam on elasto-plastic foundation programs which were normally used for braced wall design (WALLOP and PAROI2). Both programs utilise the spring elements to model the soils. The predictions were made independently using different input parameters (see Table 4).

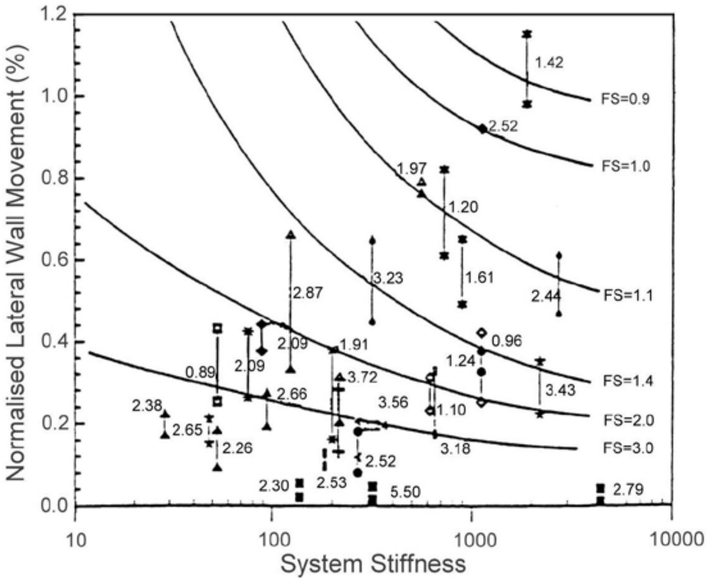

Figure 20: Variation of $\delta_{h\max} / H$ with the system stiffness $\left(EI / \gamma_w I^4\right)$ and the factor of safety against basal heave

8.1 Observed ground movement

Figure 23 summarises the actual lateral movements as measured with the inclinometers of the walls at various stages of excavation. In the first stage of excavation down to 2m depth, when the wall behaved in a cantilever mode, $\delta_{h\max}$ of 12 to 16mm occurred at the top of the wall. Toe fixity of the wall was observed below a depth of 17-18m, below which the wall was embedded in the stiff clay layer. In the subsequent stages of excavation the wall movement followed the braced excavation mode in which the maximum lateral movement near the base of the excavation as the upper part of the walls were restrained by struts installed in the early stages. In the embedded section of the wall, toe fixity no longer existed as the wall rotated about its bottom tip. The measured lateral movements showed significant variations from location to location and the magnitude of $\delta_{h\max}$ ranged from 20 mm to 54 mm, when the excavation reached 10.6 m. The causes of the variation were attributed to be partly due to the effect of the existing deep piled foundation of the multi-storey buildings next to the excavation area as well as due to the effect of the large diameter bored piles within the excavation area. The existence of the main canal at a close proximity (7m away) along the south boundary did not show any obvious adverse effect on the movements of the wall in the area.

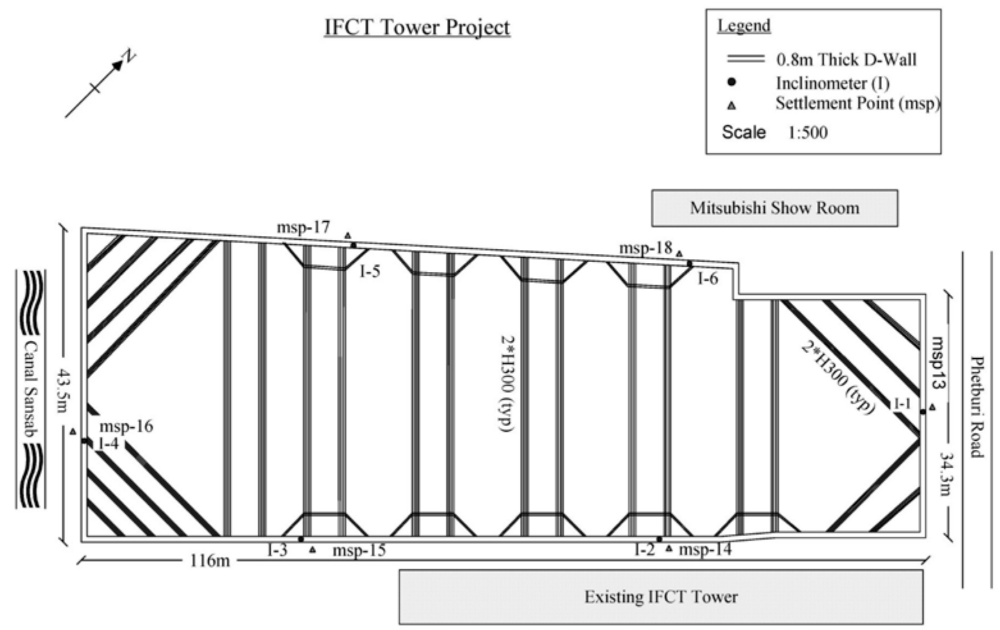

Figure 21a: Layout of excavation, struts and instrumentation (IFCT project)

8.2 Method of analysis

For comparison with the pre-excavation predictions made by the two beams on elasto-plastic analysis using PAROI2 and WALLAP codes, additional predictions were made using the well known PLAXIS and CRISP programs. However the predictions with the latter two were made after the completion of the excavation. The PAROI2 treat the soil as spring elements, whose stiffness (*k*) were estimated from the undrained shear strength of the clay using a relationship E_u = 135 S_u (see Table 4). In the case of WALLAP, for soft clay E_u = 250 S_u was adopted. Both analysis considered elasto-plastic sporing to model the soil behaviour. The limiting pressure is either the active pressure or passive pressure with the adopted factor of safety. In the CRISP90 and PLAXIS programs the soils are modelled as continuum media. The yielding was assumed to be according to the Mohr-Coulomb criterion. Undrained analysis was considered in this analysis. Interface slip elements between soil elements and beam elements of the walls were included in both analyses. The soil moduli of elasticity (E_u) initially adopted in both CRISP and PLAXIS were equivalent values corresponding to the spring stiffness values used in PAROI2 and WALLAP analyses (see Table 4).

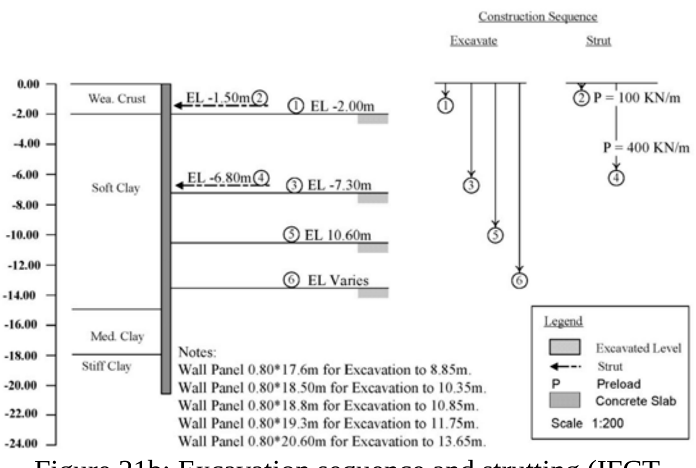

Figure 21b: Excavation sequence and strutting (IFCT project)

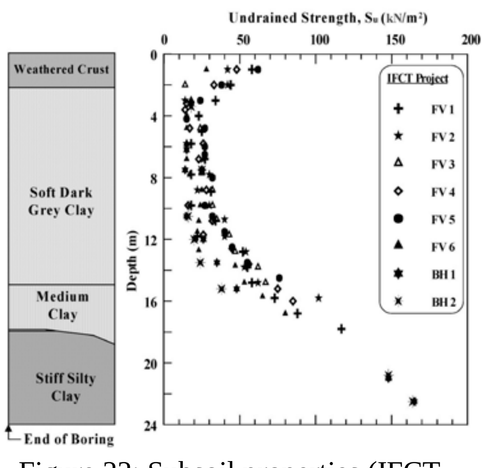

Figure 22: Subsoil properties (IFCT project)

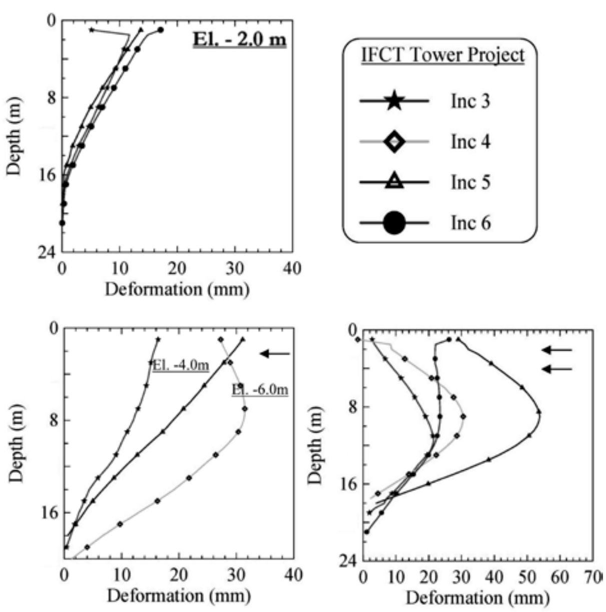

Figure 23: Observed lateral wall movements (IFCT project)

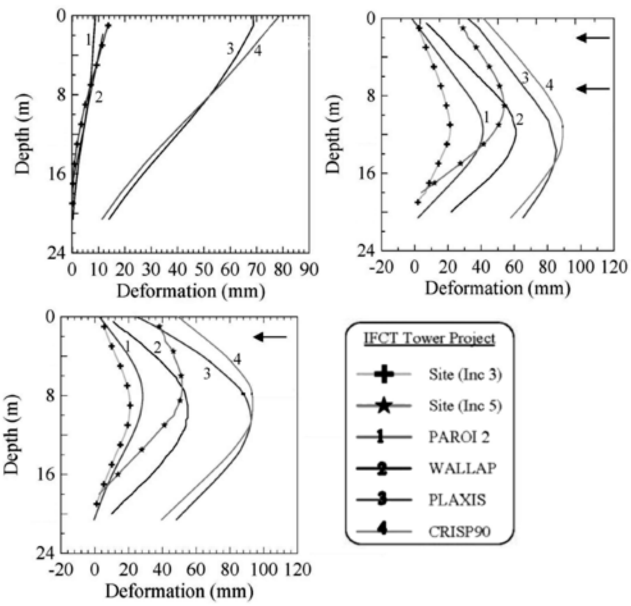

Figure 24: Comparison of predicted and observed wall movements (IFCT project)

8.3 Predictions by the analysis

Figure 24 shows the predictions of the lateral wall movement at three excavation stages given by the four analyses in comparison with the actual observations. It can be seen that the CRISP90 and PLAXIS analyses based on a continuum approach make over-prediction especially in the early stages of the excavation, while the PAROI2 and WALLAP based on beam- spring analysis yielded good predictions. Further it was also noticed that both PAROI2 and WALLAP analyses showed fixity of the walls well in line with the actual observations. The maximum wall movement was also over-predicted by the CRISP90 and PLAXIS analyses while the PAROI2 and WALLAP made reasonably well predictions. In order for both CRISP90 and PLAXIS programs to give reasonable predictions, back analysis of the modulus values of the soils were made and tabulated in Table 5. The CRISP90a adopted E_u = 350 S_u for soft clay while CRISP90b and PLAXIS adopted E_u = 500 S_u. For the stiff clay E_u = 1200 S_u needed to be adopted. The predictions using these modified modulus values are shown in Figure 25. The results of the back-analysis indicated the necessity to use increased modulus values of soils in excavation works. This finding was similar to those reported by Heluin (1991) and others.

Table 4: Input parameters adopted in numerical analysis

(a) Soil properties for PAROI2

Soil layers	Depth (m)	Eu (kN/m^2)	Su (kN/m^2)	ϕ_u	γ_t(kN/m^3)	Ko
Weathered clay	0-1	135Su	33.0	0	15.5	0.50
Weathered clay	1-2	135Su	27.5	0	15.5	0.50
Int. soft clay	2-4	135Su	20.3	0	15.5	0.87
Soft clay	4-15	135Su	15+11.3z	0	15.6	0.87
Med. Clay	15-18	135Su	56+8.3z	0	20.0	0.50
Stiff clay	18-24	135Su	105.0	0	20.0	0.50
Very stiff clay	24-30	135Su	135.0	0	20.0	0.50
1st sand	30-36	65,000	0.0	30^o	20.0	0.75

(b) Soil properties for WALLAP

Soil layers	Depth (m)	Eu (kN/m^2)	Su (kN/m^2)	ϕ_u	γ_t(kN/m^3)	Ko
Weathered clay	0-1	208Su	28.8	0	17.2	0.60
Weathered clay	1-2	208Su	26.5	0	17.2	0.60

Int. soft clay	2-4	161Su	15.5	0	15.1	0.88
Soft clay	4-15	250Su	16.5	0	15.7	0.88
Med. Clay	15-18	296Su	113.0	0	17.1	0.60
Stiff clay	18-24	300Su	180.0	0	19.2	0.60
Very stiff clay	24-30	300Su	200.0	0	20.0	0.60
1st sand	30-36	65,000	0.0	30	20.0	0.75

(c) Properties of wall and struts

Structure elements	E (kN/m^2)	I (m^4/m)	A (m^2)	EI (kNm^2/m)	W (kN/m^2)
Diaphragm wall (0.8m thick)	2.00E+07	4.267E-02	0.8	8.53E+05	23.5
H pile (350*350*136 kg/m)	2.00E+08		1.739E-2		υ=0.25
H pile (400*400*172 kg/m)	2.10E+08		2.187E-2		υ=0.25

(d) Properties of interface element

Interface element	Cu (kN/m^2)	ϕ_u	K_n(kN/m^3)	K_s(kN/m^3)	K_{sres}(kN/m^3)	Thick, t (m)
Slip element	12.5	12.5^o	46447	1000	100	4.37E-3

Table 5: Back-analysis input soil parameters for FEM analysis

Soil layers	Depth (m)	Eu (kN/m^2)	Su (kN/m^2)	ϕ_u	γ_t(kN/m^3)	Ko
Weathered clay	0-1	700Su	80.0	0	17.5	0.62
Weathered clay	1-2	700Su	80.0	0	18.0	0.62
Int. soft clay	2-4	500Su	35.0	0	17.0	0.62
Soft clay	4-15	500Su	17.5+3.3z	0	16.0	0.67
Med. Clay	15-18	700Su	65.0+8.5z	0	17.5	0.65
Stiff clay	18-24	1200Su	110.0+1z	0	18.5	0.62
Very stiff clay	24-30	1200Su	135.0	0	20.0	0.62
1st sand	30-36	65,000	1.0	30^o	20.0	0.75

8.4 Effect of foundation piles

The effect of foundation piles on the passive resistance to the wall movements for typical Bangkok soils was studied by conducting a parametric study using CRISP90. Bored piles of 1.5 m diameter were simulated. The bored pile was treated in the analysis using the equivalent stiffness method proposed by Lee *et al* (1989). The effect of the foundation pile on the wall movement is presented in the summary plot in Figure 26, which shows $\delta_{h\max}$ for various distances of the piles from the wall at various stages of excavation. The analysis shows that the pile effect in reducing the wall movement is more pronounced in the early stages of excavation. The effect diminishes as the pile is located more than 5 m away from the wall in the final excavation stage.

8.5 Strut loads and earth pressure on walls

The monitored strut loads using hydraulic jacks at one section of IFCT 9 close to Inclinometer 6 revealed that, the actual strut loads in both strut levels were very close to the predictions given by PAROI2 analysis 1750kN and 4950kN as compared to 2400kN and 4800kN for the upper and lower strut levels respectively. The back calculated rectangular earth pressures on the wall are shown in Figure 27. Also, shown in the figure are the earth pressures given by CRISP90 and PAROI2 analyses and the apparent earth pressure (AEP) envelope $(\gamma H - 4c)$ (Peck, 1969). The back calculated pressures are close to CRISP90 values as well as those due to Peck method.

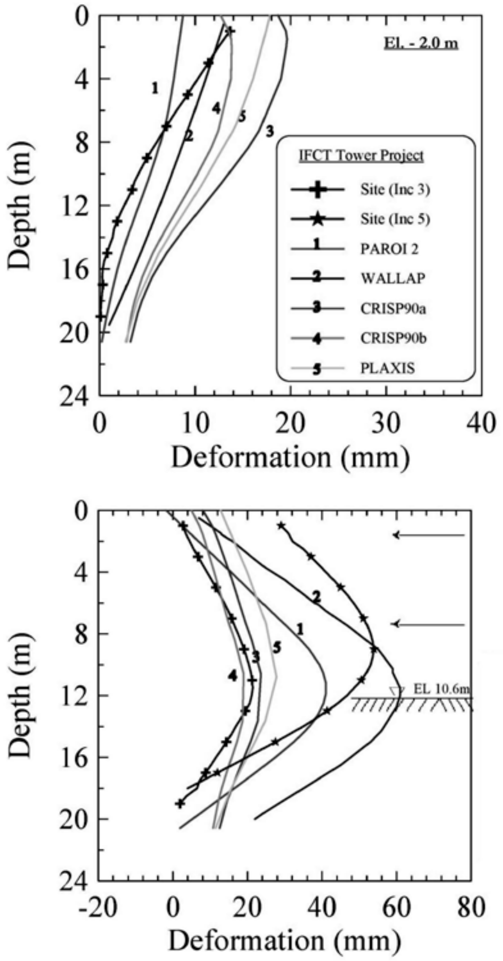

Figure 25: Observed and FEM prediction of wall movement using back-analyse modulus

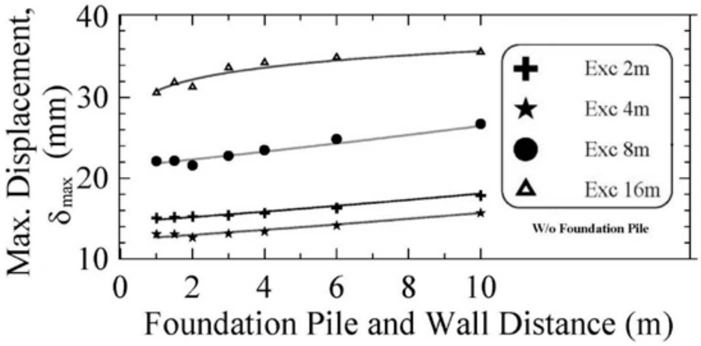

Figure 26: Effect of foundation piles on maximum lateral movement of the wall

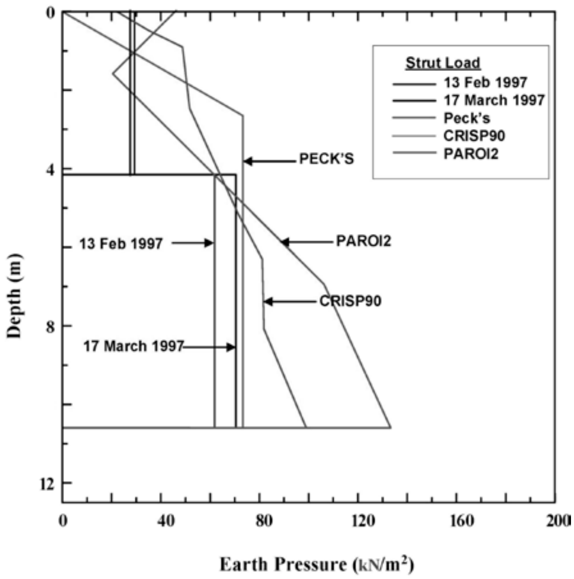

Figure 27: Measured and predicted earth pressure patterns on diaphragm wall

8.6 Ground settlement

The ground settlement adjacent to the wall predicted by the FEM analysis is shown in Figure 28a. The FEM analysis gave too wide a settlement zone as expected. The observed ground movements within 1.5m from the wall were smaller than 0.5 percent of the excavation depth (Figure 28b).

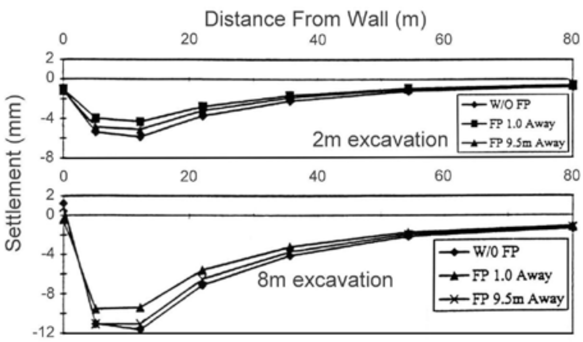

Figure 28a: Predicted ground settlement adjacent to the wall

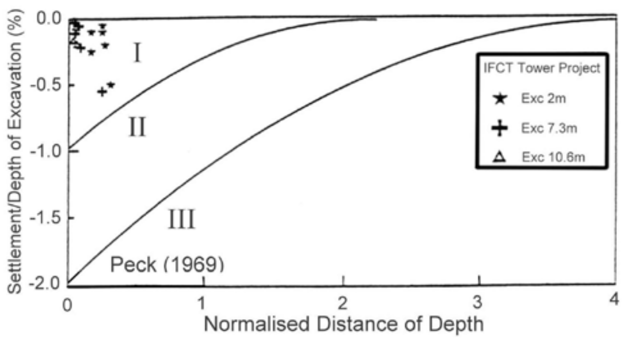

Figure 28b: Ground settlements (IFCT project)

9. CONCLUSIONS

The paper summarizes the role of back-analysis and interpretations of pile load tests, and diaphragm wall supported deep excavations for tall buildings in Bangkok sub-soils. 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.

The construction control of deep excavations in Bangkok subsoils is purely based on the measured lateral movements adjacent to the support system as used in the excavation methods. The main conclusions that can be drawn from the study on the deformation of deep excavations are that, the maximum lateral movements of braced diaphragm walls in the Bangkok subsoils are in the range of 0.2 to 0.6 percent. If the ground movement in the cantilever mode of the excavation support is minimised then the overall lateral movement can be kept to a minimum. No clear relationship can be found for the ratio of the maximum lateral movement to the depth of excavation and the factor of safety against basal heave failure. In the computer softwares used, the beam elasto-plastic spring analyses using an undrained modulus of 135-250 times the undrained strength of the clays yielded good prediction of the observed performance. However, with the use of the FEM analysis and the continuum approach the corresponding undrained modulus need to be in the range of 500- 1200 times the undrained strength.

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