PROCEEDINGS OF THE 3RD INTERNATIONAL GEOTECHNICAL SEMINAR ON DEEP FOUNDATIONS ON BORED AND AUGER PILES/ GHENT/ BELGIUM/19-21 OCTOBER 1998

Deep Foundations on Bored and Auger Piles BAP III

Edited by

W.F.VAN IMPE & W.HAEGEMAN

Laboratory of Soil Mechanics, Ghent University, Belgium

Large diameter bored piles in multi-layered soils of Bangkok

N. Thasnanipan, P. Tangseng & M. A. Anwar

Seafco Company Limited Bangkok, Thailand

A.A. BALKEMA / ROTTERDAM / BROOKFIELD / 1998

Deep Foundations on Bored and Auger Piles, Vam Impe & Haegeman (eds) © 1998 Balkema, Rotterdam, ISBN 90 5809 022 1

Large diameter bored piles in multi-layered soils of Bangkok

N. Thasnanipan, P. Tanseng, M. A. Anwar
Seafco Co., Ltd., Bangkok, Thailand

ABSTRACT: Static pile load test results of ten large diameter bored piles founded in the multi-layered soils of Bangkok are discussed in this paper. All piles were instrumented with vibrating wire strain gauges (VWSG) and telltale extensometer rods. Range of shaft friction and end bearing values mobilized in different soil horizons are reported. Progressive type of failure mechanism which is peculiar with the long piles in multi-layered soils, and found to be initiated from the soil layers present at the middle reach of pile shaft and then extending towards the top and bottom layers, is also presented. Brittle type of failure observed in some sand and very stiff to hard clay layers is also discussed. Settlements at design loads calculated from global factor of safety concept and limit load concept are also compared. Additionally, back calculated α and β values are also compared with the previously recommended values for similar soils.

1 INTRODUCTION

Bored cast in situ piles are extensively used as a foundation element in Bangkok due to the requirements of huge loads to be transferred and limitations of using driven piles like limited capacities, associated soil movements and some unwanted environmental effects. Diameters of these piles normally fall in the range of 0.80 to 1.50 m and toe depths down to 60m from ground level are quite common. Soil profiles of plain of Bangkok are always evident of the presence of marine Bangkok Soft Clay (BSC) at the top changing to medium at about 15 to 18m. First sand layer is usually 5 to 10m thick and found at 25m to 30m depth, below is a series of stiff to hard clay and medium to very dense silty sand layers (Ref. to the Figs. 5 to 14 for typical soil profiles). Actual pore pressure conditions in the upper BSC are hydrostatic from circa 1m below ground level. Then the hydrostatic conditions changes to piezometric draw down near the bottom level of BSC. Piezometric draw down resulted in increased effective overburden pressure of about 20 ton/m^2 in the first sand layer and below as shown in Figure 1. The under drainage in the Bangkok soils is attributed to the deep well pumping in the area. According to the latest reports, piezometric draw down conditions have started recovering since last few years due to the control over deep well pumping and shifting of industrial areas away from Bangkok metropolitan.

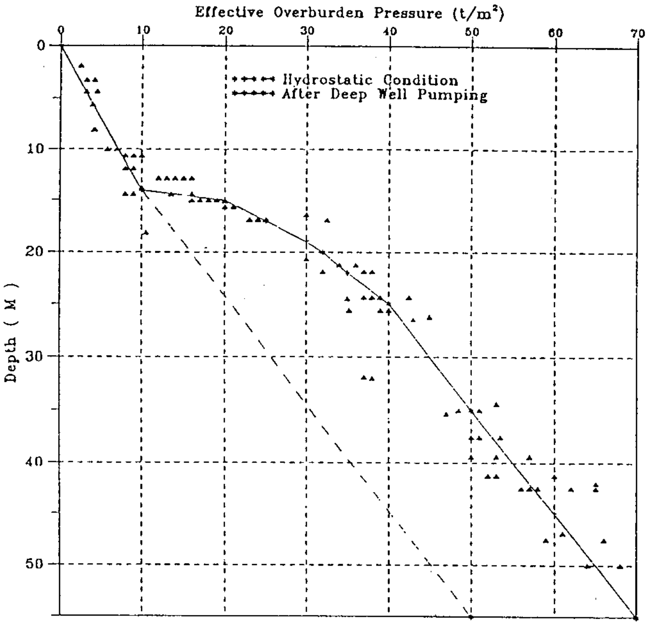

Figure 1. Variation of effective overburden pressure with depth, (after KERDSUWAN, 1984)

2 METHOD OF CONSTRUCTION

Bentonite slurry in conjunction with rotary bucket is the normal drilling procedure of pile construction when piles are to be founded in the first sand or below. Top 15 to 18m depth of soft to very soft clay is almost always temporarily cased to assure the stability of borehole. Firstly, auger is used to drill within the temporary casing followed by rotary bucket with bentonite slurry down to final depth of excavation. Special cleaning bucket or air lift technique is normally applied to clean the borehole base of any congregated sediments before lowering the reinforcement cage which is followed by tremie concreting. Bentonite slurry viscosity is normally maintained with in the range of 30 to 50 seconds (Marsh cone viscosity) and construction time, starting from the casing driving till the completion of concreting, usually fall in the range of 10 to 20 hours excluding some accidental delays due to equipment break down, unavailability of ready mixed concrete or similar reasons.

3 DESIGN PARAMETERS

A combination of total stress method for clay layers and effective stress method for sand layers is normally adopted due to its simplicity and limitations regarding unavailability of reliable effective stress parameters. There is no standard criteria of acceptable settlement for these piles, but a limit of 5mm at working loads is normally accepted for high rise buildings.

3.1 *Skin friction*

Skin friction capacity of clay layers is estimated as $f_s = \alpha.Cu$, with adhesion factor α varying from 0.9 to 0.3 depending upon the undrained shear strength of clay layers. Undrained shear strength Cu of shallow clay layers is usually determined in the laboratory with unconfined compression test while for deeper stiff to hard clay layers Cu is indirectly estimated from Standard Penetration Test (SPT-N) data as Cu = C_1.SPT-N (ton/m^2), with C_1 equal to 0.674 and 0.507 for high and low plasticity clays respectively. C_1 values mentioned here are based on the statistical analysis carried out by Pitupakorn, 1985. Skin friction capacities for the sand layers are calculated as $f_s = \sigma v'.Ks.\tan\delta$, with coeficient of horizontal earth pressure Ks equal to 0.7 and δ equal to 0.75ϕ. Angle of internal friction ϕ is also estimated from SPT-N values by first correcting the N values for overburden correction (Bowels, 1988). Some designers also use an equivalent term of β instead of $Ks.\tan\delta$. It must be noted that the empirical parameters discussed here are based on the effective overburden pressure $\sigma v'$ which is calculated with the assumption of hydrostatic conditions from the ground level, ignoring the piezometric draw down conditions below BSC shown in Figure 1.

3.2 *End bearing*

Ultimate end bearing capacity for the sand layers is calculated as Nq.$\sigma v'$ with bearing capacity factor Nq in the range of 5 to 20 depending upon the relative density of the sand layers. A limit of 500 ton/m^2 for maximum end bearing is normally used for piles seated in the second sand layer. For clay layers ultimate end bearing is estimated as Nc.Cu with bearing capacity factor Nc equal to 9. Global factor of safety of 2.5 on the accumulated ultimate skin friction and end bearing is normally used to calculate the safe working loads.

Table 1. Principal data of test piles with interpreted failure loads.

Pile No.	Dimensions (Dia. x Depth)m	Qd1 Design Load (ton)	Total Mobilized Skin Friction (ton)	Estimated Skin Friction (ton)	Mobil/Estim Skin Friction	Failure Load (ton) De'Beer	Davisson	Brinch-Hansen's	Chin-Konder's	Mazurkiewicz's	Average Failure Load Qu ave	Qd2= Qu ave/2.5	Qd2/Qd1	SETTLEMENT(mm) AT Qd1	1.5xQd1	2.0xQd1	QL Limit load at a settlement of 1% of Dia. (ton)	Qd3 Design load '=QL/1.5 (ton)	Gross settlement at Qd3 (mm)	Difference of Qd3 as compared to original Qd1 (%)
1	2	3	4	5	6=(4/5)	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
TP-1	ϕ1.2x57.10	650	1582	1230	1.29			1435	1715		1575	630	0.97	6.8	13.2	35.5	975	650	6.8	0.0
TP-2	ϕ1.2x46.25	650	1929	930	2.07			2815	2017		2416	966	1.49	5.5	9.3	15.0	1140	760	5.5	16.9
TP-3	ϕ1.0x46.51	500	1118	800	1.40			1477	1651	1544	1557	623	1.25	4.5	8.2	12.3	920	610	5.5	22.0
TP-4	ϕ1.0x49.47	450	970	930	1.04			1172	1270	1138	1193	477	1.06	6.3	12.8	40.5	650	430	5.5	-4.4
TP-5	ϕ1.0x43.00	450	1106	690	1.60			1399	1642	1347	1463	585	1.30	4.5	11.1	34.5	675	450	4.5	0.0
TP-6	ϕ1.0x41.00	360	641	620	1.03	759	997				878	351	0.98	3.2	5.0	7.2	830	550	5.0	52.8
TP-7	ϕ1.2x43.6	450	1499	850	1.76	891	1390				1141	456	1.01	2.5	4.2	5.4	1150	770	4.5	71.1
TP-8	ϕ1.2x43.5	450	519	640	0.81			1012	1180	978	1057	423	0.94	3.3	5.9	48.5	850	570	4.7	26.7
TP-9	ϕ1.0x43.5	400	701	530	1.32			867	946	975	929.3	372	0.93	4.0	5.2	36.5	750	500	4.9	25.0
TP-10	ϕ1.2x54.00	500	799	1100	0.73									3.5	6.4	16.3	1000	670	5.0	34.0

4 TEST PILES

Test piles discussed in this paper were all constructed following the procedure discussed in section 2. All piles were instrumented with VWSG at five to seven different levels to estimate the shaft friction transferred to different soil horizons. One set of telltale extensometer rods was also used near the tip of each test pile to measure the elastic shortening of the pile shaft and finally pile base movement at different stages of load testing. Static maintained load testing method was used for all piles. Test loads were applied using a system of hydraulic jacks against the reaction frame of steel girders fixed against anchored reaction piles. Normally three cycles of loading are applied with first cycle up to the design load Q_d and maintained for 12 hours followed by second cycle of loading up to 2 times of Q_d and maintained for 24 hours. Third cycle of loading is applied up to 2.5 times Q_d or maximum pre-decided test load. Some times a fourth quick loading cycle up to the maximum test load is also applied and maintained for two hours. All test piles except TP10 were tested to well above the expected failure loads with the concept of sufficiently mobilizing the end bearing.

5 TEST LOAD RESULTS

Test load results with pile dimensions are summarized in Table 1 and Figures 5 to 14. It is quite evident that the safe design loads (Table 1, col.13) calculated from the average of failure loads, interpreted by different available methods (cols.7-11), are in reasonable agreement with the estimated design loads (col.3). But the settlements at the design loads (col.15) have a wide scatter between 2.5mm and 6.8mm. So instead of computing failure loads from methods mentioned in Cols. 7-11, if we use the limit load concept at settlement of 1% of the diameter (col. 18) and then use a partial FOS of 1.5 to calculate design loads (col.19), resulted settlements at this design load (col. 20) will be quite uniform and within the limits of allowable settlement at design loads for high rise buildings. Global FOS against overall ultimate failure will still be in excess of 2.5 against the ultimate failure of the pile which still have not been reached even at large pile head movements (Buttling, 1992). This can also be confirmed from the maximum end bearing values mobilized under maximum test loads i.e. only 25% of the ultimate end bearing is mobilized (ref. Section 5.2)

5.1 S*kin friction*

Actually mobilized shaft friction capacities (ref. Table 1, col.4) are on the average by 50% higher than the estimated skin friction values with the exception of piles TP8 and TP10 who have 19% and 27% less capacity respectively. It must be noted that TP10 was not tested to failure so ultimate shaft capacity was not yet mobilized.

Unit skin friction values mobilized in different soil layers are given in Table 2. It must be noted that the unit skin friction values reported here are the actual maximum mobilized capacities only. Since piles were not instrumented to measure the corresponding displacements at the interface of each soil horizon, minimum values are not reflecting the ultimate unit friction values.

Table 2. Skin friction values mobilized in different soil horizons and corresponding α values.

Soil Type	SPT-N	Ave. Cu	Skin Friction Mobilized (ton/m^2)		α (Ave.)
			Average	Maximum	
Soft to medium Clay	2-12	4.7	6.0	13.9	1.3
Stiff to very stiff Clay	20-35	14.0	7.5	18.8	0.54
Hard Clay	40-46	21.5	7.3	24.6	0.34
First silty Sand	18-68	-	11.2	19.9	-
Second silty Sand	20->100	-	15.6	25.1	-

Average skin friction values mobilized in the soft to medium clay layers seems to be even higher than average Cu of these layers, most likely reasons for this are: 1) actual diameter in the soft to medium clay layer is slightly higher than the nominal diameter of the pile due to the use of temporary casing; 2) actual Cu of the top few meters of weathered crust is higher than the Cu used in the analysis. Values of α reported in Table 2 are quite conforming to the previous recommendations like Ng, (1983) and Tomlinson (1995). Skin friction values mobilized in the very stiff to hard clay layers are relatively low, since at failure loads considerable portion of the pile head settlement is absorbed by the long pile shaft itself which reduces the relative pile/soil interface movement near the deeper soil layers which results in partial mobilization of skin friction capacity of these soil horizon. In other words, in case of long piles the geotechnical capacity of deeper soil layers may not be fully exploited before they are declared to be failed due to the excessive pile head movement, same is the case for end bearing of these piles. Average values of unit skin friction mobilized in the first sand and second sand layers are 11.2 ton/m^2 and 15.6 ton/m^2 respectively which corresponds to a β value of 0.45 to 0.55, without considering piezometric draw down discussed in section 1. But if we consider piezometric draw down, corresponding β values will be 0.23 to 0.28. This confirms the suggestions made by Meyerhof (1976) who concluded that β value is also dependent on the length of the pile and can be as low as 0.15 for very long piles. Maximum unit skin friction mobilized in sand layers is also comparable to test results by Reese and O'Neill (1988) who measured a maximum value of 19.15 ton/m^2 for sand layers.

Dense sand and very stiff to hard clay layers found to exhibit a brittle type of failure mechanism in majority of the cases, a typical family of curves representing such type of behavior are shown in Figure 2. Residual shaft friction in dense sand layers with increasing butt loads found to be dropped to 50% of the maximum mobilized values in some cases. For very stiff to hard clay layers, such reduction is found to be as high as 60%.

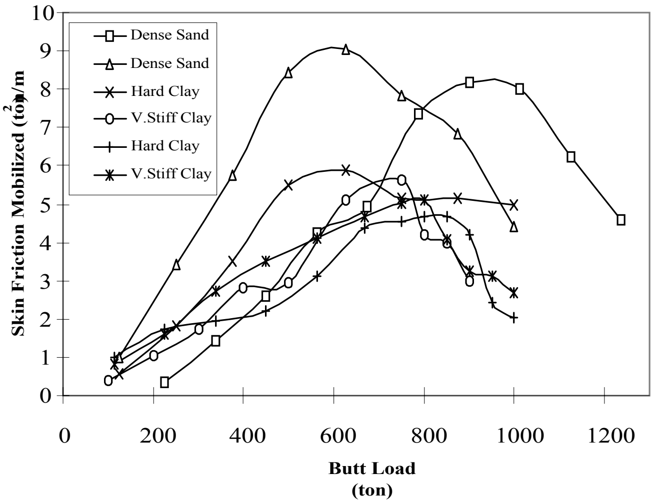

Figure 2: Family of curves showing typical brittle failure in dense sand and very stiff to hard clay layers.

Failure mechanism of different soil horizons with increasing butt loads exhibit a progressive type of behavior for some test piles. A typical, unit skin friction development with increasing butt load (TP10) is shown in Figure 3. In number of cases dense sand and very stiff to hard clay layers found to reach their ultimate unit friction values well before the maximum test loads which shows that pile head displacements required to mobilize ultimate shaft friction in these layers are quite small. An other point to be noted here is that the progressive failure mechanism found to be started from the layers present in the middle reach of the pile shaft and then it extended towards top and bottom. Once the sand and stiff to hard clay layers started yielding most of the loads were shifted to the medium and soft clay layers at the top.

5.2 *End bearing*

End bearing values mobilized under maximum test loads are given in Table 3. It is quite evident that considerable portion of the total pile head settlement is absorbed by the elastic shortening of the pile shaft. So even at 3 to 3.5 times working loads maximum pile tip movement is less than 7% of the diameter of the pile. Ultimate end bearing values which need a pile base settlement of 20% to 30% of the pile base diameter (Tomlinson, 1995) are still only 25% mobilized.

Maximum mobilized end bearing for piles with tips in the sand layers are plotted in Figure 4. It is clear that the estimated end bearing value of

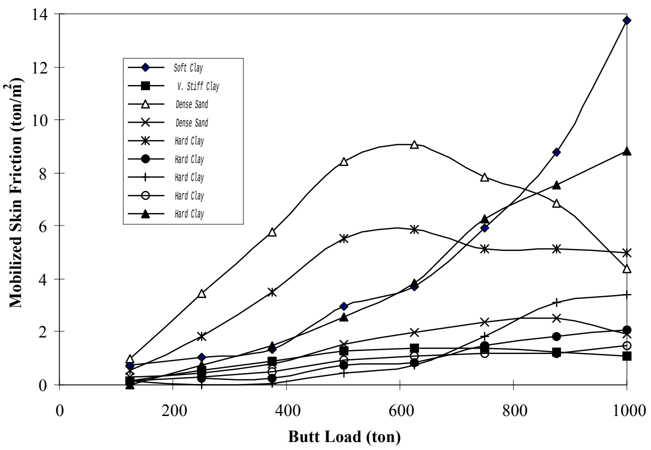

Figure 3: Skin friction mobilization in different soil horizons for TP10

Table 3. Maximum end bearing mobilized and corresponding pile base settlement from telltale data.

Pile No.	Max. Load Transfered to the Base VWSG (ton)	Elastic Compression of Pile (mm)	Total Settlement of Pile Head (mm)	Net Settlement of Pile Base (mm)	% of Pile Dia.	Max. End Bearing Mobilized (ton/m^2)	Estimated End Bearing (ton/m^2)	Average SPT-N Near the Toe	Soil Type near the Toe
TP-1	418	25.7	94.1	68.4	5.7	370	500	100	Clayey Sand (SC)
TP-2	71	16.2	34.7	18.4	1.5	63	500	100	SAND (SP-SM)
TP-3	382	18.3	81.6	63.3	6.3	487	500	70	Silty Sand (SM)
TP-4	230	16.3	90.7	74.4	7.4	293	206	45	Silty CLAY (CH)
TP-5	194	13.0	60.5	47.5	4.7	247	430	35	Silty Sand (SM)
TP-6	199	9.1	13.8	4.7	0.5	254	500	50	Silty Sand (SM)
TP-7	101	15.8	58.4	42.7	3.6	89	183	40	Silty CLAY (CL)
TP-8	481	9.1	90.9	81.9	6.8	426	500	60	Silty Sand (SM)
TP-9	199	14.1	69.0	55.0	5.5	253	500	60	Silty Sand (SM)
TP-10	201	8.0	16.3	8.3	0.7	178	228	50	Silty CLAY (CL)

500 ton/m^2 in most of the cases correspond to a base settlement of approximately 7% of the pile diameter and this level of settlement have rarely been achieved even at such a high pile top displacements.

End bearing mobilized in the sand layers for different values of base settlement (ref. Figure 4) match well with the recommendations of Reese and O'Neill (1988) who recommended a value of critical end bearing equal to 5.76*SPT-N (ton/m^2) corresponding to a base settlement of 5% of the pile base diameter for sand layers of similar relative density. Since the average value of SPT-N for test piles founded in sand layers is approximately 60, critical end bearing comes out to be 5.76*60 = 350 ton/m^2 and from the test piles plotted data Figure 4,

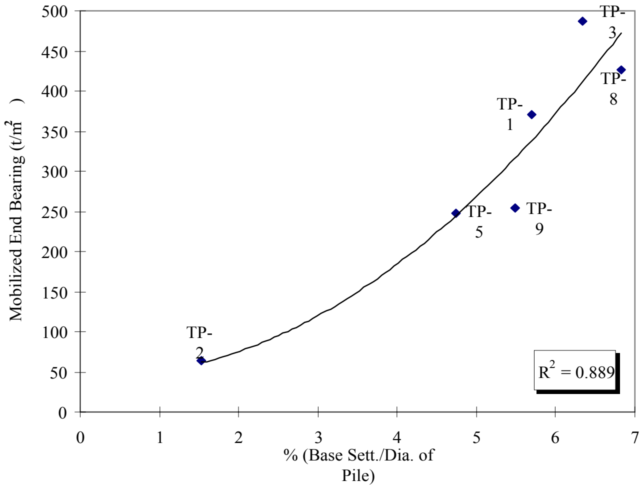

Figure 4: Development of end bearing with base settlement as % of pile diameter.

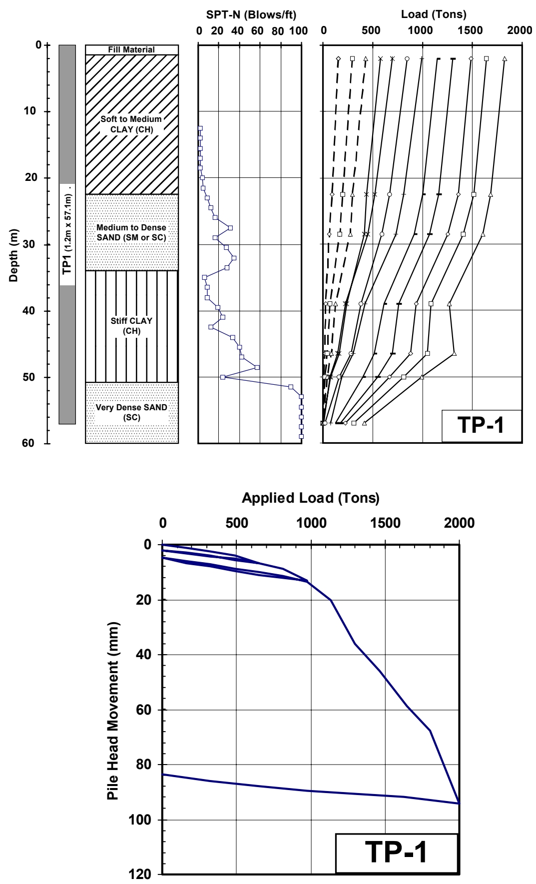

Figure 5: Test pile TP-1

critical end bearing corresponding to 5% base settlement is 270 ton/m^2, a difference between the predicted value of end bearing is due to the reason that end bearing values predicted by Reese and O'Neill (1988) are based on the SPT data with energy ratio (ER) of 0.55, while SPT-N used for the test piles are not corrected for ER.

Back calculated average value of Nc for clay

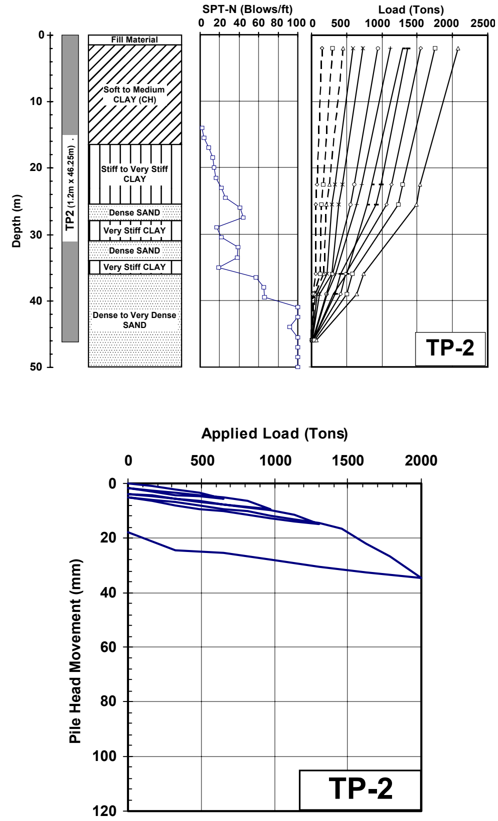

Figure 6: Test pile TP-2

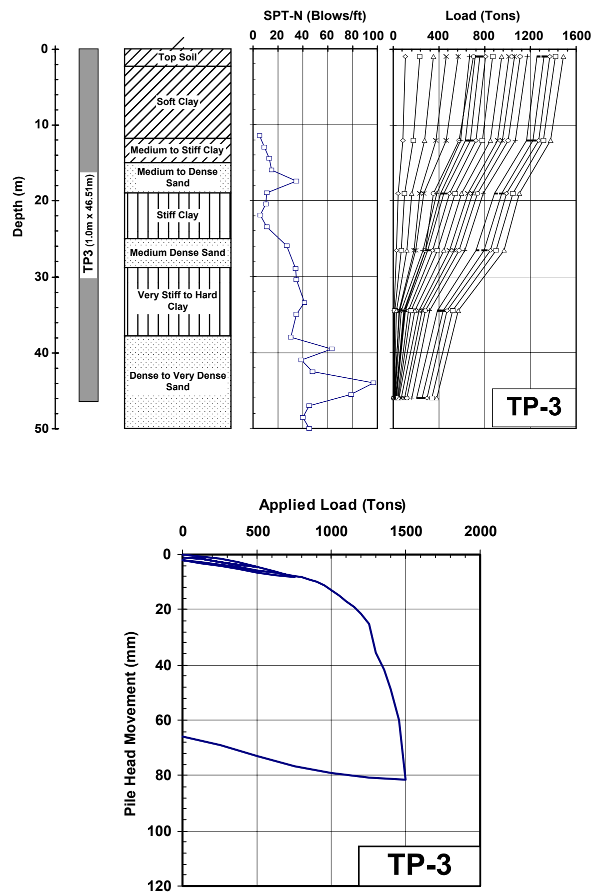

Figure 7: Test pile TP-3

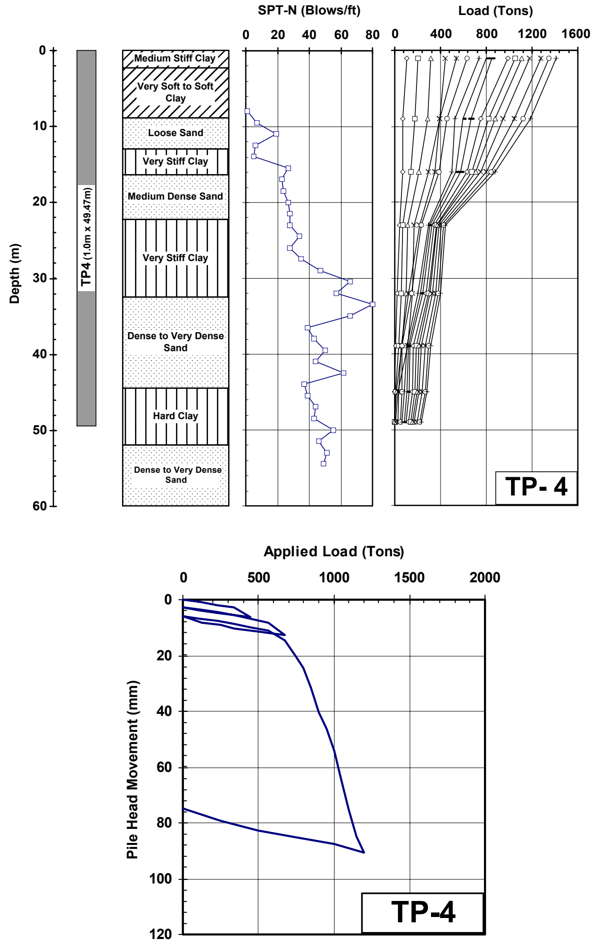

Figure 8: Test pile TP-4

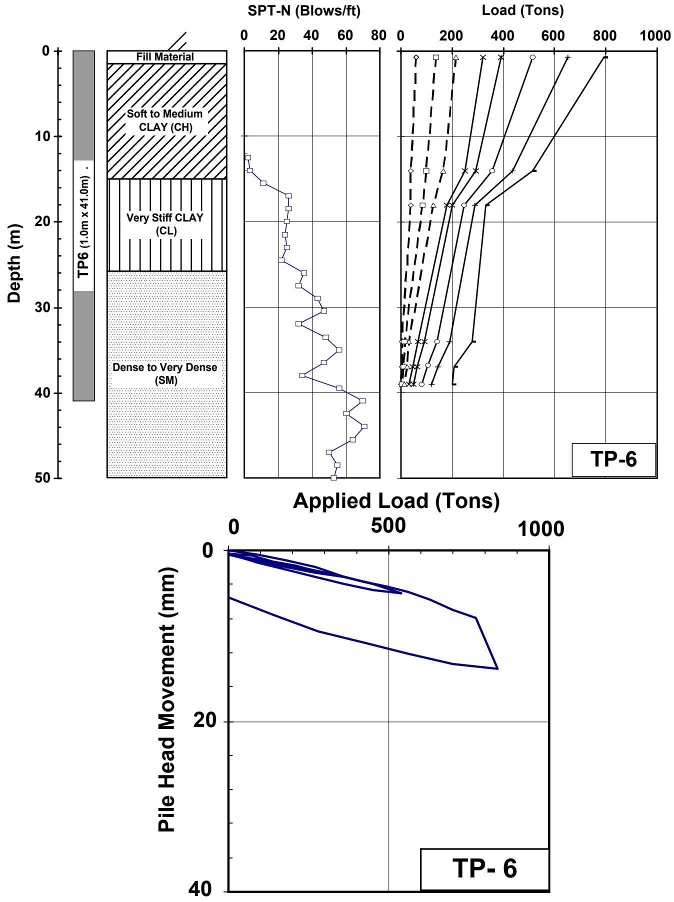

Figure 10: Test pile TP-6

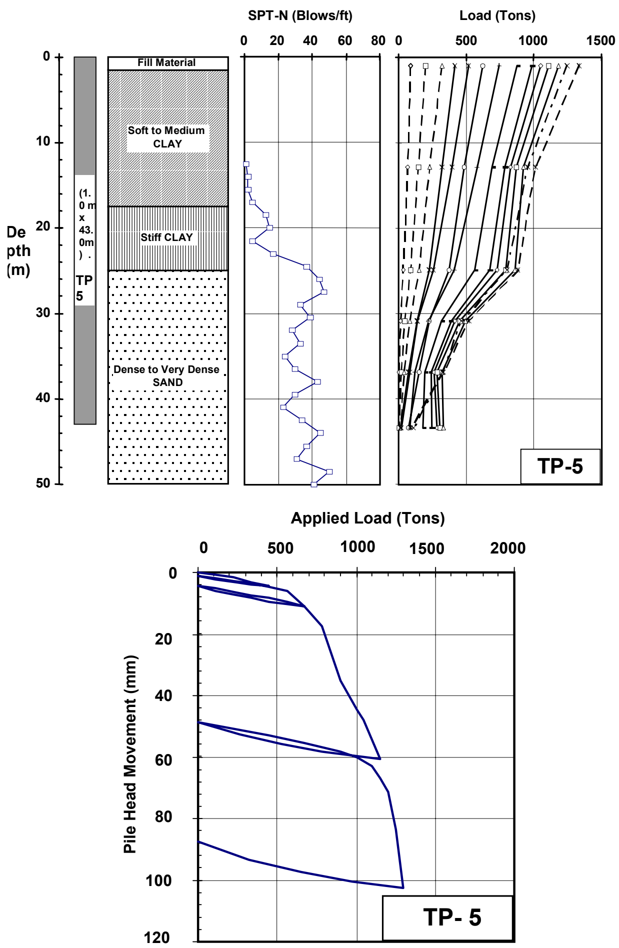

Figure 9: Test pile TP-5

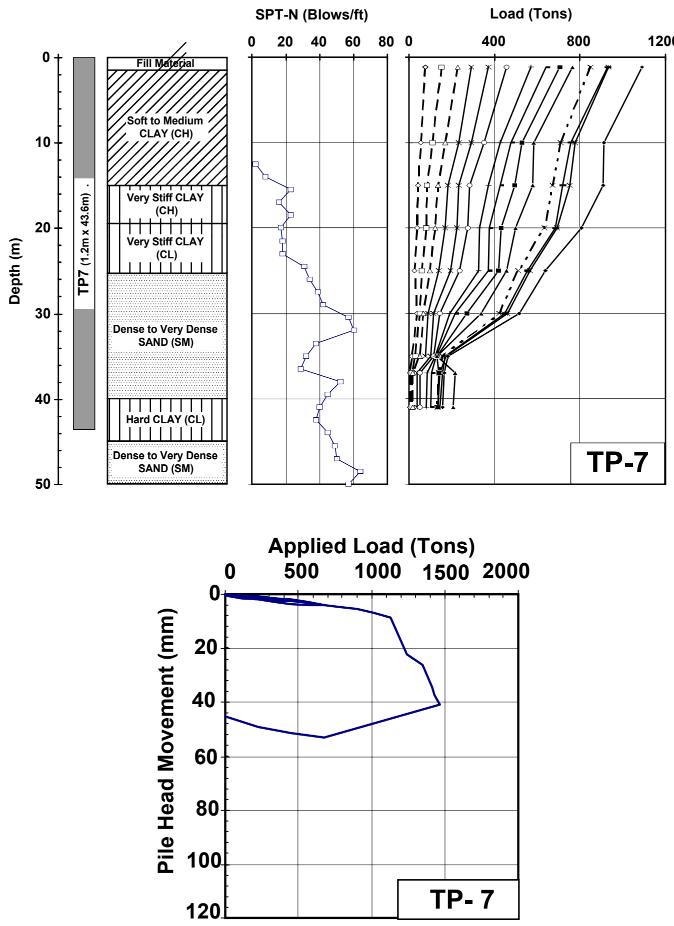

Figure 11: Test pile TP-7

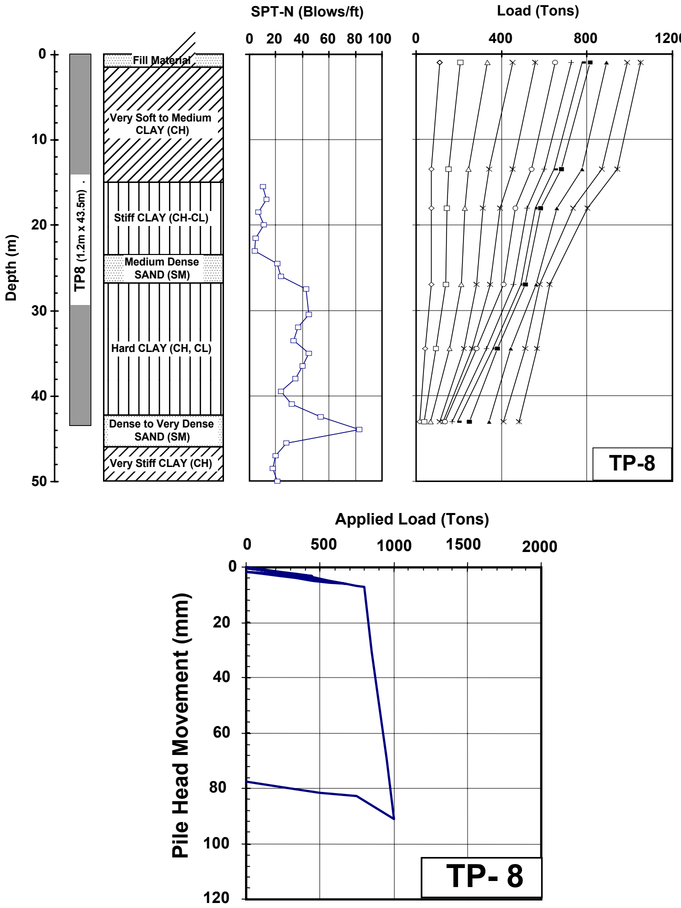

Figure 12: Test pile TP-8

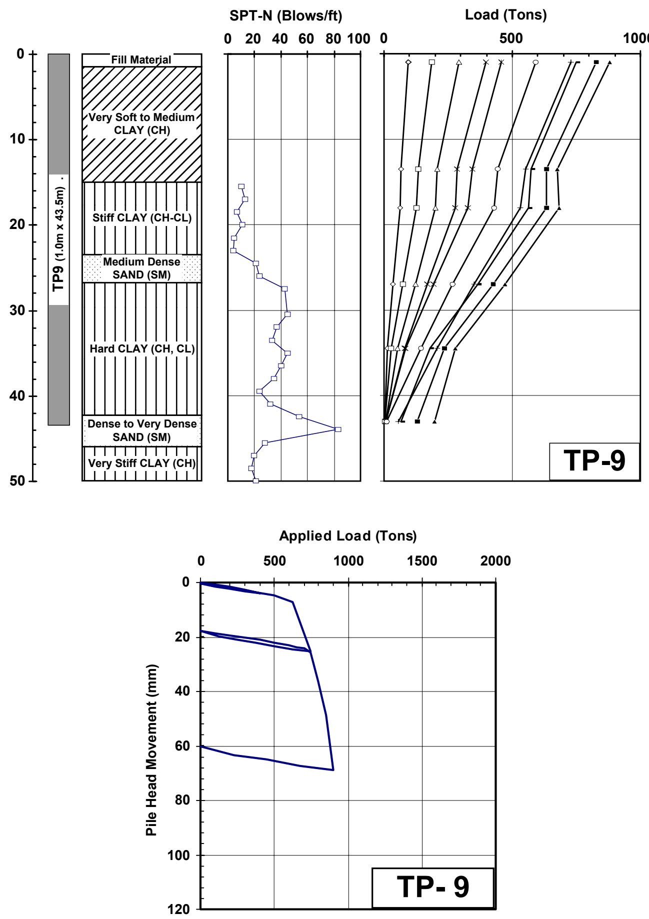

Figure 13: Test pile TP-9

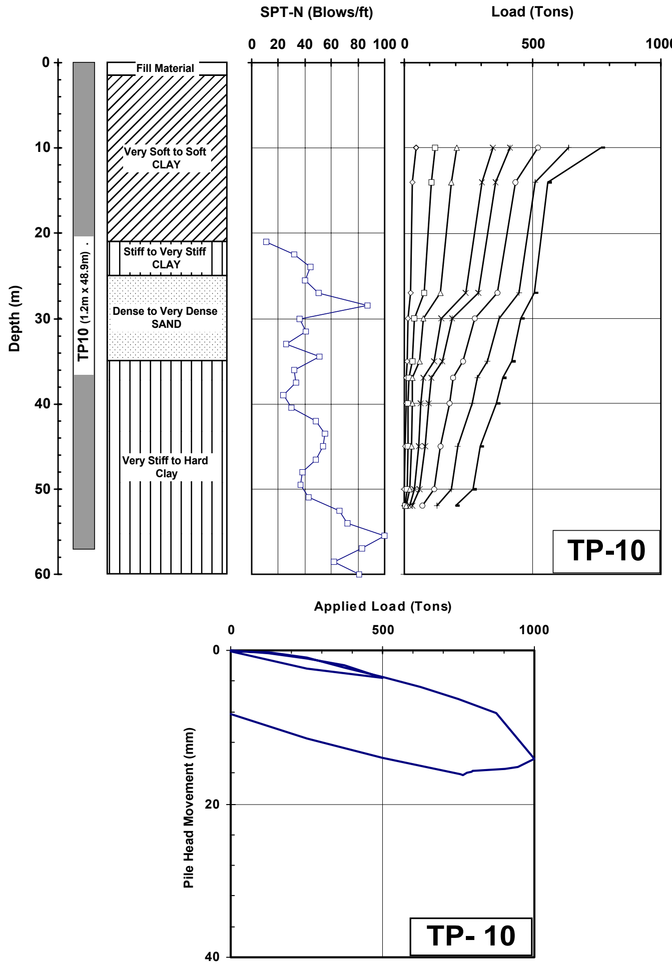

Figure 14: Test pile TP-10

layers is 8 with 4.5, 6.0 and 12.8 for the three piles seated in clay layers. From Table 3 it can be confirmed that the end bearing values in clay layer also depend upon the base settlement and values of Nc more than 9 can also be mobilized at high base settlement ratios. An other possible reason for the value of Nc more than 9 may be the underestimation of undrained shear strength of clay from correlation with SPT-N discussed in section 3.1.

6 CONCLUSIONS

1. Skin friction and end bearing values of wet process bored piles mobilized in the subsoils of Bangkok are reported and compared with the previous researches made for similar cases.
2. Dense sand and very stiff to hard clay layers often exhibit a brittle type of failure with residual friction capacities as low as 50 to 60% of the peak mobilized values.
3. Failure of long piles in multi-layered soil is dominantly progressive in nature and some soil horizons reach their ultimate friction capacities well before the over all ultimate failure loads.
4. Calculation of ultimate failure loads from the normally available methods and then design

loads by applying a global FOS give widely scattered settlements at design loads and if we calculate design loads from limit load concept, quite uniform settlement at design loads can be achieved.

AKNOWLEDGEMENTS

Writers gratefully acknowledge the continuous support and invaluable suggestions made by their colleagues, especially Mr. Aung Win Maung and Mr. Ganeshan Baskaran from analysis to final draft of this paper.

REFRENCES

Bengt B.Broms and M.F.Chang (1988), Bored Piles in Residual Soil and Weathered Rocks in Singapore, Deep Foundations on Bored and Auger Piles, Van Impe (ed.), Balkema, Rotterdam.

Bowles, J.E.(1988), Foundation Analysis and Design, McGraw-Hill.

Buttling, S. (1992), Prediction of the Performance of Some Instrumented Pile Tests in Hong Kong, Proc. of Symposium on Prediction Vs Performance in Geotechnical Engineering, SEAGS, Bangkok.

Meyerhof, G.G. (1976), Bearing Capacity and Settlement of Pile Foundation, Jour. Of Geotch. Engg. Div., ASCE Vol102, No GT3, pp195-228.

Ng, K.C. (1983), The Construction Problems and Performance of Large Diameter Bored Piles in Second Sand Layer, M. Engg. Thesis no. GT-82-26, AIT, Bangkok.

Pitupakorn, W. (1985), Predictions of prestressed concrete pile capacity in Bangkok stiff clay and clayey sand, 8th Southeast Asian Geotechnical conf, Kuala Lumpur.

Reese and O' Neil (1988), Drilled Shafts: Design Procedures and Design Methods, US Dept. of Transportation F. H.A. Office of implementation, McLean, Va.

Tomlinson, M.J. (1995), Pile Design and Construction Practice, 4th Ed. Chapman & Hall.