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Effect of construction time and bentonite viscosity on shaft capacity of bored piles

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Effect of construction time and bentonite viscosity on shaft capacity of bored piles

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ABSTRACT: Effect of construction time and slurry viscosity on the shaft friction capacity of cast in situ bored piles have been studied by various investigators in the past but the extent to which these parameters effect the shaft capacity is still not clear. Major obstacle in this regard is that it is hard to normalize the shaft capacity degradation against other numerous parameters which are influencing simultaneously. Results from eleven pile load tests, constructed with different slurry viscosities and construction times, in the layered alluvial strata of Bangkok are presented in this paper.

It has been concluded that slurry viscosity, do not have significant effect on the shaft load transfer of these piles but considerably reduced with increase in construction time. Trend of reduction in capacity seems to follow an exponential decrease with increase in construction time with major part of degradation within first 24 hours of construction time.

1 INTRODUCTION

Shaft friction capacity of bored piles, especially, in granular soils is greatly affected by the installation procedures, but the sole effect of bentonite slurry (slurry) properties and its exposure time is still unclear. Results from the previous researches on deleterious effects of slurry properties and its contact time on the shaft load transfer of wet process bored piles are a bit anecdotal and to some extent contradictory. Major problem in this regard is that, the effect of either parameter can not be totally isolated from other influencing variables. Due to this reason researches have been found to report almost converse conclusions for the similar type of soils.

In order to assess the effect of slurry viscosity and exposure time on the shaft load transfer, data from eleven instrumented bored pile load tests is presented in this paper. All test piles were constructed using same type of equipment and procedures, in a similar type of soil conditions. So it is assumed that the influence of other variables would be minimized and the effect of slurry viscosity and exposure time, to some extent, could be estimated.

2 BORED PILES AND SOIL CONDITIONS

Use of bored cast in situ piles in Bangkok, is very common. Diameters of these piles normally fall in the range of 0.80 to 1.50 m and toe depths down to 60 m from ground level are quite usual. Soil profile of plain of Bangkok generally consists of Quaternary alluvial deposits of alternating sand and clay layers as shown in Figure 1, down to the rock face which is reported to be at least 550 m deep in the area (Balasubramanium, 1991).

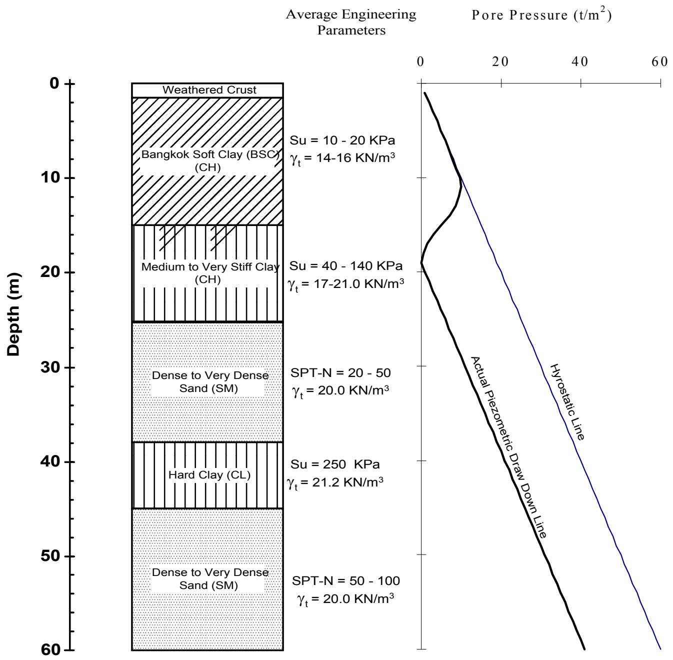

Figure 1. Typical soil profile of Bangkok plain with piezometric draw down conditions.

Below the top few meters of weathered crust, a thick layer of well known "Bangkok Soft Clay" (BSC) changing to medium at about 15 to 18 m, is present. First sand layer is usually 5 to 10 m thick and found at 25 m to 30 m depth, below is a series of stiff to hard clay and medium to very dense silty sand layers. Actual pore pressure conditions in the upper BSC are hydrostatic from approximately 1 m below ground level. Then the hydrostatic conditions changes to piezometric draw down near the bottom level of BSC, Figure 1. Piezometric draw down resulted in increased effective overburden pressure of about 20 ton/m2 in the first sand layer and below. The depressurization of sand layers is attributed to the deep well pumping in the area, Nutalaya (1981).

3 REVIEW OF PREVIOUS WORK

Degradation of perimeter load transfer in permeable soil layers is attributed to the formation of filter cake which is left in place and get sandwiched at soil-pile interface and is not scrapped of by fluid concrete. Scouring capability of fluid concrete depends upon its shear strength and that of the filter cake and it is argued that if shear strength of filter cake is more than the fluid concrete, it can not be scoured and left in place resulting in the degradation of load transfer capacity of the shaft. In case of impermeable soils reduction in the shaft load transfer is mainly due to the softening of the soil at the perimeter of the borehole, because the formation of thick filter cake is not possible as slurry can not permeate in to the soil, but some researchers e. g. Veder, C. (1963) observed a thin cake of few millimeters even in clay formations and argued that it may be the result of electrical forces or chemical reaction of bentonite suspension on the wall of the borehole.

O'Neil et al. (1992), reported the effect of slurry properties on two drilled shafts constructed in similar ground conditions (alternating layers of very stiff clay and dense sand). Shafts 1 and 2 were constructed with Marsh cone viscosity of 37 and 49 sec. respectively, time between opening the borehole and completion of concreting for both piles was 5 to 7 hours. Minor difference in load transfer of both shafts was found and authors concluded that the difference in slurry properties used for the construction for both shafts had a little effect on shaft load transfer.

Cernak (1976), performed full-scale load tests on three barrettes in sandy gravels, two barrettes were constructed using slurry with different exposure times, 8 and 97 hours. Load test results indicated a decrease of skin friction capacity of 43 percent and 56 percent, respectively, as compared to the barrette excavated dry and concreted immediately. It must be noted that major part of the reduction i. e. 43 percent took place in the first 8 hours of contact time and only 13 percent (56-43) reduction took place in the rest of 89 hours (97-8), for the second barrette. These findings support the concept that major part of the shaft friction capacity reduction with increase in construction time, took place in first few hours of construction time and further increase in construction time have minor contribution towards the reduction in shaft friction capacity.

Corbette (1975), presented the results of load testing of a diaphragm wall panel excavated in conjunction with bentonite suspension, and construction time of approximately 48 hours. He inferred that the presence of bentonite suspension even for such a long time has not adversely affected the development of skin friction.

Littlechild and Plumbridge (1998), concluded from the pile load tests of BERTS project Bangkok, that shaft friction capacity of bored piles constructed under slurry tends to decrease as the construction time and slurry viscosity increases.

Majano and O'Neil (1993) attempted to model the formation of filter cake in the laboratory with different slurry dosages, differential pressures and exposure times. They stated that perimeter load transfer is a complex function of the physical and chemical characteristics of the slurry and the geomaterials, the roughness of the borehole, the fluid pressures exerted by concrete, the shearing properties of the soil, and possibly the chemistry of the fluid concrete. They attempted to correlate the potential degradation of the soil-pile interface to the thickness and shear strength of the cake formed against the walls, and argued that the shear strength and thickness can not be measured with only one or two parameters like slurry dosage and differential pressure. With a bentonite concentration of 72 kg/m3 and differential pressure of 0.5 psi they achieved a filter cake thickness of 3.14 and 4.5 mm after a contact time of 4 and 24 hours. Authors also compared the laboratory tests with the actual field conditions and proved that due to the presence of enormously high differential pressures in the field in some cases, thicknesses of filter cakes which would be considered detrimental for shaft load transfer, require few hours to develop (They proved that a filter cake of 6 mm thickness would require only 2.3 hours to develop in the field under a differential pressure of 10 psi).

4 METHOD OF CONSTRUCTION

All the eleven piles reported in this paper were constructed, following the internationally accepted guidelines and specifications. Slurry in conjunction with rotary bucket is used for all piles.

Temporary casing, having length depending upon the thickness of the BSC at the site is driven using a high frequency vibro-hammer. Drilling operation is commenced with an auger down to the base of the temporary casing. Further drilling below the bottom of temporary casing is continued using a drilling bucket by first filling the borehole with slurry. Bentonite, classified as Activated Sodium Bentonite is usually used and slurry properties are controlled within the limits given in Table 1.

Table 1. Slurry specifications followed.

Slurry Property	**Control Limits**
Viscosity (Marsh cone)	30 - 60
Density (g/cc)	1.04 - 1.15
pH	7 - 11
Sand Content	<4%

Outer diameter of the drilling bucket is slightly less than the nominal size of the pile to provide an annular by-pass for the slurry during lowering and hoisting operations. Required nominal size of the pile is achieved using a cutting edge with sidecutters. Slurry level in the borehole is maintained within 1 to 1.5 m from top of temporary casing. After drilling down to the required tip level of pile, special cleaning bucket and/or air lift technique is normally applied to clean the borehole base of any congregated sediments before lowering the reinforcement cage. Concreting is done using a tremie pipe at the center of the borehole. Total time to carry out this operation, 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, delay in ready mixed concrete due to traffic congestion or similar reasons. In most of the cases whole operation of pile construction is accomplished within 24 hours, but in some cases, due to the local authority regulations which do not permit to operate during night time hours, borehole has to be left exposed for 10 to 12 hours unagitated. In such cases, drilling is intentionally curtailed above the final excavation level and resumes next day. We assume that in such cases, during drilling after a delayed period of 10 to 12 hours, any possible filter cake on the borehole walls is automatically scraped of. Construction times more than 24 hours reported in Table 2 are examples of such cases. Construction time reported in Table 2, col. D is the time elapsed between the start of auger drilling to the finishing of concreting. Longer construction times for the test pile reported are usually due to the time consumed in instrumentation installation with the rebar cage. It must also be noted that the actual contact time for slurry starts, once it is fed to the borehole till the up flowing concrete reaches the bottom level of temporary casing. Since the accumulated time required for concreting above the bottom level of temporary casing and auger drilling inside the temporary casing roughly required one hour, construction times given in Table 2 are supposed to be appropriate for comparison.

5 SHAFT CAPACITY ESTIMATION

A commonly accepted design approach which consists of a combination of total stress method for clay layers and effective stress method for sand layers is adopted to estimate the shaft friction capacity reported in Table 2, col. F. Skin friction capacity of clay layers is estimated as $fs = \alpha.Cu$, with adhesion factor α. taken from the curve proposed by Suchada (1989), Figure 2, for Bangkok subsoils. It can be noted that value of α used is quite comparable to the previous recommendations by Tomlinson (1957) and Kulhawy (1984).

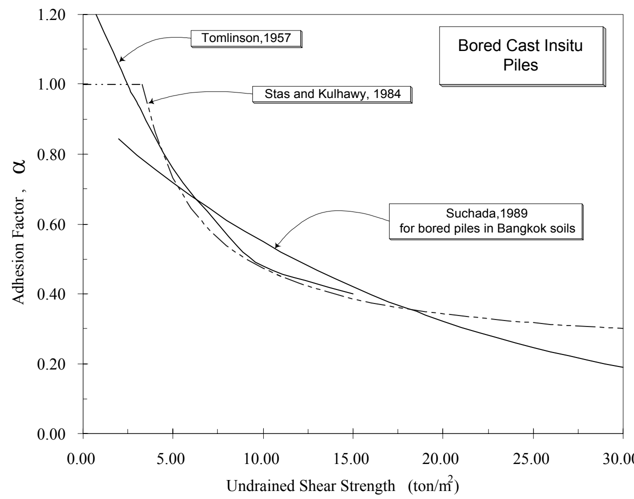

Figure 2. Comparison of Adhesion factor α used for bored piles in Bangkok Subsoils.

Undrained shear strength Cu of shallow clay layers is normally 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), as Cu = C1.SPT-N (ton/m2), with C1 equal to 0.674 and 0.507 for high and low plasticity clays respectively. C1 values mentioned here are based on the statistical correlations suggested by Pitupakorn, 1985. Skin friction capacities for the sand layers are calculated as $fs = \sigma v'. Ks.\tan.\delta$ with coefficient of horizontal earth pressure Ks equal to 0.7 and δ equal to 0.75 ϕ.

Table 2. Construction parameters and shaft capacities of test piles.

Pile No.	Pile Dimensions (m)	Bentonite Slurry Viscosity (sec.)	Construction Time (hrs)	Pile Age at the Time of Load Testing (days)	Estimated Shaft Capacity (ton)	Actual Shaft Capacity (ton)	Actual/Estimated Shaft Capacity
A	B	C	D	E	F	G	H=(G/F)
TP-1	ϕ1.2x57.1	38	43.0	32	1230	1200	0.98
TP-2	ϕ1.2x46.3	38	13.1	29	930	1300	1.40
TP-3	ϕ1.0x46.5	37	9.8	25	800	1250	1.56
TP-4	ϕ1.0x49.5	37	38.7	15	930	750	0.81
TP-5	ϕ1.0x43.0	38	26.0	19	690	700	1.01
TP-6	ϕ1.0x41.0	45	11.8	32	620	800	1.29
TP-7	ϕ1.2x43.6	55	16.8	32	850	1100	1.29
TP-8	ϕ1.2x43.5	38	12.3	24	640	750	1.17
TP-9	ϕ1.0x43.5	37	11.3	18	530	700	1.32
TP-10	ϕ1.2x62.0	41	32.4	27	1534	2000	1.30
TP-11	ϕ1.2x54.2	38	30.0	39	1050	1300	1.24

Value of Ks used is in good agreement with the recommended values by different researchers like Fleming (1977), who suggested a value not more than 0.75 to account for the possible degradation due to the formation of filter cake. 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.δ It must be noted that the empirical parameters discussed here are based on the effective overburden pressure σv' 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.

6 TEST PILES

Test piles discussed in this paper were all constructed following the procedure discussed in section 4. All piles were instrumented with Vibrating Wire Strain Gauges (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 Qd and maintained for 12 hours followed by second cycle of loading up to 2 times of Qd and maintained for 24 hours. Third cycle of loading is applied up to 2.5 times Qd or maximum pre-decided test load. Occasionally, a fourth quick loading cycle up to the maximum test load is also applied and maintained for two hours. All test piles, except TP-2 and TP-10, were tested to well above the expected failure loads with the concept of sufficiently mobilizing the end bearing. Load displacement curves of the test piles are shown in Figure 3 for comparison.

7 ACTUAL SHAFT FRICTION CAPACITIES

Actually mobilized shaft friction capacities are calculated from the VWSG load transfer results at a level where load displacement curves show a maximum curvature and are given in Table 2, col. G.

8 EFFECT OF SLURRY VISCOSITY

Effect of slurry viscosity on the shaft load transfer can best be seen from the plot between the ratio of Actual/Estimated shaft capacity versus slurry viscosity, Figure 4. Over all scatter of data do not suggest any trend. Since the effect of slurry viscosity is difficult to separate, without normalizing the effect of construction time. Test piles TP-6, 7 and 9 have a slight difference in construction time and pile age at load testing but slurry viscosity is significantly different, 45, 55, and 37 sec. respectively but the ratio of Actu./Esti. shaft capacities for these piles are necessarily the same

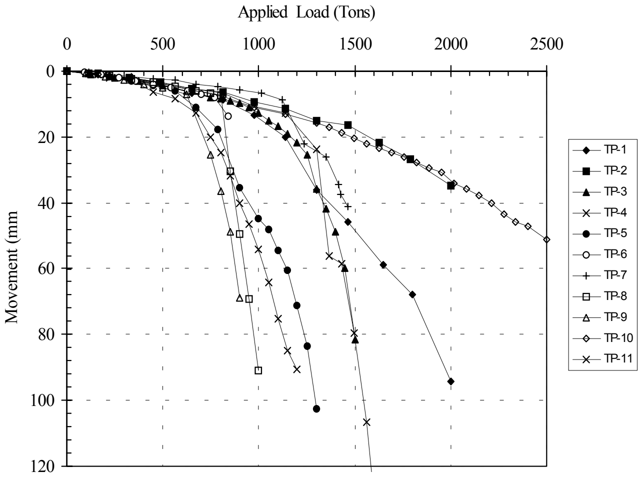

Figure 3. Load settlement curves of the test piles.

which suggest that the sole effect of slurry viscosity on shaft capacity reduction is insignificant. Additionally, TP-1, 4 and 5 which exhibit a maximum reduction in shaft capacity have almost same slurry viscosity but different construction time which confirms that slurry viscosity within the data set available do not directly have any effect on the shaft load transfer of the piles.

9 EFFECT OF CONSTRUCTION TIME

Since ten out of eleven bored piles were constructed with slurry viscosity varying within narrow range effect of construction time can best be seen in Figure 5, plot between Actu./Esti. shaft capacity versus construction time. Though the data is widely scattered, best fit curve shows the trend of exponential decrease of shaft load transfer with increase in construction time. Best fit curve for the available data also reveals that the available shaft capacity is more than the estimated capacity for a construction time as long as 40 hours. This confirms the validity of recommendations made by some researchers like Fleming (1977), who suggest to complete the concreting within 24 hours after finishing the drilling, and seems to be more reasonable because slurry is kept continuously agitated during drilling operation and actual exposure time for slurry starts, once the drilling is completed. Test pile TP-4 which exhibited the lowest capacity and has the minimum slurry viscosity (37 sec.), second longest construction time (38.7 hours) and the minimum age (15 days) at the time of load testing, in the group reveals that the shaft capacity reduction is contributed by increased construction time as well as reduced pile age while slurry viscosity have minor effect.

It can also be noted that the major part of the shaft capacity reduction is contributed by the first 24 hours of construction time which normally considered feasible and agreeable by most of the piling contractors under normal circumstances, and further delays beyond 24 hours have a minor addition to the shaft load capacity degradation. These findings are in good agreement with the previous observations discussed in section 3.

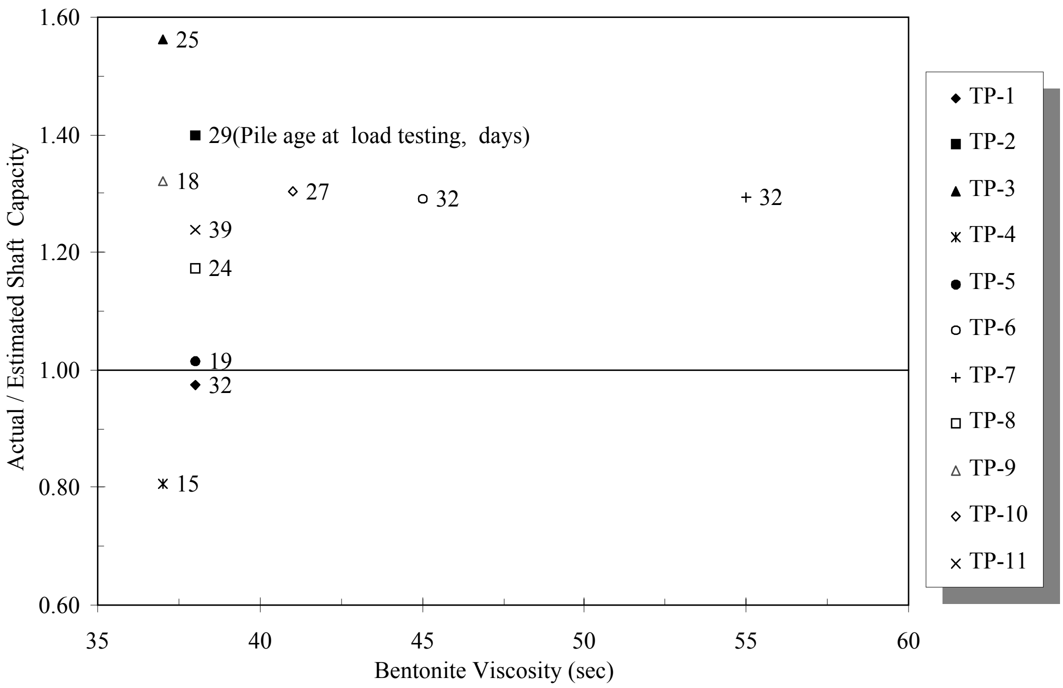

Figure 4. Effect of slurry viscosity on shaft capacity of bored piles.

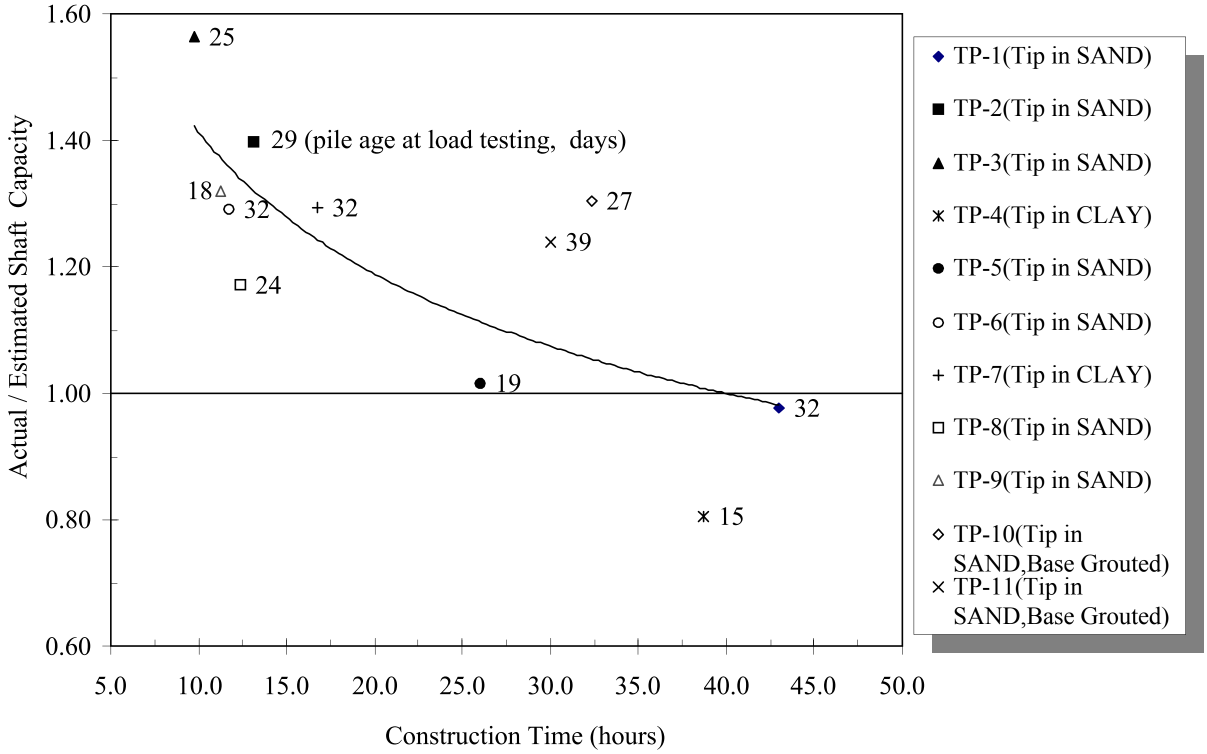

Figure 5. Effect of construction time on shaft capacity of bored piles.

10 DISCUSSION

From Figure 5 it can be noted that actually mobilized shaft capacity is equal to the estimated shaft capacity for a construction time of approximately 35 - 40 hours or in other words, the empirical design parameters commonly used to estimate the shaft friction capacities discussed in section 5, are justifiable for a total construction time of longer than 24 hours, especially, to account for the unexpected delays during construction in the downtown areas of Bangkok which are notorious for its traffic congestion and other constraints imposed by local authorities. This also confirms that the allowable time of 24 hours after finishing the drilling operation (Fleming, 1977) is also justifiable. However, with no dispute, shorter the construction time better the shaft load transfer response.

It must therefore be emphasized that the selection of empirical parameters like α, β, δ and Ks, to estimate shaft load capacities at the design stages must also consider the practically possible construction times by suitably incorporating the effects of any possible delays specific to the site and working conditions. Empirical factors mentioned above are not the fundamental properties of the soil-pile system, and need to be accounted for the installation procedures or even the method of pile load testing like Constant Rate of Penetration (CRP) or Maintained Load (ML). Reduction in construction time, generally, lead to better performance of pile shafts but this reduced construction time could never exclude the accidental occurrence of delays. Any potential degradation in load carrying capacity of such piles caused by uncontrollable delays must be considered along with the conceivable trend in reduction under these conditions.

It must also be noted that because of high differential pressures imposed due to depressurized sand layers in Bangkok sub soils (differential pressures of the order of 30 psi will be imposed if slurry level is maintained near the ground level due to piezometric draw down of 20 m in the sand layers), formation of filter cakes which are considered to degrade the shaft load transfer would theoretically take fraction of seconds to develop, if we calculate on the basis of procedure recommended by Majano (1993). This means that the degradation of pile shaft interface due to the formation of filter cake in permeable soil layers is unavoidable within the practical construction time limits.

11 CONCLUSIONS

1. Bentonite slurry viscosity within the data set available do not have significant effect on the shaft load transfer of bored cast in situ piles.

2. Longer construction time reduces the shaft carrying capacity of bored piles constructed under slurry. However, the major portion of degradation occurs within the first 24 hours period, which is unavoidable under normal construction time.
3. Degradation of shaft capacity with increase in construction time beyond 24 hours is not excessive and can be adjusted within the empirical design parameters or factor of safeties chosen at the design stages.
4. Piles constructed within 24 hours showed generally higher shaft capacities than estimated as per normal design practice adopting empirical parameters suggested by different researchers for Bangkok subsoils and can safely be used for slurry viscosity up to 55 sec. and total construction time up to 40 hours.

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