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THE SKIN FRICTION OF LARGE BORED PILE IN COHESIONLESS SOIL

Siwawet Obma[1] and Watcharin Gasaluck[2]
[1]Master's Degree Student, Department of Civil Engineering, Faculty of Engineering, Khon Kaen University, Khon Kaen, Thailand Tel. +66-43-202846 Fax. +66-43-202847.
email: siwawet@hotmail.com
[2]Associate Profressor, Department of Civil Engineering, Faculty of Engineering, Khon Kaen University, Khon Kaen, Thailand Tel. +66-43-202846 Fax. +66-43-202847.
email: watgas@kku.ac.th

ABSTRACT

Thai geotechnical engineers started using bored piles more than 40 years ago but only in Bangkok, the capital of Thailand in which the soil is strata consists clay and sand. Piling projects outside Bangkok still prefer driven piles than bored piles because of the low cost. Recently bored pile is extensively used, especially in downtown but the pile size is small. The large bored piles were used for the first time in Khon Kaen in which the soil is cohesionless. A 1.2 m diameter bored pile with 40 m long was tested by static method. The frictions along pile shaft were measured by 21 vibrating wire strain gauges installed in pile and then compared with the frictions computed by various methods proposed in the literatures. The method developed by Davies & Chan(1981) gave the best fit. The results show that K and β should not be constant but should vary in direct proportion to strength of the soils around pile.

KEYWORDS: Large Bored Pile, Bored Pile in Khon Kaen, Pile in Cohesionless Soil

1. Introduction

The Northeast of Thailand comprises an area of about 168,800 square kilometer, about one-third of the total area of the country as shown in Figure 1. During Quaternary period loess was deposited in this area. Figure 2 shows the distribution of loess in northeastern Thailand. Loess is described as silt textured eolian material. Loess in this area can be interpreted as an accumulation of wind-blown dust. The thickness normally ranges from a few to more than six meters. The soil characteristic is non-plastic red sandy silt or silty sand (ML or SM). Some loess has small clay content (SC or SC-SM), so it is not sticky but rather slippery sediment.

About 60% by weight of soil particles have the size of 0.03 to 0.2 mm [1]. Soil grains have a smooth and sub-rounded surface. The microstructure is loose to medium dense. The pore sizes are usually 0.2-0.5 millimeters although some can be as large as 1 millimeter [2]. Figure 3 illustrates the Khon Kaen loess taken by scanning electron microscope (SEM).

Driven piles are widely used in the Northeast of Thailand dues to the low construction cost. The fast development of urban area has led to the need of large bored piles because of 3 reasons. First, the heavy load of high-rise building cannot be carried by coventional driven piles. Second, the transportation of long piles is very difficult in downtown. Third, the vibration induced by piles driving could cause some effect to the neighbor building. Bored piles in this area are usually smaller than 1 meter. In 2013, the construction of 8-storey hospital building in Khon Kaen University necessitated large bored piles, the diameters of which were 1.2 and 1.5 meters.

Figure 1. Thailand map.

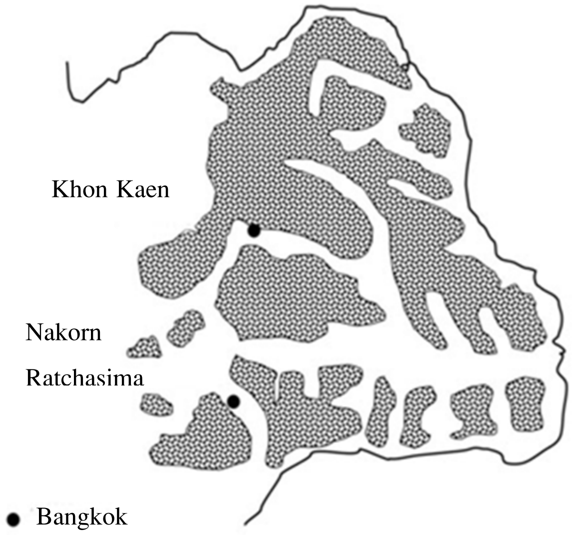

Figure 2. Distribution of loess in Northeastern Thailand. [2]

In Bangkok, the capital of Thailand, large bored piles have been used for more than 40 years. Since Bangkok soil is clayey, the bored pile knowhow is almost related to clay. The construction of large bored piles in Khon Kaen could be the first case study of the large bored pile in cohesionless soil. This paper presents the study of bearing capacity of the piles in this project.

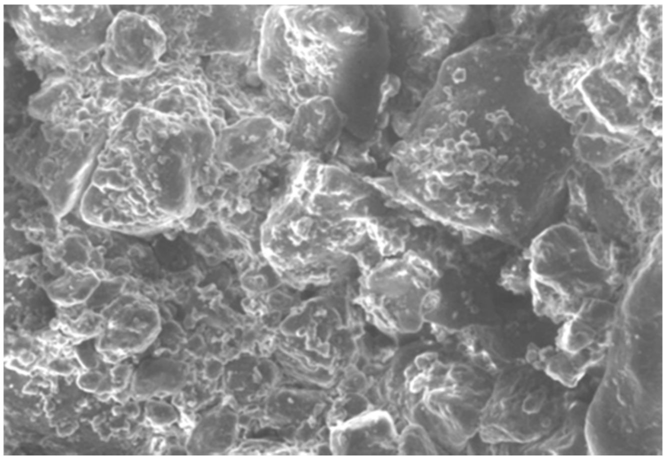

Figure 3. Khon Kaen loess. [1]

2. Instrumented Pile Load Test

[1] Pile Details

Figure 4 is the pile layout indicates soil investigation and test pile location. The building's foundation studied herein is composed of 104 nos and 43 nos of 1.5 and 1.2 m-diameter bored pile, respectively. The length of all piles are 40 m. The pile were constructed by using wet process method because the groundwater level was about 2 m depth. Seismic test results showed integrity of all piles. The piles design was based on the data from 2 boring logs, BH1 and BH2 as shown in Figure 5. Data of BH2 is not much different from BH1, only the SPT-N is rather smaller. Bearing capacities of piles estimated by the geotechnical engineers are depicted in Table 1. Static and dynamic load tests were performed at 4 piles, the tested piles positions were shown in Figure 4. The maximum loads of static load tests are as shown in Table 1. For the 1.2 m-diameter pile, the loads transferred to 7 levels of pile were measured by the 21 vibrating wire strain gauges installed in the pile, 3 gauges at each level.

[2] Supporting Fluid

Construction engineers would like to avoid the filter cake problem of bored pile, and thus they planned to use polymer for the wet process. Polymer is extensively use as supporting fluid for bored pile in Bangkok clay [3]. However, they found that the Polymer is not suitable to be used as a supporting fluid in this project. Therefore, Bentonite is used instead of polymer. Even though bentonite was used, the construction process must be planned to continuously carry on from the starting of boring until the end of concreting to avoid the risk of the borehole collapse.

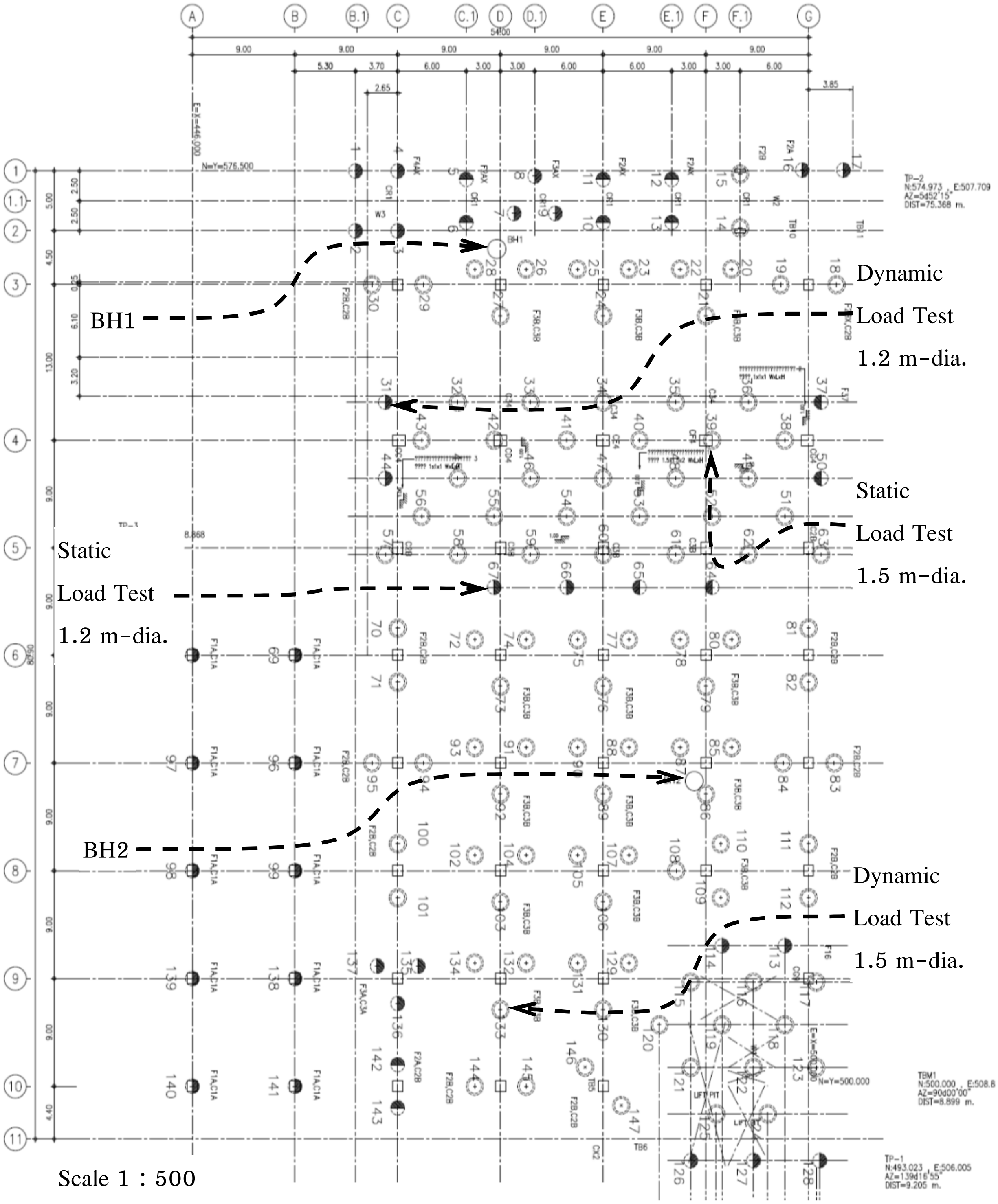

Figure 4. Site plan.

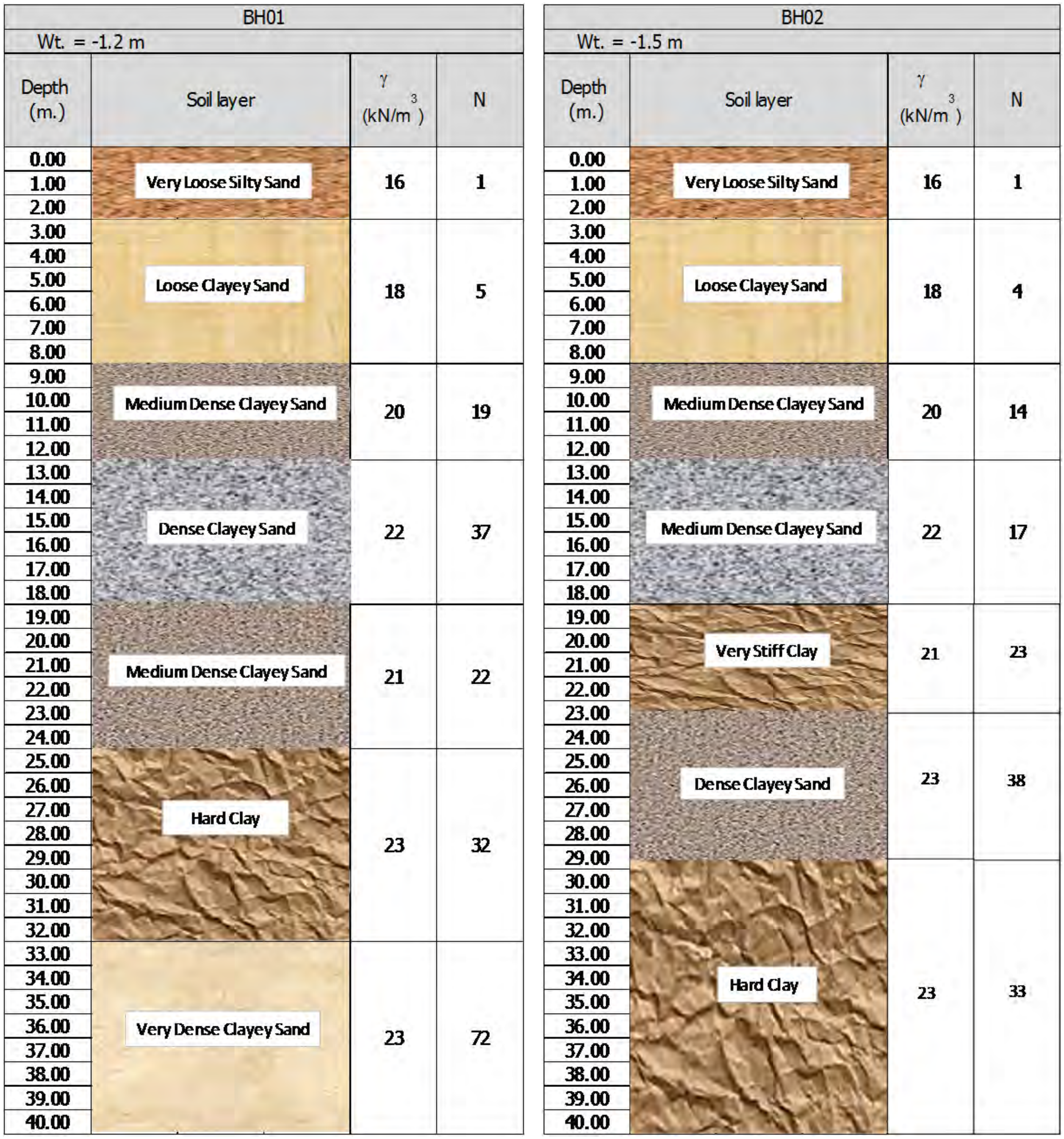

Figure 5. Boring Logs.

[3] Static Pile Load Test Result

The dynamic load test reported that the ultimate bearing capacities of piles were 12.86 and 13.84 MN for 1.2 and 1.5 m-diameter pile respectively. The static load test of 1.2 m-diameter pile gave the result as shown in Figure 6, with no sign of failure. The permanent settlements are shown in Table1. Figure 7 shows the load distribution along pile shaft obtained from strain gauges. Only the upper 18 m of pile reached the ultimate state. Table 2 shows the mobilized friction of each pile interval. PLAXIS 3D, the finite element program, was employed to estimate load distribution as well. Table 3 shows input parameters for calculating the program, Mohr Colomb was uesd for constitutive model and using fine mesh for area around the pile. The results are demonstrated in Figure 8.

Table 1. Static load test results.

Pile diameter (m)	Design load		Maximum test load		Permanent settlement (mm)	Settlement at Maximum Test Load (mm)
	ton	MN	ton	MN		
1.2	415.0	4.07	1245.0	12.21	1.50	6.05
1.5	565.0	5.54	1412.5	13.86	0.85	4.91

Table 2. Mobilized friction of 1.2 m-diameter pile.

Depth interval (m)	Mobilized friction		Maximum test load	
	ton/m^2	kPa	ton	MN
0-2	0	0	1245	12.21
2-8 8-12	1.54 3.95	15.11 38.75		
12-18	17.91	175.70		

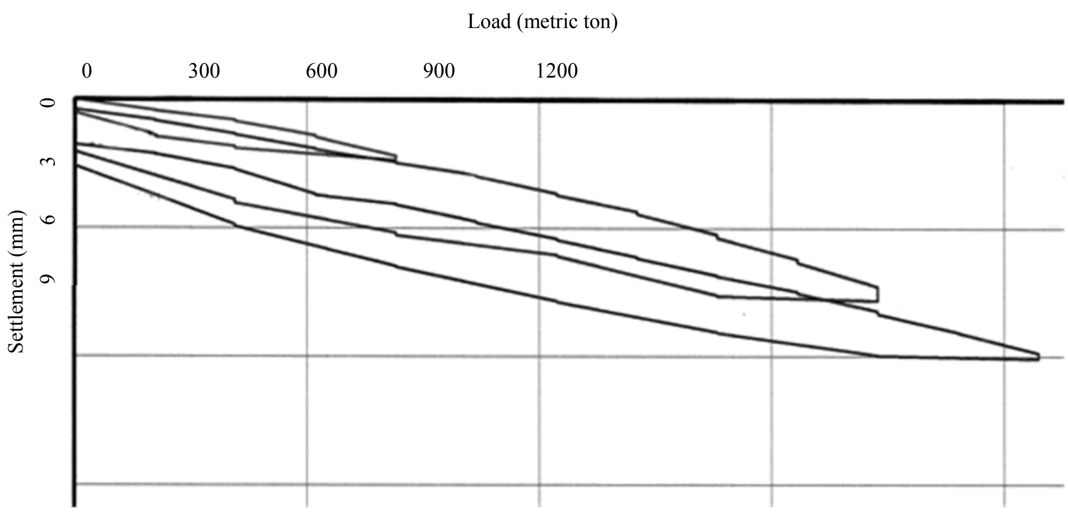

Figure 6. Static pile load test result of 1.2 m-diameter pile.

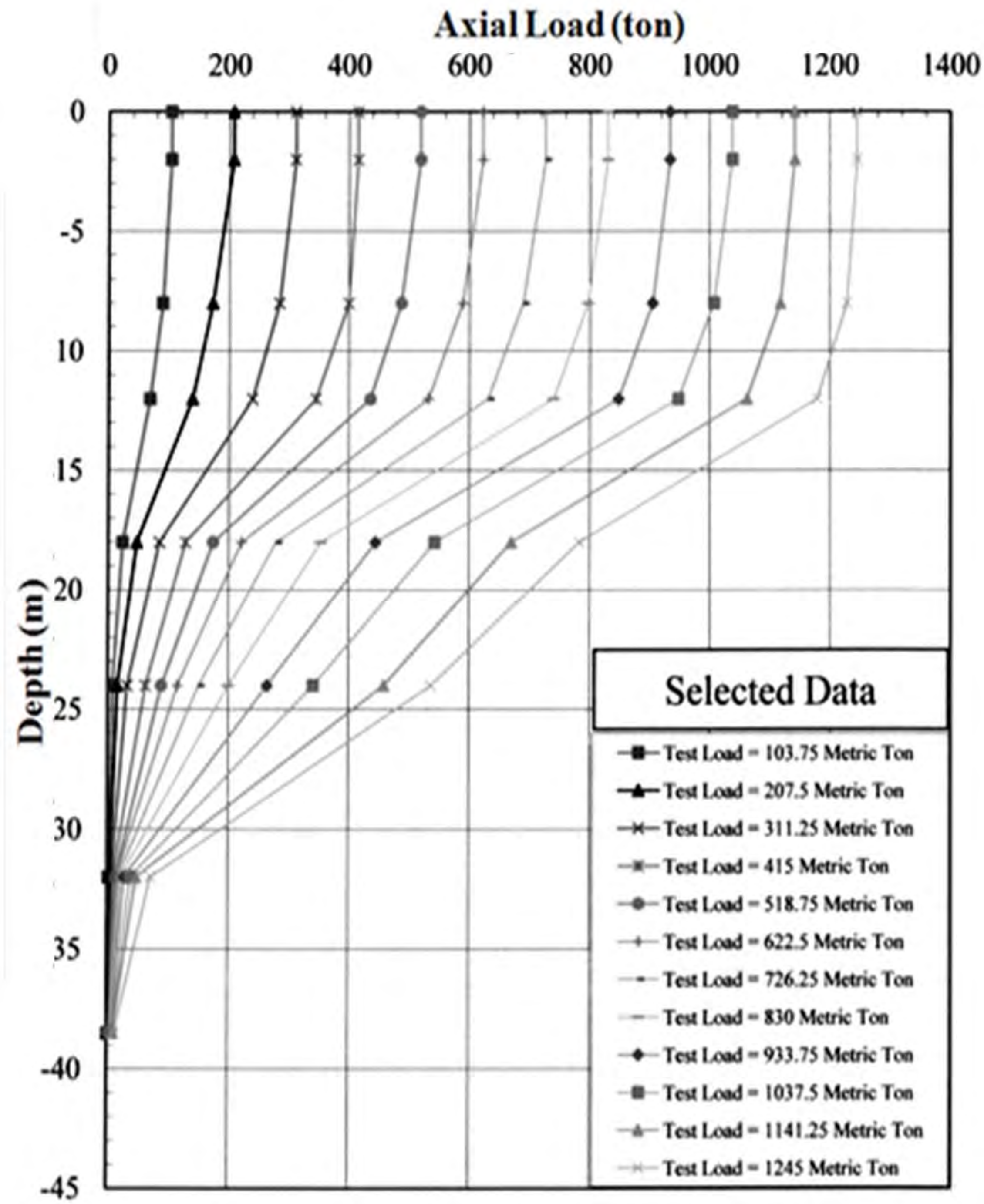

Figure 7. Load distribution along 1.2 m-diameter pile shaft.

Table 3. Input parameters for calculating PLAXIS 3D Foundation program.

Layer No.	Depth (m)		c′ (kPa)	ϕ' (°)	E (kPa)	ν	R_{inter}	g_{unsat} (kN/m^3)	g_{sat} (kN/m^3)	K_0 (1-sin ϕ')	ψ (°)
	From	To									
1	0.0	2.0	1	27.41	5018	0.30	0.75	14	16	0.540	0
2	2.0	8.0	1	28.27	5891	0.30	0.75	16	18	0.526	0
3	8.0	12.0	1	30.33	8509	0.30	0.75	18	20	0.495	0
4	12.0	18.0	1	31.23	10473	0.30	0.75	20	22	0.482	0
5	18.0	24.0	1	29.63	8836	0.30	0.75	18	21	0.506	0
6	24.0	32.0	352	0.00	175823	0.30	0.75	21	23	1.000	0
7	32.0	40.0	1	31.31	14291	0.30	0.75	22	23	0.480	0
Pile	0.0	39.5	-	-	31265388	0.15	1.00	24	–	-	-

3. The Estimation of Skin Friction

Many researchers proposed equations for estimating pile friction (fs) of cohesionless soil. The equations and summarized are concluded in Table 4. In the table, Navg is the average SPT-N along the interval under consideration; z is the distance from pile head to the middle of the interval under consideration. Internal friction angles, ϕ', were estimated from SPT-N by Equation 1 [4]. The ultimate skin frictions of pile estimated from those equations were compared with the mobilized skin taken from the test in Table 5. The calculations were done by using the data from both BH1 and BH2. The calculated frictions shown in Table 5 were the average value. Note that soil borings were done on the same month but 2 years before pile load test was carried out.

$$\phi' \equiv 27.1 + 0.3N_{60}{}' - 0.00045\left((N_1)_{60}\right)^2$$

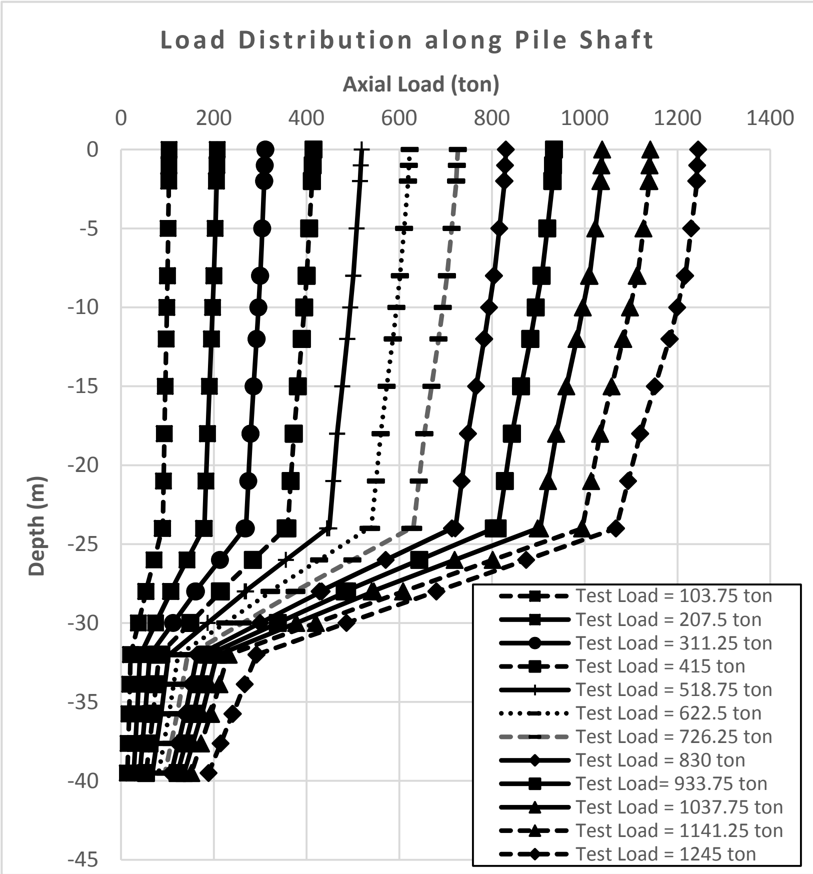

Figure 8. Load distribution along 1.2 m-diameter pile shaft analyzed by PLAXIS 3D.

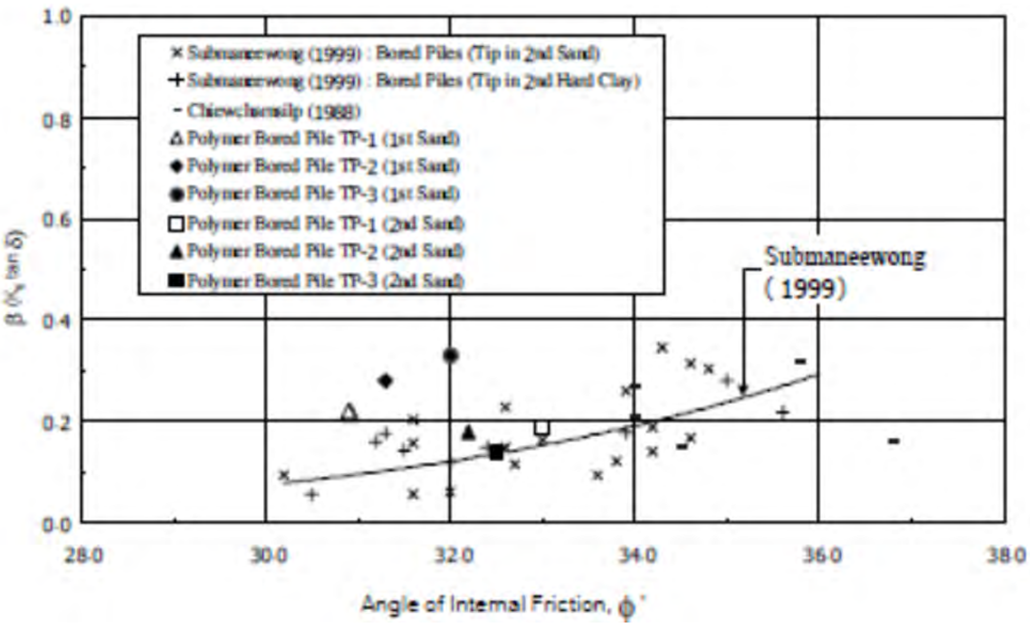

Figure 9. Relation between â and ϕ' . [19]

Table 4. Equations for the estimation of mobilized friction of pile in cohesionless soil.

Researcher	Equation	Remark
[5]	$f_s = K\sigma_z' \tan\phi' < 240$	kPa $K = 0.7$ for $z \leq 7.5$ m $K = 0.6$ for $7.5\text{ m} < z \leq 12$ m $K = 0.5$ for $z > 12$ m
[6]	$f_s = N_{avg}$	kPa
[7]	$f_s = 2.5N_{avg} < 190$	kPa
[8]	$f_s = 2.8N_{avg} < 160$	kPa
[9]	-	Graph
[10]	$f_s = \beta\sigma_z'$	β =0.2, 0.4 and 0.6 for loose, medium dense and dense sand respectively.
[11]	$f_s = (N_{avg}/3)+1$	t/m^2
[12]	$f_s = 3N_{avg}$	kPa
[13]	$f_s = n_s N_{avg}$	kPa; n_s =2, 3 and 4 for loose, medium dense and dense sand respectively.
[14]	$f_s = \beta\sigma_z' \leq 2; 0.25 \leq \beta \leq 1.2$ $\beta = 1.5 - 0.1354z^{0.5}$	ksf σ_z' in ksf z in ft
[15]	$f_s = K\sigma_z' \tan\delta'$	$K = 0.5$, $\delta' \approx 2/3\phi'$
[16]	$f_s = 0.325(2.8N_{avg} + 10)$	kPa
[17]	$f_s = K\sigma_z' \tan\delta'$	$K \approx 0.5$; $\delta' \approx \phi'$
[18]	$f_s = \beta\sigma_z' \leq 200; 0.25 \leq \beta \leq 1.2$ $\beta = (N_{60}/15)1.5 - 0.1354z^{0.5}$for N $\beta = 1.5 - 0.1354z^{0.5}$for $N_{60} \geq 15$	kPa z and B in m
[19]	$f_s = \beta\sigma_z'$	β form Figure 9
[20]	$f_s = K\sigma_z' \tan\delta'$	$K \approx 1 - \sin\phi'$; $\delta' \approx 0.8\phi'$
[21]	$f_s = K\sigma_z' \tan\delta'$	$K \approx 0.73(1 - \sin\phi')$; $\delta' \approx \phi'$

Remark : K is Coefficient of lateral earth pressure.

4. Discussion

Let only 1.2 m-diameter pile be in consideration. The bearing capacities estimated by design engineer and by dynamic load test are 10.18 and 12.87 MN respectively and They are very low when compared with static load test result. The maximum load of 12.21 MN applied on the pile could make fully mobilized friction at only the 18 m upper part of pile shaft. Design engineers usually worry about the construction quality and the decrease of soil strength due to the increase of water content in soil. They are also anxious about non-homogeneity of soil profile and construction quality. Those could result in using high factor of safety. Suspicious result of dynamic load test is in many engineers' mind-the ultimate bearing capacity taken from dynamic load test should not be discussed with the estimated value. Comparing to the test result, [7] equation gave the good prediction for loose and medium dense sand but gave the underestimated value for dense sand. [17] equation gave the good prediction for dense sand but gave the overestimated value for loose and medium dense sand. [10] gave the best predictions that were still too low for dense sand.

[22] reported the underestimated friction obtained by Reese & right equation. In 2007, [23] reported that the skin friction developed in the test pile was greater than the prediction value. [24] also concluded that Meyerhof's equation gave the underestimated value. In contrast, [25] concluded that Meyerhof equation gave the overestimated value. However, the ultimate bearing capacity they used in the discussion was not from the test but estimated from the load settlement curve.

[10] used the low and high â for the soil with low and high SPT-N respectively. It conforms to a conclusion that K varies in direct proportion to OCR [26]. It could be said that K varies in direct proportion to SPT-N as well. OCR of sand might be estimated by Equation 2 [27], σ'_z in kPa. Hence the K value used in Das's equation was changed to be as in Equation 3 [28]. The higher estimated frictions were obtained as shown in Table 5.

$$\mathrm{OCR} = 47\left(N_{60}\right)^{0.8}/\sigma'_z \qquad (2)$$

$$K = 0.5\,\mathrm{OCR}^{0.5} \qquad (3)$$

Table 5. Mobilized friction of pile (kPa) from static load test and the equations in Table 2.

Method	Depth interval (m)		
	2-8	8-12	12-18
Static load test	15.11	38.75	175.70
PLAXIS 3D	11.56	-	-
[5]	26.14	41.47	50.18
[6]	5.17	15.13	25.92
[7]	12.92	37.81	64.79
[8]	14.47	42.35	72.57
[9]	4.00	11.50	22.00
[10]	14.00	48.00	102.00
[11]	26.71	59.27	94.56
[12]	15.50	45.38	77.75
[13]	10.13	45.38	103.67
[14]	13.33	63.89	93.91
[15]	11.86	21.79	31.56
[16]	7.95	17.01	26.83
[17]	37.38	69.12	100.37
[18]	12.12	61.25	95.84
[19]	5.79	15.70	27.31
[20]	17.88	31.66	45.28
Equation 3 was used for K	22.04	52.41	70.68
[21]	13.06	23.11	33.05

The calculation of pile skin friction is based on the assumption that the pile shape is perfectly cylindrical. However, uniform soil-pile interface is quite impossible in construction. Protuberance at the pile surface could increase the bearing capacity of pile. This might be the reason of the underestimation of pile friction in strong soil layer. If the diameter of 12-18 m interval of pile becomes 1.4 m instead of 1.2m, the mobilized friction of this part calculated from mobilized load will decrease about 14 %. This is rough and simple calculation. If there is a protuberance, not only friction but also bearing must be considered. However, in this discussion need to more study for conclusion.

Finite element method gave the different results. The mobilized friction was found only 8 m upper part of pile shaft by applied load 12.21 MN. Skin Friction along pile shaft wase fully mobilized when applied was about 5 times of design load that are demonstrated in Figure 10.

The parameters used in the analysis may be another cause of the inaccuracy of the prediction of pile friction. All parameters were not obtained from the appropriate test, but estimated from by SPT-N by means of empirical relationship. Also, non-homogeneity of natural soil could make the prediction imprecise.

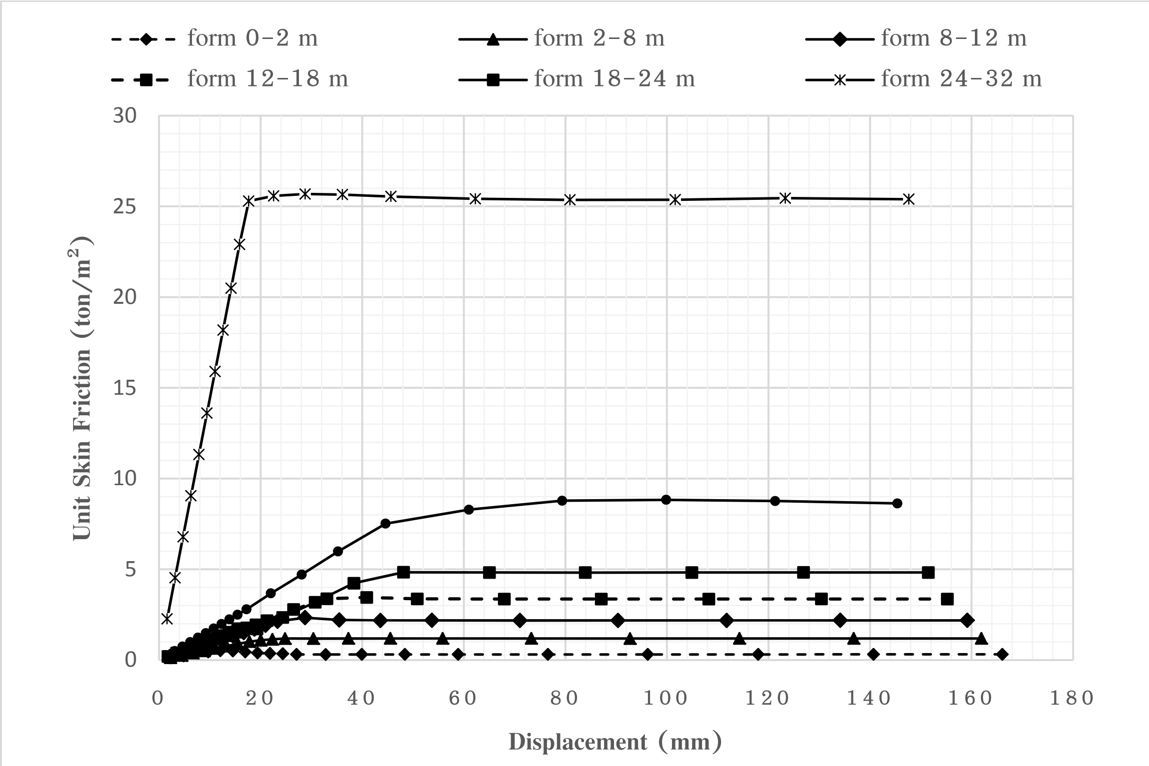

Figure 10. Mobilized skin friction of 1.2 m- ϕ pile shaft analyzed by PLAXIS 3D with 5 times of design load.

5. Conclusion

The equation introduced by [10] shows the results that are better than the other equations. The parameter used in pile friction estimation , K and â , should not be constant but should be related to strength of soil. The suspicious construction quality, the dubious parameters and non-homogeneity of soils are the obstruction of any design equation.

The result presented herein is from only one pile, and hence more research is necessary. In 2015 the other 3 building will be constructed near the building used as a case study in this paper. Eight large bored piles will be tested. More instrumented pile load test is planned to obtain more information for further study.

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Refferences

[1] Gasaluck, W. & Houngjing, S. (2007). Problematic Soil in Northeast Thailand. Development, Advancement and Achievements of Geotechnical Engineering in Southeast Asia: The 40th Anniversary of the Southeast Asia Geotechnical Society, 269-282. Kuala Lumpur.

[2] Phien-wej, N. Pientong, T. & Balasubramaniam, A.S. (1992). Collapse and strength characteristics of loess in Thailand, Eng. Geology 32, Elsevier, 59-72.

[3] Zaw, Z.A. Singtogaw, K & Submaneewong, C. Application of polymer-based slurry for wet-process bored piles construction in multi-layered soil of Bangkok. Proceedings 8th National Convention in Civil Engineering, Khon Kaen, 2002, pp. GTR237-242.

[4] Peck, R.B. Hansen, W.E. & Thornburn, T.H. (1974). Foundation Engineering 2nd ed, Wiley, New York.

[5] Touma, F.T. & Reese, L.C. (1974). Behavior of bored pile in sand, ASCE Journal of the Geotechnical Engineering Division 100, 749-761.

[6] Meyerhof, G.G. (1976). Bearing capacity and settlement of pile foundations, ASCE Journal of the Geotechnical Engineering Division 102, 197-228.

[7] Quiros,G.W. & Reese, L.C. Design procedures for axially loaded drilled shafts, Research Report 176-5F, Project 3-5-72-176, Center for Highway Research, U. of Texas Austin, USA, 1977.

[8] Reese, L.C. & Wright, S.J. Construction procedures and design for axial loading, Drilled shaft manual vol.1. HDV-22, U.S. Department of Transportation, McLean, VA, 1977

[9] Coyle, H.M. & Castello, R.R. (1981). New design correlation for Piles in sand, ASCE Journal of the Geotechnical Engineering Division 107, 965-986.

[10] Davies, R.V. & Chan, A.K.C. (1981). Pile design in Hong Kong. Hong Kong Engineer, March, 21-28.

[11] Decourt, L. Prediction of the bearing capacity of piles based exclusively on N values of the SPT. Proceeding of the Second European Symposium on Penetration Testing, Amsterdam, 1982, pp. 29-34.

[12] Gwizdala, K. (1984). Large diameter bored piles in non-cohesive soils, Research Report No.26, Swedish Geotechnical Institute, Sweden.

[13] Bazaraa, A.R. & Kurkur, M.M. (1986). N-values used to predict settlement of piles in Egypt. Proceedings of In Situ'86, 462-474. New York, USA.

[14] Reese, L.C. & O'Neill, W. Drilled Shafts: Construction Procedures and Design Methods, FHWA-HI-88-042. USA, 1988.

[15] Prakash,S. & Sharma, H.D. Pile Foundations in Engineering Practice, John Wiley & Sons, Inc. USA, 1990.

[16] Decourt, L. (1995). Prediction of load-settlement relationships for foundations on the basis of the SPT-N. Ciclo de Coferencias Internationale, Leonardo Zeevaert. UNAM, 85-104. Mexico.

[17] Brown, R.W. Practical Foundation Engineering Handbook, McGraw-Hill, USA, 1996.

[18] O'Neill, W. M. & Reese, C. L. Drilled Shafts: Construction Procedures and Design Methods, FHWA-IF-99-025. USA, 1999.

[19] Submaneewong, C. Behavior of instrumented barrette and bored piled in Bangkok subsoils. Master thesis, Chulalongkorn University, Bangkok, 1999.

[20] Das, B.M. Principles of Foundation Engineering 7thEd., Cengage Learning, USA, 2011.

[21] Chen, Y.J. Lin, S.S. Chang, H.W. & Marcos, M.C. (2011). Evaluation of side resistance capacity for drilled shafts, Journal of Marine Science and Technology 19, No.2, 210-221.

[22] Beck, W.K. & Harrison, P.J. (2009). Load tests on small diameter augered cast-in-place piles through fill, ASCE Contemporary Topics in Deep Foundations, GSP No. 185, (Eds: Islander, M. Laefer, D.F. & Hussein, M.H.), 430-437. Orlando, Florida, USA.

[23] Vembu, K. & Vipulanandan, C. Side friction development in ACIP test pile and reaction piles in very dense sand. CIGMAT-2007 Conference & Exhibition, Houston, 2007, pp. 1-2.

[24] Brahana, D.C. Wang, J. & Russo, R. Performance of in situ testing method in predicting deep foundation capacity. Proceedings of ISC'98, Georgia, USA, 1998, pp. 1225-1228.

[25] Dass, R.N. & Puri, V.K. Load tests on drilled shafts for highway bridges. Proceedings of 4th International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, 1998, pp. 373-378.

[26] Kulhawy,F.H. & Mayne, P.W. (1982). K0-OCR relationship in soil, ASCE Journal of the Geotechnical Engineering Division 108, 851-872.

[27] Mayne, P.W. (1992). In-situ characterization of Piedmont residuum in eastern US. Proceeding of NSF US-Brazil, Geo-Workshop: Application of Classical Soil Mechanics to Structured Soils. 89-93. Belo Horizonte.

[28] Day, R.W. Geotechnical and Foundation Engineering, McGraw-Hill, USA, 1999