A Review on Design of Pile Foundations in Bangkok

T. Boonyatee[1], J. Tongjarukae[2], T. Uaworakunchai[3] and B. Ukritchon[4]
[1,4]*Department of Civil Engineering, Chulalongkorn University, Bangkok, Thailand*
[2,3] *STS Pile Testing Co., Ltd, Bangkok, Thailand*
[1,2,3,4]*E-mail*: tirawat.b@chula.ac.th

ABSTRACT: A review was made on the design of pile foundations in Bangkok area. Particular attention was paid on design parameters based on local practice which had been reported during 1985 to 2012. An additional study was made on thirty six static load tests on instrumented piles, which were larger and longer than ones in the past. Parameters for the determination of bearing capacity and settlement were back analyzed and compared with those in the literature. Noteworthy studies in the past as well as findings in this study were then summarized and discussed.

Keywords: Pile foundations, Bangkok, Bearing capacity, Settlement, Design parameters.

1. INTRODUCTION

A thick soft clay layer near to the ground surface in Bangkok is famous for its low shear strength and high compressibility. Due to its presence, most of buildings in the city are supported by pile foundations to reduce settlements. Although the primary reason to inclusion of the pile would be to reduce settlement, Randolph (1994) pointed out that a consideration number of pile designs were still carried out by the traditional approach which was to ensure that the structural load can be carried by the piles with an adequate factor of safety against bearing failure.

Based on a survey by Amornfa et al. (2012), 70% of interviewees checked the settlement of piles explicitly. Among various methods proposed in the literatures, the equivalent raft method was found to be the most popular (adopted by 71.4% of the group), followed by various methods including FEA. Nonetheless, the remaining 30% of interviewees still used the traditional approach in their routine designs.

Since the safety factor in the traditional design approach has a subtle role in limiting the settlement of piles, it is crucial to understand assumed conditions of each design parameters. For instance, the N_q value proposed by Meyerhof (1976) was for fully mobilized condition where the deformation of about 10% of the pile diameter has to be expected. Using the Meyerhof's value may lead to a large settlement even the pile would not fail under bearing failure. The current design practice for reducing the settlement of end bearing piles is to limit the end bearing resistance by a prescribed value or divide calculated value with an additional safety factor.

A number of studies on design parameters of piles in Bangkok had been carried out since the early age to the present (Ng, 1983; Sambhandharaksa, 1989; Pimpasugdi, 1989; Submaneewong, 1999; Boonyarak, 2002; Thasnanipan, 2006). Unfortunately, many of them were not widely known or partially adopted. It is not uncommon to see large discrepancy among formula developed from different assumptions. In this study, necessary design parameters, which are α, β and N_q, were reassessed based on data in the literatures and an addition of new thirty six static load tests on recent instrumented piles. The conditions assumed in the literatures were reintroduced and arranged in a uniform manner.

As the construction industry is moving towards performance-based design, the estimation of pile settlement or soil-pile stiffness is more often required than in the past. Normally, the soil-pile stiffness is preferably determined from static pile load tests but empirical or sophisticated methods were also used when pile load tests were not carried out (Amornfa et al., 2012). The suitability of methods proposed by Poulos and Davis (1980) and Terzaghi and Peck (1948) for piles in Bangkok were examined by Sambhandharaksa et al. (1987) by comparing with settlement records during constructions. Instead of relying on geotechnical parameters, Kiattivisanchai (2001) analyzed data from 237 static pile load tests in Bangkok and pointed out that the soil-pile stiffness of bored piles ranged between 0.5EA/L to 4 EA/L where E, A, L are Young's modulus, cross sectional area and length of piles, respectively. In this study, necessary parameters for estimating the settlement of piles were back-calculated from the 36 pile load tests and compared with values reported by Sambhandharaksa (1989). A comparison was also made with other prediction methods (Terzaghi and Peck, 1948; Randolph and Wroth, 1978; Kiattivisanchai, 2001).

2. SUBSOIL CONDITION AND PILING PRACTICE

Bangkok subsoil consists of two parts. The upper part is Bangkok soft to medium marine grey clay that extends to a depth of about 12 – 18 m. The lower parts are alluvial deposits comprised of alternating layers of stiff to hard clay and dense sand. The general properties of soils were reported in the literatures (i.e. Balasubramaniam, 2009).

Table 1 Subsoils relevant to pile foundations in Bangkok

Strata	Depth (m)	Description
Crust	0 – 2	Weathered crust or backfills
Upper layers	1 – 16	Very soft to medium stiff clays
The 1st clay	10 – 25	Stiff to very stiff clays
The 1st sand	14 – 38	Medium to very dense sand
The 2nd clay	24 – 43	Very stiff to hard clays
The 2nd sand	30 – 58	Very dense sand

The soft clay in the upper part is sensitive, highly plastic and highly compressible. This layer has been used as a foundation stratum for buildings in the early days or very light structures. The undrained shear strength, S_u, of the soft clay is a range of 5 – 25 kPa. For normal practice, the S_u value is normally determined from unconfined compressive tests or field vane tests. However, when no reliable data were available, the concept of normalized soil properties (Ladd and Foott, 1974) can be applied to Bangkok soft clay (Sambhandharaksa and Taesiri, 1987, Bergado et al., 2002). The relationship between corrected vane shear strength and OCR of Bangkok soft clay was proposed by Kietkajornkul and Vasinvarthana (1989) as follows;

$$\frac{S_u}{\sigma'_{v0}} = 0.26\left(OCR\right)^{0.8} \tag{1}$$

Most of piles for modern structures are seated on the lower part of Bangkok subsoil. Soil deposits, from the shallowest to the deepest in range of engineering applications, comprise the alluvial brown stiff overconsolidated clay (locally known as the first clay, found between depths of 15 to 33m), the dense silty fine sand (the first sand, found at 35- 40m), the stiff to hard slight overconsolidated clay (the second clay, thickness around 10 – 12 m) and the dense coarse sand with some gravel (the second sand, found at about 60 – 65m).

The stiff clay can be either CL or CH. The S_u ranges between 50 – 140 kPa. The natural water content is between 20 to 40% with liquidity index of less than 30%. The plasticity index of this clay ranges between 10 – 50%. The SPT value ranges between 8 – 40. Pitupakorn (1983) made correlations between uncorrected SPT and unconfined compressive strengths of CL and CH clays based on 86 boring logs. The relationships can be shown as follows;

$$\begin{aligned} S_u &= 0.520N \quad \text{t/m}^2 \quad \text{for CL clays} \\ S_u &= 0.685N \quad \text{t/m}^2 \quad \text{for CH clays} \end{aligned} \tag{2}$$

It is noted that the SPT value in Eq. (2) was taken from safety hammers which are more energy efficient than donut hammer around 25%.

Piles under large loads are normally seated on the first and second sand layers. The first sand layer is found at depth between 20 to 40 m depth. This layer may be absent in some locations (Maconochie, 1998). The first 2-3 m of the first sand layer contains some clay and usually classified as clayey sand (SC) which is then underlain by clean silty sand (SM). The combined thickness of the first sand layer ranges between 3-12 m, occasionally interweaving with stiff clay. The SPT ranges between 18 to more than 100. The upper value tends to be the clean silty sand (Sambhandharaksa and Pitupakorn, 1985). The natural water content is between 10 to 40%. Soil samples closed to the ground surface tend to exhibit high water contents compared to the deeper soil samples. Based on triaxial tests on undisturbed samples collected from an exposed face at about 13 m below natural ground surface of a garbage dump site, it was found that Peck et al. (1974) correlation can be used to determine the effective frictional angle of silty sand (SM) from the SPT value (Sambhandharaksa and Thanudklueng, 1990). Since the same formula over-estimated the frictional angle of clayey sand (SC) and under-estimated the frictional angle of poorly graded clean sand (SP), correlations developed by Pitupakorn (1983) and Meyerhof (1956) were recommended for SC and SP sands, respectively.

The second clay strata is very stiff to hard clay. Its color varies from light grey to grayish brown. The SPT is normally higher than 30 with water content ranging from 15 -22% (Surarak, 2011). The undrained shear strength of this layer is higher than 150 kPa. The second clay layer was reported to be more troublesome than the first clay layer due to higher PI and liquidity index (Sambhandharaksa, 1989).

The second sand layer consists of yellowish brown to brownish grey silty sand and poorly graded sand with silt. The thickness of this layer ranges from 7 m to 30 m. The correlation of its frictional angle to SPT has not been well studied. Currently, the correlations by Peck et al. (1974) is normally used by local soil investigation companies.

Due to excessive uses of underground water since the late 1970s, the piezometric surfaces of the sand layers dropped to around the bottom of the first clay layer, leading to an increase of the effective stresses of around 200 kPa (Phien-wej et al., 2006). Results from pile load tests during this period were apparently higher than regular condition. Some piles were even designed based on increased stresses conditions. Unfortunately, the recent measures show the rising of phreatic surface, resulting in the decrease in effective stresses.

In this study, necessary design parameters, which are α, β, N_c and N_q, were reassessed based on data in the literatures and thirty six pile load test results on recent piles. The effect of ground water draw down was also considered when determining for parameters in sand layers.

3. STUDIES ON PILE LOAD TESTS

A large number of pile load tests had been carried out in Bangkok since 1980s. Back analyses and designed parameters based on those tests were reported by many researches (Pitupakorn, 1983; Pimpasugdi, 1989, Thasnanipan et al., 1998; Submaneewong, 2009; Balasubramaniam, 2009). However, nominal capacities of piles in the literature were not defined by a single criterion. Therefore, it is important to be conscious of assumed conditions and the 'failure' criterion when referring to some calibrated parameters.

Methods for determining the nominal capacity of a pile can be divided into two groups, namely, the yielding criteria and the plunging criteria. A yielding criterion tries to identify the limit load at a characteristic point before plunging. Methods belong to this category are such as Davisson offset limit, slope-tangent method, Butler and Hoy method, L_1-L_2 method, etc. The plunging criterion, on the other hand, tries to identify the maximum capacity of piles. For a pilot pile, the test could be performed until plunging failure occurs. However, it was more often that tests were carried out on working piles, to about 1.5 to 2.5 times of design loads. In the later case, the plunging capacity of piles is obtained by extrapolation. Methods belong to the plunging criteria are such as ones proposed by Fuller and Hoy, Chin, Brinch Hansen, Mazurkiewicz, etc. Definitions and discussions of these methods can be seen elsewhere (i.e. Fellenius, 1980; Hirany & Kulhawy, 1991).

Some criteria found in the local literature were Davisson limit, Butler & Hoy method, Mazurkiewicz method and Chin method. Pitupakorn (1983) proposed correlations between capacities of driven piles and SPT value based on 43 pile load tests. His database comprised 34 piles, of 250 – 600 mm diameter, resting on the first clay layer and 9 piles rested on the clayey sand on the top of the first sand layer. Data of piles seating on the clean silty sand layer was not available at that time. Pitupakorn adopted the minimum slope criteria (Vesic, 1963), to determine the nominal capacity from load testing results and assumed piezometric drawdown condition in his study. For large diameter spun piles (600 – 800 mm diameter), Balasubramaniam et al. (2003) reported that a scale factor of 1.2 shall be applied to Pitupakorn's formula. However, it is noted that the criteria used in the later study was based on Mazurkiewicz method.

One of the most referred studies in the local community was done by Pimpasugdi (1989). A result of practical interest from her study was the relationships between design parameters of bored, driven and auger press piles. In her work, the following assumptions were used;

- The failure criteria proposed by Butler and Hoy (1977) was used.
- The undrained shear strength of clay was taken from unconfined compression test or estimated from SPT by correlations proposed by Pitupakorn (1983).
- The N_c value of 9.0 was used to estimate ultimate end bearing capacity of piles having their tips in very stiff to hard clay. This leaded to estimated end bearing capacities of 1.0 to 2.2 MPa.
- The frictional angle of sand was estimated by Peck (1974), Pitupakorn (1983) and Meyerhof (1956) as described earlier.

The relationships between adhesion factor (α) and undrained shear strength of Bangkok clay is shown in

Figure 1. From the figure, it can be seen that the estimated values are rather scatter and not significantly affected by the construction method of piles.

The ultimate skin friction of sand, on the other hand, is highly affected by pile construction methods. Chiewchansilp (1988) studied on bored piles having their tips in the first sand layer and reported that the relationship between β and ϕ' by Meyerhof (1976) can be used. For driven piles, a conservative design by assuming at rest condition and $\tan\delta = \tan\phi'$ may be used. It was noted that the error in estimating sand skin friction of driven piles is negligible due to the limited penetration into the first sand layer (Sambhandharaksa, 1989).

For end bearing resistance of sand, Pimpasugdi (1989) calibrated the 'mobilized' N_q value with Butler and Hoy's failure loads. Her N_q value was not for estimating the fully mobilized end resistance where the deformation of about 10% of the pile diameter is expected (Sambhandharaksa, 1989).

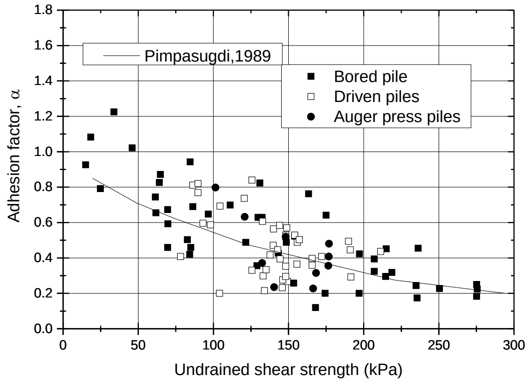

Figure 1 Relationship between adhesion factor and undrained shear strength of Bangkok clay (after Pimpasugdi, 1989)

The proposed value, as shown in Figure 2, was recommended to use with the gross safety factor of 2.0 – 2.5 without an additional factor which usually applied to the end bearing resistance's portion for reducing the deformation at failure. It is noted that there are also a number of designers in Thailand who use the fully mobilized values but clamping the upper limit for end bearing pressure to about 3.0 MPa or applying a partial factor to the end bearing resistance term. When compared to Meyerhof's value, Pimpasugdi's value was smaller by about three times. This factor may be considered as a guideline when a fully mobilized N_q value will used.

After 2000s, the use of polymer in stabilizing fluid became common among large bored pile contractors. Due to the reduction in thickness of filter cake along pile shaft and the ease of base cleaning, skin resistance in sand layers and end bearing resistance of piles built with polymer modified slurry are normally better than those built with pure bentonite slurry. Based on eight pile load tests, Boonyarak (1999) reported that the addition of polymer had no effect on the skin resistance of clay layers but increased the skin friction of the 1st sand layer by about 80% (see Figure 3). The improvement on the skin friction in the 2nd sand layer and on the end bearing resistance of piles were, however, not observed in his study as they were not fully mobilized when tests stopped.

4. AN ADDITIONAL STUDY ON 36 PILE TESTS

In this study, data from 36 static load tests on recently constructed piles were analyzed based on slope-tangent, Butler & Hoy and Mazurkiewicz criteria. All piles in this study were instrumented with vibrating wire strain gauges at 4 – 10 depths. Rod extensometers were also installed at pile tips to determine the elastic shortening of piles. The size, length and important parameters are summarized in Table 2.

4.1 Nominal capacity determination

Load-displacement curves of piles can be approximately described by mathematic equations. For long piles (*D/B* > 20), Phoon et al. (2006) reported that the load-displacement curves could be normalized by the hyperbolic equation shown in Eq. (3). In this study, the Levenberg–Marquardt algorithm was used for non-linear curve fitting of the observed load-displacement curves with Eq. (3). Unfortunately, results seemed to be highly dependent on the initial search values. When Eq. (4) which has fewer parameters was used instead, consistent results could be obtained.

$$\frac{Q}{Q_{st}} = \frac{x}{a+bx} \tag{3}$$

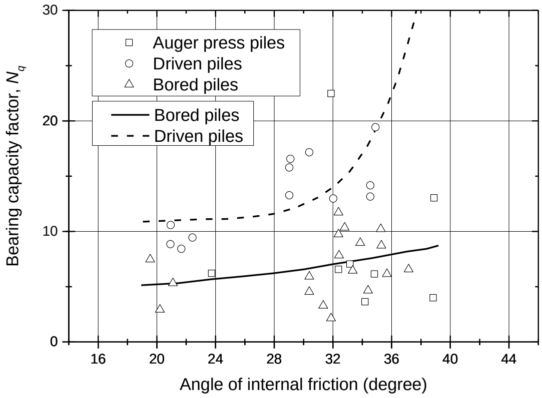

Figure 2 Relationship between bearing capacity factor (N_q) and frictional angle of Bangkok sand (after Sambhandharaksa, 1989)

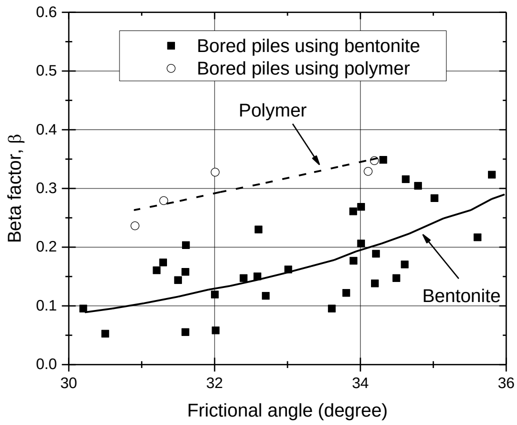

Figure 3 Relationship between β factor and frictional angle of Bangkok sand (after Boonyarak, 1999)

$$Q = a\left(1-e^{-bx}\right) \tag{4}$$

where Q = load, Q_{st} = Slope-tangent capacity, x = settlement, a and b are fitting parameters.

Actually, Eq. (4) fits with the procedure proposed by Mazurkiewicz which assumed two constraints as follows;

- the relationship between the second derivative of load with respect to settlement and the first derivative of load with respect to settlement is linear (Eq. (5)).

- the maximum capacity occurs when the first derivative of load with respect to settlement becomes zero (Eq. (6)).

$$\frac{d^2Q/dx^2}{dQ/dx} = -b \tag{5}$$

$$\frac{dQ}{dx} = 0 \tag{6}$$

Physical meanings of fitting parameters in Eq. (4) are shown in Figure 4. It is noted that the parameter a can be slightly different from original Mazurkiewicz capacity as the fitting algorithm equally weighs each data point while the graphical method tries to fit only the final part of the curve. This discrepancy was observed only on few cases on barrette piles where the measured curves were closed to bilinear than exponential function. Nonetheless, we found that this compromise results in a better approximation of the initial stiffness of piles. Once the fitting parameters were determined, settlements at nominal capacities can be determined analytically from formula shown in Table 3.

Table 2 Summary of pile properties and important parameters

No.	Pile type	Slurry type	Dia. (m)	Length (m)	Soil at pile tip	Settlement (mm)				Load (MN)						Error (%)
						x_w	x_p	x_{st}	x_{BH}	Q_w	Q_p	Q_{st}	Q_{BH}	Q_{Mz}	Q_{est}	
1	BP	P	1.5	56.5	S	7.2	43.8	28.0	50.8	10.79	27.86	25.08	27.81	28.38	32.18	13
2	BP	P	1.5	56.5	S	6.0	83.6	27.9	48.9	8.34	24.23	21.31	23.53	24.08	27.26	13
3	BP	P	1.5	49	S	4.5	25.5	27.3	47.1	8.34	25.02	24.02	26.26	26.78	21.53	-20
4	BP	P	1.5	52	S	4.8	70.5	26.6	43.3	8.34	25.02	21.89	23.58	24.07	23.43	-3
5	BP	P	2.0	58.5	S	3.4	13.2	28.8	38.1	11.77	30.41	36.59	37.42	37.84	42.13	11
6	BP	P	1.5	52	S	5.9	27.4	27.8	49.4	9.81	25.11	24.19	26.69	27.25	31.29	15
7	BP	P	1.2	55	S	4.3	12.0	21.7	32.9	7.26	14.52	17.57	18.60	18.97	20.92	10
8	BP	P	1.0	55	S	4.4	17.1	21.2	36.3	5.00	12.51	13.39	14.84	15.25	17.58	15
9	BP	P	1.2	53	S	4.3	38.5	28.4	66.3	6.38	25.51	22.40	27.62	28.36	20.91	-26
10	BP	P	1.5	53	S	6.4	32.0	27.7	50.7	11.77	29.43	27.86	30.77	31.34	27.44	-12
11	BP	P	1.5	47	S	11.3	149.8	28.3	49.2	11.77	29.43	19.32	21.43	21.98	23.95	9
12	BP	P	1.5	64.5	C	8.2	33.8	31.4	68.3	11.77	29.43	27.92	33.10	33.87	36.55	8
13	BP	P	1.5	55	S	3.5	47.7	27.8	51.0	7.85	31.39	27.85	30.81	31.38	28.68	-9
14	BP	P	1.0	55	S	5.3	34.0	29.7	81.9	4.91	19.62	17.72	24.41	25.35	18.33	-28
15	BP	P	1.5	55	S	7.4	33.5	29.4	58.5	11.77	29.43	28.09	32.02	32.68	29.59	-9
16	BP	P	1.5	60	S	7.4	46.2	28.8	54.9	11.77	29.43	25.85	29.13	29.74	33.98	14
17	BP	P	1.5	60	S	7.0	151.9	26.5	44.1	11.77	29.43	25.47	27.45	27.95	27.94	0
18	BR	B	1.00*3.00	60	S	5.2	193.7	19.3	34.6	17.66	44.15	34.82	37.35	37.72	53.25	41**
19	BR	B	1.00*3.00	60	S	3.5	96.0	17.8	28.3	17.66	44.15	41.32	42.93	43.23	53.25	23
20	BP	P	1.2	60	S	4.8	12.4	22.5	35.9	6.87	13.73	17.01	18.30	18.70	20.26	8
21	BP	P	1.2	47.1	S	5.9	21.1	22.6	35.6	7.85	15.70	15.94	17.15	17.55	18.44	5
22	BR	B	0.80*2.00	50	C	7.4	153.9	21.9	55.8	12.26	44.15	24.45	30.56	31.18	22.64	-27
23	BR	B	0.80*2.00	47.5	S	7.0	183.2	20.4	47.2	12.26	44.15	23.14	27.65	28.18	21.48	-24
24	BP	P	0.6	35	C	1.8	15.5	15.3	21.3	1.32	4.41	4.45	4.79	5.04	4.06	-19
25	BP	P	0.6	35	C	1.6	79.0	13.1	13.7	1.10	3.73	3.69	3.71	3.87	3.07	-21
26	BR	B	1.00*3.00	48	C	3.7	10.1	24.8	68.8	14.03	28.06	49.21	61.13	61.88	45.64	-26
27	BP	P	2.0	55	S	8.0	28.1	32.7	56.7	21.58	43.16	43.64	46.93	47.56	47.73	0
28	BP	P	1.5	56.2	S	7.4	21.1	30.0	60.0	10.79	21.58	24.97	28.74	29.42	23.73	-19
29	BP	P	1.8	52.4	S	6.1	23.4	27.7	38.9	14.03	28.06	28.85	29.89	30.31	33.68	11
30	BP	P	1.5	51.9	S	7.8	156.0	25.1	35.2	10.68	21.37	16.84	17.63	18.03	21.20	18
31	BR	P	0.80*2.50	52.8	S	3.8	33.0	17.1	30.5	9.77	24.44	22.87	24.77	25.10	27.81	11
32	BR	B	1.20*3.00	66.0	S	98.1	302.0	24.8	56.1	48.76	58.57	45.95	52.73	53.34	61.55	15
33	BR	B	1.20*3.00	56.0	S	98.5	203.4	20.2	29.0	28.45	32.70	26.86	27.79	28.11	46.70	66**
34	BR	B	1.20*3.00	65.8	S	14.8	24.6	26.2	68.7	48.76	65.39	66.80	79.49	80.24	61.55	-23
35	BR	B	0.80*2.50	50.1	C	3.7	76.6	15.1	21.3	12.75	24.03	23.12	23.79	24.02	22.70	-5
36	BR	B	0.80*2.50	53.6	S	6.2	168.5	17.1	29.9	12.80	23.95	21.24	22.97	23.30	28.47	22

Remarks: BP = Bored pile, BR = Barrette pile, P = Polymer modified slurry, B = Bentonite slurry, S = Sand, C = Clay
For Q and x; w = working condition, p = maximum tested load, st = slope-tangent, BH = Butler & Hoy, MZ = Mazurkiewicz
Q_{est} = Prediction by parameters proposed in this study, Error = $[Q_{est} - Q_{Mz}] / Q_{Mz}$
** Piles having large prediction error which will be discussed in section 6

Table 3 Closed-form solutions for nominal capacities from the curve fitted model.

Criteria	Formula for settlements	Formula for capacities
Slope-tangent	$x_{st} = \dfrac{W\left(-e^{-c}\right)+c}{b}$; $\quad c = b \cdot x_{offset} + 1, W(x) =$ Lambert W function	$Q_{st} = a\left(1-e^{-bx_{st}}\right)$
Fuller & Hoy	$x_{FH} = \dfrac{\ln(FHR \cdot ab)}{b}$	$Q_{FH} = a\left(1-e^{-bx_{FH}}\right)$
Butler & Hoy	$x_1 = \dfrac{\left(Q_{FH}\right)\cdot FHR - x_{FH}}{(ab)\cdot FHR - 1}$; $x_{BH} = \dfrac{-1}{b}\ln\left(1-\dfrac{Q_{BH}}{a}\right)$ $x_{BH/2} = \dfrac{-1}{b}\ln\left(1-\dfrac{Q_{BH}}{2a}\right)$	$Q_{BH} = ab\left(x_{BH}\right)$
Mazurkiewicz	∞	$Q_{Mz} = a$

Remarks: Q and x are in consistent units.
FHR = Fuller & Hoy's limit rate = 0.05 inch/ton, x_{offset} = Slope-tangent offset = 0.15 inch + Pile's diameter/120

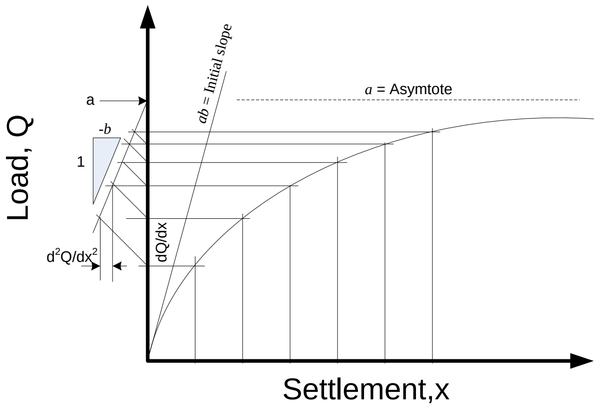

Figure 4 Mazurkiewicz criterion and the corresponding mathematic model used in this study

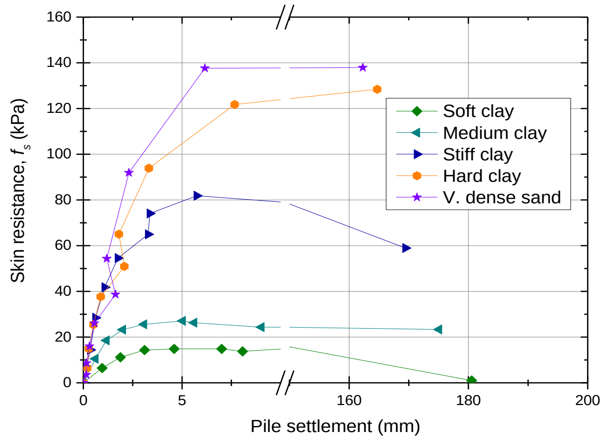

Figure 5 Mobilization of skin resistance compared to pile head settlement

4.2 Mobilization of skin friction

To determine the maximum skin resistance along pile shaft, the skin resistance determined from strain gauges of each soil layer was plotted with the settlement at pile head. An example was shown in Figure 5. The skin resistance was considered to be fully mobilized when there was no change in skin resistance with respect to pile settlement or softening of skin resistance occurred. In this study, only fully mobilized skin resistances were used to determine the α and β parameters.

4.3 Skin resistance of clay layers

The relationship between fully mobilized skin resistance with SPT's uncorrected N value is shown in Figure 6. It was agreed with past studies that the skin resistance of clay layers is not affected by the type of stabilizing slurry. The skin resistance may be estimated directly from the uncorrected N value from the equation shown in the same figure.

As shown in Figure 7, the relationship between the adhesion factor (α) and the undrained shear strength was rather scattered. However, the overall trend was consistent with those in the literature. In this study, the observed data seemed to fit best with the curve of Stas and Kulhawy (1984). However, it is still recommended to follow Pimpasugdi (1989) when the S_u is less than 50 kPa. This modification results in a more conservative designs for short piles and negligible reduction of the skin friction in the soft clay for long piles.

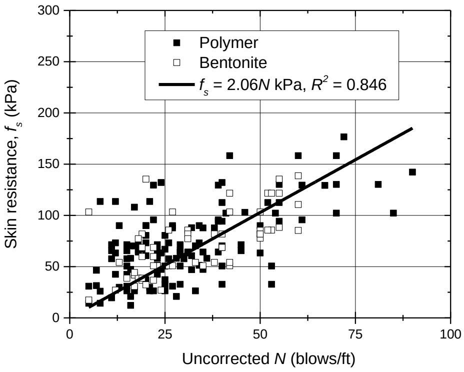

Figure 6 Relationship between skin resistance vs. uncorrected N value of Bangkok clays

4.4 Skin resistance of sand layers

The relationship between fully mobilized skin resistance and uncorrected N value of each sand type is shown in Figure 8. In addition to correlations shown in the same figure, regression analyses were also done for all combinations of sand types and slurry types. Results from the analyses, which are the slope of linear regression lines, are summarized in Table 4.

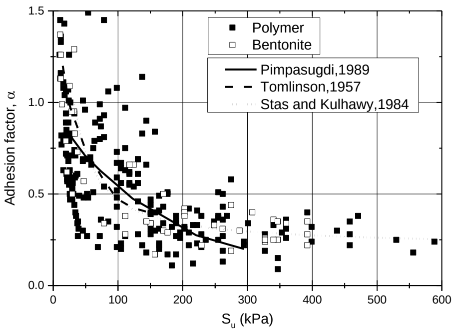

Figure 7 Relationship between adhesion factor (α) vs. undrained shear strength of Bangkok clays

Table 4 Results of regression analyses for skin resistance vs. uncorrected N for Bangkok sands

Sand / Slurry		All sands (1)	SM (2)	SC (3)
All types	(a)	2.15	2.17	1.81
Polymer	(b)	2.29	2.31	1.66*
Bentonite	(c)	1.84	1.84	1.82

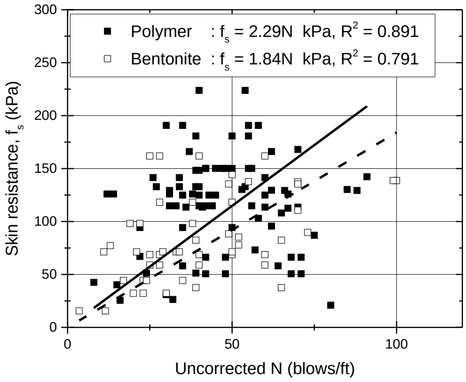

Figure 8 Relationship between skin resistance vs. uncorrected N value of Bangkok sands

From Table 4, the following conclusions were made;

- When bentonite slurry was used, skin resistance was not affected by sand types (cf. row (c)).
- When polymer modified slurry was used, skin resistance of SM sand was higher than the value of SC sand (cf. row (b)).
- With an exception for the Polymer-SC combination, skin resistances of piles built with polymer modified slurry were higher than those built with bentonite slurry (compared row (b) with row (c)). While the number of data point in other cases ranged between 13 – 126, there were only three data for the Polymer-SC case. Therefore, it was inconclusive to justify that the skin resistance of Polymer-SC case is lower than Bentonite-SC case.

Based on above observations, the following equations are recommended for the estimation of skin resistance from the uncorrected N value.

for SM sand

$$f_s = 2.31N \text{ kPa} \quad \text{for polymer modified slurry} \tag{7}$$

$$f_s = 1.84N \text{ kPa} \quad \text{for bentonite slurry} \tag{8}$$

for SC sand

$$f_s = 1.81N \text{ kPa} \quad \text{for all slurry types} \tag{9}$$

It is noted that Eqs. (7) to (9) were calibrated under phreatic drawn down condition and are given here for the sake of convenience for comparing with other empirical formula in the literature. Since the β method assumes that f_s varies linearly with the effective overburden pressure (σ_v') but the uncorrected N in Eqs. (7) to (9) is linearly proportional to the square root of σ_v', estimates from both approaches will not be consistent for all stress conditions.

To back analyze for parameters used in the β method, the skin resistance of sand layers was related to the beta factor (β), coefficient of lateral earth pressure (K_s), a reduction ratio (R), the effective frictional angle (ϕ') and effective overburden pressure (σ_v') by

$$\begin{aligned} f_s &= K_s \cdot R\tan(\phi') \cdot \sigma_v' \\ &= \beta \cdot \sigma_v' \end{aligned} \tag{10}$$

Based on fully mobilized skin resistance and effective overburden pressure under drawdown condition, the beta factor (β) can be back calculated as shown in Figure 9. The average value of β for polymer modified and bentonite slurry are 0.30 and 0.20, respectively. When compared to Boonyarak (1999), the values for bentonite slurry were in the same range. However, the values for polymer modified slurry in this study, which based on more data points, were higher than Boonyarak's value.

For bored piles, Stas and Kulhaway (1984) suggested that the ratio K_s/K_0 ranges from 0.67 to 1.0. Therefore, it was assumed for further back-analyses that

$$K_s = 1 - \sin\phi' \tag{11}$$

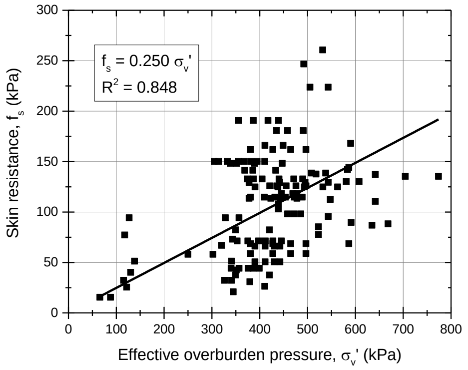

Figure 9 Relationship between skin resistance vs. effective overburden pressure of Bangkok sands

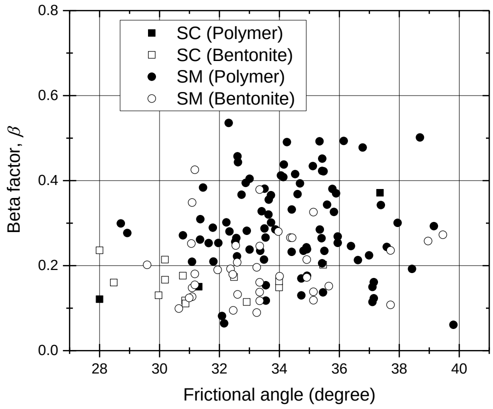

Figure 10 Relationship between β factor vs. frictional angle of Bangkok sands

Using Eqs. (10) and (11), the reduction ratio can be calculated from the data shown in Figure 10. To further determine the relationship between the reduction ratio and frictional angle, two prediction models defined by Eq. (12) and Eq. (13) were used in this study. Based on comparisons between two prediction models for all combinations of sand and slurry types by Akaike information criteria (AIC) and Bayesian information criteria (BIC), it was concluded that model 2 was more preferable over model 1. Therefore, only fitting parameters from model 2 will be discussed.

Model 1:
$$R = a \cdot \mu = a \cdot \tan(\phi) \tag{12}$$

Model 2:
$$R = b \tag{13}$$

From values in Table 5, the influence of sand and slurry types on the skin resistance were more consistent than the correlations made earlier with the SPT value. It can be seen that;

- The reduction ratio of SM case was higher than SC case for a similar slurry type.
- The reduction ratio of Polymer case was higher than Bentonite case for a similar sand type.

Therefore, four values in the lower right corner of Table 3 are recommended for the estimation of skin friction in sands.

Table 5 Reduction ratios for Bangkok sands

Slurry \ Sand	All sands (1)	SM (2)	SC (3)
All types (a)	0.88	0.92	0.59
Polymer (b)	1.00	1.01	0.73
Bentonite (c)	0.68	0.72	0.56

4.5 End bearing resistance

From Table 2, it can be seen that x_{st} ranges between 13.1 to 32.7 mm. Its average value of 24.5 mm was close to 25.4 mm, which is one of popular criteria applied in practice. For this reason, slope-tangent criterion was considered in this study as a suitable criterion for conventional design approaches when the settlement should be limited to a small value. In addition, a calibration was also made with loads defined by Mazurkiewicz criterion which is considered as a suitable method to determine the plunging capacity of piles.

$$Q_{e,st} = Q_{st} - Q_s = q_{e,st} A_e \tag{14}$$

$$Q_{e,Mz} = Q_{Mz} - Q_s = q_{e,Mz} A_e \tag{15}$$

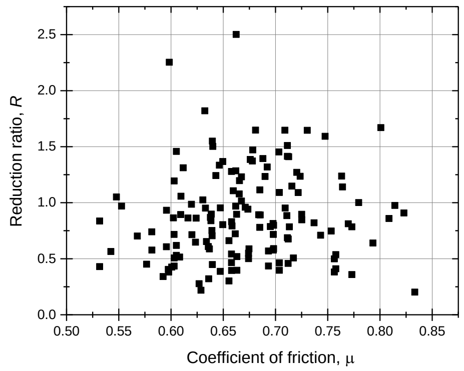

Figure 11 Relationship between the reduction ratio (R) vs frictional angle of Bangkok sands

The mobilized end bearing load of a pile can be calculated from where N_q is a bearing capacity factor, q_e is mobilized end bearing pressure, σ_v' is the effective stress under draw down condition at pile tip and A_e is the cross section area of pile tip.

In this study, the mobilized end bearing loads at Q_{st} and Q_{MZ} were estimated by subtracting loads at pile head by skin resistances calculated by Eqs. (10), (11) with the reduction ratio of 1.01, 0.72, 0.73 and 0.56 for corresponding cases shown in Table 5. Mobilized end bearing pressures were then obtained by dividing mobilized end bearing loads by cross sectional area of piles. From the distribution of $q_{e,st}$ and $q_{e,Mz}$ shown in Figure 12, it can be seen that end bearing pressures were less than 6 MPa when the slope tangent criterion was used. On the contrary, values as high as 12 MPa can occurred under nominal loads defined by Mazurkiewicz. It is noted that negative values in Figure 12 did not represent suction pressures. They were merely calculation artifacts which occurred when applying loads were not high enough to fully mobilize skin resistance and activate the end bearing resistance of piles.

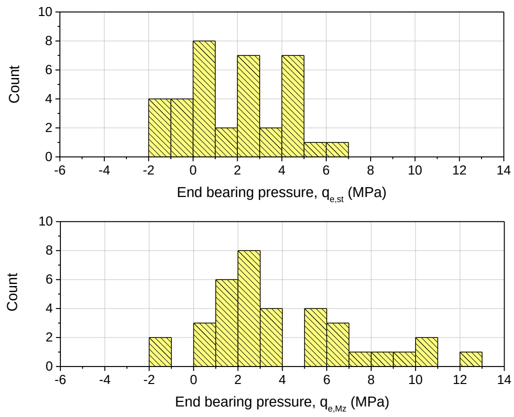

Figure 12 Distribution of mobilized end bearing pressures at nominal loads.

4.6 End bearing resistance of clay layers

Based on the calculated end bearing pressures and Eq. (16), bearing capacity factor for slope-tangent criteria ($N_{c,st}$) and Mazurkiewicz criteria ($N_{c,Mz}$) were determined.

$$q_e = N_c S_u \tag{16}$$

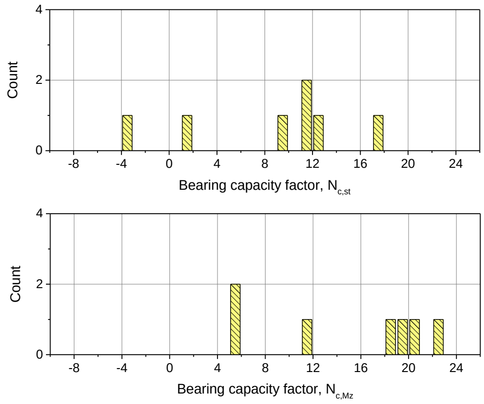

Figure 13 Distribution of N_c calibrated for each criterion

As shown in Figure 13, $N_{c,st}$ ranged between -4 to 18 while $N_{c,Mz}$ ranged between 5 to 23. Although low N_c values were observed in two cases, a typical value of 9 may be applied for $N_{c,Mz}$ in designs providing that borehole bases are properly cleaned. Based on ratios between observed $N_{c,st}$ and $N_{c,Mz}$ values, it is recommended to use a value of 5.4 for $N_{c,st}$ in designs. In addition, maximum limits of 6 MPa and 12 MPa may be applied in determining $q_{e,st}$ and $q_{e,Mz}$ for the safety's sake.

4.7 End bearing resistance of sand layers

Bearing capacity factors, N_q for slope-tangent criteria ($N_{q,st}$) and Mazurkiewicz criteria ($N_{q,MZ}$), were determined from end bearing pressures in section 4.5 and Eq. (17).

$$q_e = N_q \sigma_v^{'} \tag{17}$$

Relationships between N_q and frictional angle shown in Figure 14 and Figure 15 were rather scattered. However, they were agreed with Sambhandharaksa (1989). Since the frictional angle in this study varied over a narrow range, the relationship with the frictional angle was not clearly seen. The averaged values of 2.6 for $N_{q,st}$ and 5.8 for $N_{q,Mz}$ may be used for estimating the end bearing pressure at the nominal loads when the frictional angle is between 32 to 38 degree. Again, maximum limits of 6 MPa and 12 MPa may be applied in determining $q_{e,st}$ and $q_{e,Mz}$.

5. Prediction error of proposed parameters

Since some fitting parameters were compromised during study by various reasons described earlier, the performance of proposed model was reassured by comparing prediction results with measured values (cf. Q_{est} and Q_{Mz} in Table 2). As seen from the last column in Table 2 and Figure 16, prediction errors ranged from -28% and 66%.

When an investigation was carried out on two cases on the right of Figure 16, it became clear that predictions for barrette piles were inferior to those of bored piles. When predictions for barrette piles were excluded, prediction errors ranged from -27% and 17%.

Since the end bearing resistance of barrette piles was comparable to the skin resistance, their load-displacement curves seemed to be bi-linear rather than assumed exponential function. An example of load-displacement curve for pile no. 33, which had the largest error of 66%, is shown in Figure 17. It can be seen from the figure that the pile could carry more load but the fitted curve became flat at the Q_{Mz} of 28.1 MN. Although further studies on barrette piles using different fitting functions will be done, the proposed parameters still gave satisfactory results when using with safety factors in a range of 2.0 – 2.5.

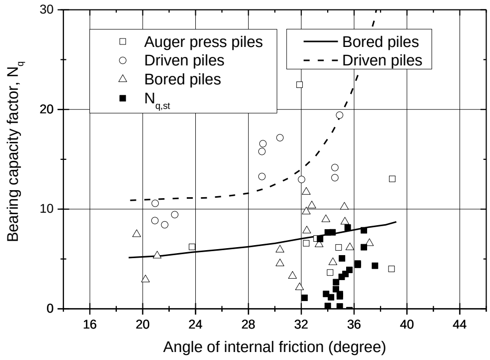

Figure 14 Relationship between mobilized $N_{q,st}$ and frictional angle

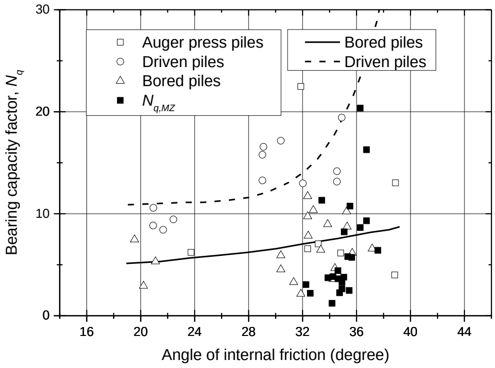

Figure 15 Relationship between mobilized $N_{q,Mz}$ and frictional angle

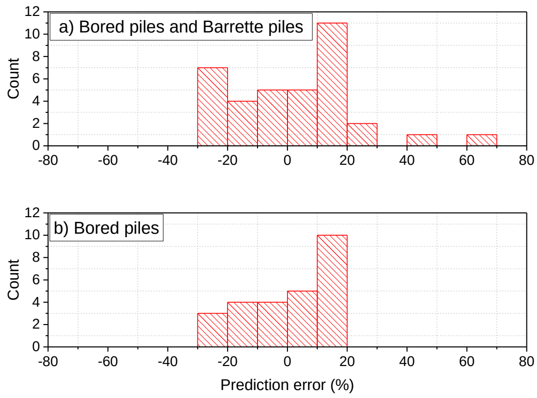

Figure 16 Prediction errors of proposed parameters

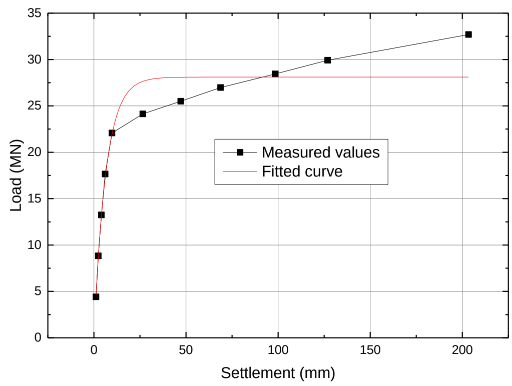

Figure 17 Load-displacement curves of a barrette pile (no. 33)

6. CONCLUSION

In this study, empirical relations of practical interest and their assumed conditions were reviewed. Necessary design parameters, which are α, β, N_c and N_q, were reassessed based on data in the literatures and thirty six pile static load tests on bored piles recently constructed in Bangkok subsoils.

For driven piles, the adhesion factor of Pimpasugdi (1989), the β factor of Meyerhof and the N_q of Sambhandharaksa (1989) are recommended to estimate nominal capacities defined by Butler and Hoy. The original investigators of these empirical formula recommended a global safety factor of 2.0 – 2.5 for their formula.

For bored piles, the relation proposed by Stas and Kulhawy (1984) gave a better match for the skin resistance in clays than Pimpasugdi (1989). However, it is still recommended to follow Pimpasugdi (1989) when the S_u is less than 50 kPa. This modification results in a more conservative designs for short piles. In addition, the skin resistance of clay can be directly estimated from the uncorrected N value by

$$f_s = 2.06N \quad \text{kPa} \tag{18}$$

For the skin resistance of sand, the β method is recommended over empirical formula that relates with SPT value. The skin resistance can be estimated by

$$\begin{aligned} f_s &= \beta\sigma_v^{'} \\ \beta &= \left(1-\sin\phi'\right) R\tan(\phi') \end{aligned} \tag{19}$$

where ϕ' is the frictional angle of sand and R is the reduction ratio of 1.01, 0.72, 0.73 and 0.56 for combinations of SM-Polymer, SM-Bentonite, SC-Polymer and SC-Bentonite, respectively (see Table 4). For the estimation of end bearing resistance of piles having tips in sand, a constant N_q of 2.6 is recommended for slope tangent criterion and a constant N_q of 5.8 is recommended for Mazurkiewicz criterion. For the estimation of end bearing resistance of piles having tips in clay, N_c of 5.4 is recommended for slope tangent criterion and N_q of 9 is recommended for Mazurkiewicz criterion. Maximum limits of 6 MPa and 12 MPa may be applied in determining $q_{e,st}$ and $q_{e,Mz}$.

Predictions by the proposed parameters are recommended to use with a safety factor in range of 2.0 – 2.5. When pile settlements are concerned, parameters for slope tangent criterion should be used instead of those for Mazurkiewicz criterion.

When compared with measured values, prediction errors of proposed parameters ranged between -28% to 66%. However, when predictions for barrette piles were excluded, the range of error reduced to -28% to 17%.

9. REFERENCES

Amornfa, K., Phienwej, N. and Kitpayuck, P. (2012) "Current practice on foundation design of high-rise buildings in Bangkok, Thailand", Lowland Technology Interaltional, Vol. 14, No. 2, pp 70-83.

Balasubramaniam, A., Oh, E. Y.-N. and Phienwej, N. (2009) "Bored and driven pile testing in Bangkok sub-soils", Journal of Lowland Technology International, 11(1), pp. 29-36.

Bergado, D. T., Balasubramaniam, A., Fanni, R. J., & Holtz, R. D. (2002). "Prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the new Bangkok International Airport project", Canadian Geotechnical Journal, 39(2), pp. 304-315.

Boonyarak, T. (2002) "Behavior of polymer slurry for wet-process bored pile construction in Bangkok subsoils", Master thesis, Chulalongkorn University, Bangkok.

Butler, H.D. and Hoy, H.E. (1977) "Users Manual for the Texas Quick-Load Method for Foundation Load Testing", Federal Highway Administration, Office of Development, Washington, 59 pp.

Chiewcharnsilp, S. (1988) "Static load transfer and predictions of carring capacity of bored piles" (in Thai), Master thesis, Chulalongkorn University, Bangkok.

Fellenius, B. H. (1980). The analysis of results from routine pile load tests. Ground Engineering, 13(6), 19-31.

Hirany, A., & Kulhawy, F. H. (1991) "Conduct and interpretation of load tests on drilled shaft foundations: Detailed guidelines (Vol. 1)", Electric Power Research Institute.

Kiattivisanchai, S. (2001) "Evaluation of seismic performance of an existing medium-rise reinforced concrete frame building in Bangkok", Master thesis, Asian Institute of Technology, Bangkok.

Kietkajornkul, C., & Vasinvarthana, V. (1989) "Influence of different vane types on undrained strength of soft Bangkok clay", Soils and Foundations, 29(2), pp. 146-152.

Kulhawy, F., & Chen, J. (2005) "Axial Compression Behavior of Augered Cast-In-Place (ACIP) Piles in Cohesionless Soils" Advances in Deep Foundations, ASCE, pp. 1-15.

Kulhawy, F., and Hirany, A. (2009) "Interpreted Failure Load for Drilled Shafts via Davisson and L_1-L_2", Contemporary Topics in Deep Foundations, ASCE GSP 185, pp. 127-134.

Ladd, C. C., & Foott, R. (1974) "New design procedure for stability of soft clays", Journal of the Geotechnical Engineering Division, 100(7), 763-786.

Maconochie, D. J., Schulz, C., & Tachavichitpaisarn, S. (1998) "Some geotechnical aspects of design of the initial system project in Bangkok". Urban ground engineering, Hong Kong.

Meyerhof, G.G. (1976) "Bearing Capacity of Settlement of Pile Foundations", J. SMFE, ASCE, Vol. 102, GT-3, pp. 195-228.

Meyerhof, G.G. (1956). "Penetration tests and bearing capacity of cohesionless soils", Journal of the Soil Mechanics and Foundation Division, 82(1), pp. 1-19.

Ng, K.C. (1983) "The construction problems and performance of large bored piles in second sand layer", Master thesis, Asian Institute of Technology, Bangkok.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. (1974) "Foundation Engineering", John Wiley and Sons. , New York.

Phoon, K., Chen, J., & Kulhawy, F. (2006) "Characterization of Model Uncertainties for Augered Cast-In-Place (ACIP) Piles under Axial Compression", Foundation Analysis and Design, ASCE, pp. 82-89.

Pimpasugdi, S. (1989) "Performance of bored, driven and auger press piles in Bangkok subsoil", Master thesis, Asian Institute of Technology, Bangkok.

Pitupakorn, W. (1983) "Prediction of pile carrying capacity from standard penetration test in Bangkok metropolis subsoil", Master thesis, Chulalongkorn University, Bangkok.

Poulos, H.G. and Davis, E.H. (1980) "Pile foundation analysis and design", John Wiley and Sons, New York.

Randolph, M.F. (1994) "Design methods for pile groups and piled rafts", Thirteenth International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, Vol. 5, pp. 61-82.

Randolph, M.F. and Wroth, C.P. (1978) "Analysis of deformation of vertically loaded piles", J. Geot. Eng. Div., ASCE 104(12), pp. 1465-1488.

Sambhandharaksa, S. (1989) "Recent Piling Practice in Bangkok Plain", Proceedings of the Seminar on Engineering for Coastal Development, Bangkok, Vol. 4, pp 5-105 to 5-118.

Sambhandharaksa, S. and Pitupakorn, W. (1985) "Prediction of prestress concrete pile capacity in Bangkok stiff clay and clayey sand", Proceedings of the 8th Southeast Asian Geotechnical Conference" Seminar on Engineering for Coastal Development, Bangkok, Vol. 4, pp 5-105 to 5-118.

Sambhandharaksa, S., Pongrujidorn, T. and Chaloeywares, P. (1987) "Settlement of structures and their predictions in Bangkok subsoils", Proceedings of the 9th Southeast Asian Geotechnical Conference, Bangkok, Thailand, pp. 4-25 – 4-40.

Sambhandharaksa, S. and Taesiri, Y. (1987), "Development of Theory and Practice in Geotechnical Engineering", Theme Lecture No. 1, 8th ARC on SMFE, Kyoto, Vol. 2.

Sambhandharaksa, S. and Thanudklueng, C. (1990) "Modulus and Strength of Bangkok Sand Related to SPT Correlations", Proceedings of the 10th Southeast Asian Geotechnical Conference, Bangkok, Taipei, Taiwan, pp. 457 - 462.

Stas, C.V. and Kulhawy, F.H. (1984) "Critical evaluation of design methods for foundations under axial uplift and compression loading", Report EL-3771, Electric Power Research Institute, Palo Alto, CA, pp. 7-9.

Submaneewong, C. (1999) "Behavior of instrumented barrette and bored piled in Bangkok subsoils", Master thesis, Chulalongkorn University, Bangkok.

Surarak, C. (2011). Geotechnical aspects of the Bangkok MRT blue line project. Ph. D. Thesis, Griffith University.

Terzaghi, K. and Peck, R.B. (1948) "Soil Mechanics in Engineering Practice", John Wiley and Sons. , New York.

Tomlinson, M.J. (1957) "The adhesion of piles driven in clay soils", Proc. of 4th Int'l Conf. on Soil Mech. And Found. Eng., London, 2, pp. 66-71.

Thasnanipan, N., Aye, Z. Z., Submaneewong, C. and Boonyarak, T. (2006) "Application of top-down construction method for deep excavations in urban area of Bangkok", Int'l Symposium on Underground Excavation and Tunneling, Bangkok.

Thasnanipan, N., Tanseng, P., & Anwar, M. (1998) "Large diameter bored piles in multi-layered soils of Bangkok", Proc. of the 3rd Intl. Geotechnical Seminar on Deep Foundations on Bored and Auger Piles, Ghent, Belgium, Balkema.