Deep Barrette Pile Capacity with Aging Effect

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ABSTRACT: Two instrumented barrette piles of the highest high-rise building in Bangkok city were tested in order to verify the ultimate pile capacity. Their sizes were approximately 1.2x3.0x66m depth and their pile tips seated in the second very dense silty sand layer. The first tested barrette pile (TP1) was tested at 43 days after pile completion while 72 days curing time was allowed for the second tested barrette pile (TP2). Aging Effect of barrette pile was observed. Barrette pile number TP1 showed low loading capacity with high settlement while higher loading with lower settlement was recorded in the tested barrette pile number TP2. The value of adhesion factor (α) and friction factor (β) of the first tested pile (TP1) were studied and it shows well agreement with typical values of Bangkok soil while the second showed that its values were at higher bound of the typical.

KEYWORDS: Barrette pile, Aging effect, Friction factor, Adhesion factor, Bentonite slurry, Polymer slurry

1. INTRODUCTION

Generally, pile load test either static or dynamic test is to be carried out very soon after completion of pile (usually 28 days) due to cost of waiting. The result of pile tests is used to be a representative for long-term pile capacity which might be underestimated since surrounding soil around pile is disturbed during pile construction and can be recovered over time. The topic of time effect in pile or aging of pile has been researched for some decades since pile capacity was highlighted to be increased with curing time in order not to underestimate pile capacity. Past studies mostly aim to measure the difference in capacity over time and to study the mechanism in case of driven pile in sand and some in clay. Schmertmann (1991) and Chow et al. (1996) reported soil aging mechanism. In general, excess pore pressure is generated due to pile installation. This pore pressure will dissipate over time giving soil to reconsolidate and thus results in an increase in capacity.

There were some case studies of pile load test which aging effect can be found clearly. Chow et al. (1998) carried out static load tests on driven pile six months and five years after installation. The result showed significant increase in pile shaft capacity up to 85 percent. An example of pile driven in cohesive soil test can be found in Chen et al (1999). Piles were tested after 3 days and 33 days after completion. About 70 percent increase in shaft friction was observed. Another study of static tension tests was presented by Jardine and Standing (2000) at Dunkirk. Over 100 percent of shaft friction has been developed from 10 days to 250 days of curing time.

However, unlike driven pile, behavior and mechanism of aging effect in bored pile may be different due to some extent factors; for instance, boring process and stabilizing slurry. During drilling process, surrounding soil around borehole may be disturbed. Over time, strength of disturbed soil may be regenerated. Moreover, stabilizing agent may also play an important role in time effect of pile. Most of bored pile required bentonite slurry to stabilize borehole which promotes bentonite film at the interface between soil and pile. This film affects shaft friction of pile. The cake film is inevitably unavoidable as it helps in protecting fluid loss, water leakage and borehole collapse. Nevertheless, the use of polymer instead of bentonite has been introduced recently as it was found that the effect of the cake interface is much less than conventional one (Brown et al., 2002) and overall performance especially in shaft capacity of pile is better (Thasnanipan et al., 2002).

The mechanism and cause of lost and gain in strength over time in bored pile was described by Wardle et al. (1992). During pile boring process, soil softening is promoted because of stress relief and also excess water which is given off during concrete curing. Further consolidation of softened soil during drilling process and during initial constructing period is expected. This consolidation leads to an increase in pile capacity. Wardle et al. (1992) also performed constant rate of penetration test (CRP) on in-situ bored pile in cohesive soil 63 days after installation. An increase in overall capacity of 47% between the first and the last CRP test (over 1000 days were left after the first CRP). Back calculation was done to determine bearing coefficient (Nc) and adhesion factor (α) and it was found to be higher over time. Thus, aging effect does not only contribute to shaft fiction but also end bearing in case of clay. However, it is still conflicted that the pile might penetrate into more intact layer and thus gives more base resistance.

Another example of aging effect cast-in-place pile was found at London Heathrow Terminal 5 (Unwin and Jessep, 2004). CRP test was carried out on four drilled piles to find peak and critical stage load. The result of this project does not present definite pattern of an increase of peak shaft resistance. The first test pile which was tested at two weeks after testing showed very high load resistance while much lower load can be applied in the second test pile which has been waited for 12 weeks after completion. The rest from the second pile to the forth follow aging effect theory. In this case, aging effect was neither assured nor disprove because of scatter of peak resistance. Additionally, typical curve of critical stage resistance was drawn. Critical stage load seems to be decrease over curing time. Due to the fact that cost of drilled pile construction is expensive, the study is very limited.

Apart from circular pile, there is also another pile type; a rectangular bored pile or so-called barrette pile, which has been widely being used due to its high capacity. Aging effect is also observed in barrette pile at high-rise building project in Thailand. This is a very first case study of time effect in barrette pile capacity in Bangkok since pile load test is normally only be carried out immediately 28 days after casting due to the cost of waiting time. This paper presents testing process and discusses the result of different barrette pile static load test. The mechanism supposes to similar to circular bored pile as it is constructed by wet process boring. Therefore, this result may also be used in case of circular bored pile where bentonite is employed as a stabilizing slurry.

2. CASE STUDIES OF BARRETTE PILE IN THAILAND

The topic of barrette pile in Thailand has been recently studied much more intensive than in the past as the demand of barrette pile is increasing in the building foundation construction as well as the foundation of viaduct and station of Bangkok Mass Rapid Transit (MRT) system because of the fact that barrette pile gives higher loading capacity than circular bored pile and foundation plan can be easily arranged. The limitation of land has also push the use of barrette pile forward. In general, bentonite slurry is used as the stabilizing agent during construction which promotes a thick bentonite film along pile shaft and soft base problems as colloid in the slurry settles to the base of borehole. However, these problems can be solved by doing base grouting (Teparaksa et al., 1999).

Unlike general bored pile, bored hole of barrette pile is in rectangular shape which can lead to failure because of corner effect; therefore, polymer slurry in this case cannot be used.

Teparaksa (2001) intensively studied barrette pile behavior in Bangkok subsoil. As previously mentioned, using bentonite rises the issue of soft base problem which can be tackled with base grouting. Performance of grouted and non-grouted barrette pile was investigated. Back calculation from shaft friction capacity showed typical value of adhesion factor (α) and friction factor (β) as well as bearing coefficient (Nq) of both grouted and non-grouted barrette pile. Comparing charts of capacity parameters were presented. Moreover, these values of grouted barrette pile were significantly higher than those of non-grouted pile. Surprisingly, toe grouting in barrette can help in gaining more shaft friction capacity as much greater friction factor was observed. Hence grouting gives higher shaft capacity. Effect of this increase in unit friction factor can be seen clearly in sand layer. Furthermore, bearing coefficient of barrette pile was found to be very close to zero. It can be implied that end bearing has been lightly mobilized. Therefore, it was also concluded that bearing capacity of barrette pile can be neglected.

The performance of base grouting was confirmed by Teparaksa (2011). Tests on fully instrumented barrette piles with and without toe grouting in MRT Purple Line were carried out. Shaft friction capacity here again was back calculated for capacity parameters. Results including friction factor, adhesion factor and bearing coefficient were then compared with Teparaksa (2001). It was revealed that all the result agreed well with previous one. The typical values of these designing parameters were also proposed and it was used to evaluate aging effect in this paper.

In order to study aging effect in Barrette pile, two static load tests were conducted on instrumented barrette piles at high-rise building project in the heart of Bangkok city business center. One of those piles was tested at 43 days after casting and 72 days were allowed for curing time of the second pile. Strain was monitored in each soil layer and was correlated into shaft capacity which was used for back analysis. Both adhesion factor and friction factor were computed and compared with Bangkok typical values from past works. This paper describes setting of barrette pile test and presents test result of two barrettes which have different curing time. The performance and behavior of barrette pile with different curing time is discussed.

3. SOIL CONDITIONS

Soil investigation was carried out in order to design pile length and pile capacity. Undisturbed sample was taken in soft clay and SPT test was done in stiff clay and sand layer. Five different locations were bored. Results were interpreted and examined by geotechnical engineers. The worst borehole as it gives lowest pile capacity was selected to be a representative for the whole construction site. Thus, all barrette piles were designed based on this borehole data.

The soil stratuma consists of 13.5 thick soft to medium clay at the top and followed by very stiff silty clay. First medium dense to dense silty sand layer is reached at -22.7m. The second dense silty sand layer is encountered at 53.5m deep where tip of barrette piles is seated at about 66m below ground surface. There is a 16m. thick dense silty sand and a 4.5m. thick hard silty clay layer in the middle between the first and the second sand layer. Below barrette pile tip, there is once again hard silty clay layer which followed by very dense silty sand. The borehole is ended at -87.3m.

The soil conditions can be summarized in Figure 1. Strength of soil is showed in term of N-value from SPT test except for the first soft to medium clay layer as it was sampled and tested. Unconfined compression test was done for this layer and hence the result is presented in term of shear strength. Unit weight of soft to medium clay fluctuates ranging between 16.0-16.5 kN/ m^3. Unit weight of the rest of soil strata is 20.0 kN/ m^3.

Bangkok ground water condition was investigated in MRT Blue Line Construction Project in late 1997 (Teparaksa, 1999). It was found that ground water has been drawdown because of deep well pumping in the past as there was no government regulation. Thus, Bangkok piezometric level had been being lowered to about -23m from ground surface. This phenomenon helps promoting an increase in effective stress as presented in Figure 2. In addition, higher effective stress in sand gives higher shaft and base capacity.

Soil investigation also showed that recorded ground water level is at -0.95m below ground level.

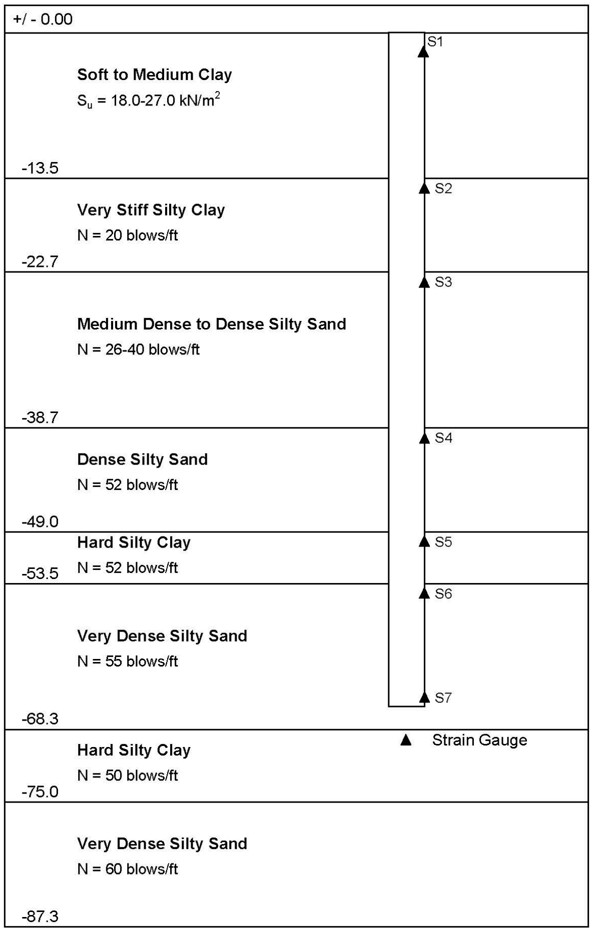

Figure 1 Soil stratuma and strain gauges installed in barrette piles

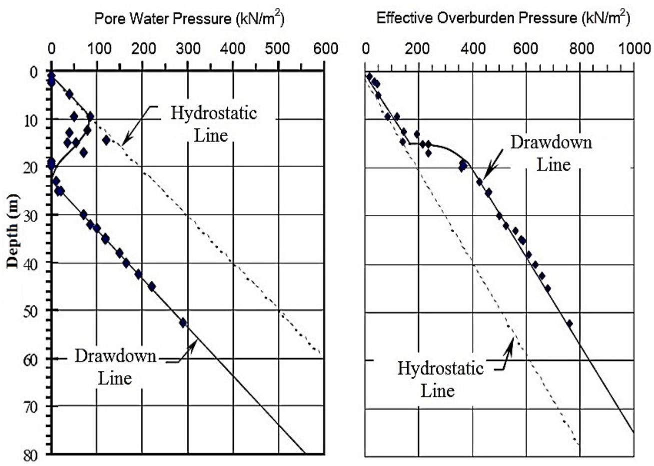

Figure 2 Ground water level and vertical effective stress of Bangkok subsoils (Teparaksa, 1999)

4. ESTIMATED BARRETTE PILE CAPACITY

Both shaft friction and end bearing are taken into account when estimating barrette pile capacity. Ultimate single barrette pile capacity can be calculated using following Formula (1).

$$Q_{ult} = Q_f + Q_b \tag{1}$$

where

Q_{ult}	=	Ultimate pile capacity (kN)
Q_f	=	Pile shaft capacity (kN)
Q_b	=	Bearing capacity (kN)

Pile bearing capacity of the bored pile or barrette pile with pile tip in clay and in sand was derived from the following approach

$$Q_b = N_c S_u \quad \text{:(For clay layer)} \tag{2}$$

$$Q_b = N_q \sigma'_{v(toe)} \quad \text{:(For sand layer)} \tag{3}$$

where

S_u	=	Undrained shear strength of clay (kN/m^2)
N_c	=	Bearing coefficient for clay layer
N_q	=	Bearing coefficient for sand layer
$\sigma'_{v\,(toe)}$	=	Effective overburden pressure at pile toe (kN/m^2)

Pile shaft capacity is

$$Q_f = f_s p \Delta L \tag{4}$$

where

f_s	=	Unit pile shaft friction (kN/m^2)
ΔL	=	Length of pile (m)
p	=	Pile perimeter (m)

The unit skin friction (f_s) of bored pile or barrette pile is generally estimated either from

$$f_s = \alpha S_u \quad \text{:(For clay layer)} \tag{5}$$

or

$$f_s = k_s \tan\delta(\sigma'_{v(mid)}) = \beta\sigma'_{v(mid)} \quad \text{:(For sand layer)} \tag{6}$$

where

f_s	=	Unit pile shaft friction (kN/m^2)
S_u	=	Undrained shear strength of clay (kN/m^2)
α	=	Adhesion factor for clay
δ	=	Angle of friction for sand
β	=	Friction factor for sand
$\sigma'_{v\,(mid)}$	=	Effective overburden pressure at the middle of layer (kN/m^2)

Because of the fact that unconfined compression test was carried out just only in the first clay layer in this case study, strength of clay in below layer is evaluated by using SPT test. In order to estimate shear strength of overlaid hard clay layer to fill in Eq. (5), a corrected formula between SPT N-value and shear strength has to be employed. Pituprakorn (1983) reported this correlation for Bangkok subsoils as follows

$$S_u = 6.85N \tag{7}$$

where

S_u	=	Undrained shear strength of clay (kN/m^2)
N	=	SPT N-value (Blow/ft)

5. BARRETTE PILE TEST OF HIGH-RISE BUILDING PROJECT

5.1 Description of barrette pile

The barrette pile was designed as a pile foundation of 310m height high-rise building in the heart of Bangkok city. Two static barrette pile tests were carried out on pile number TP1 and number TP2 to ensure pile capacity and also compare with design value. There are two buildings in this project; high-rise building and podium. Location of tested barrette pile TP1 and TP2 is in high-rise zone as presented in Figure 3. TP1 is located at left side of the site while TP2 is at the middle. Surrounding barrettes were used as anchor support. Design load of both piles is 24.85 MN with the same reinforcement. The dimension of both barrette piles were 1.2x3.0 m. and their tips were both penetrated into the second very dense silty sand layer. In order to investigate pile behavior, vibrating wire strain gauges (VWSG) were attached at seven levels as illustrated together with soil conditions in Figure 1. The selected levels were 2.0, 14.5, 24.0, 40.0, 50.0, 54.5 and 65.0m deep below ground level. Rod extensometer is also installed at -65.00 m. depth in TP1 and at -64.76 m. depth for TP2 (one meter above pile tip). Detailed description and construction timing of barrette piles is summarized in Table 1 and Table 2. Photo of barrette pile test in this site are shown in Figure 4 and Figure 5.

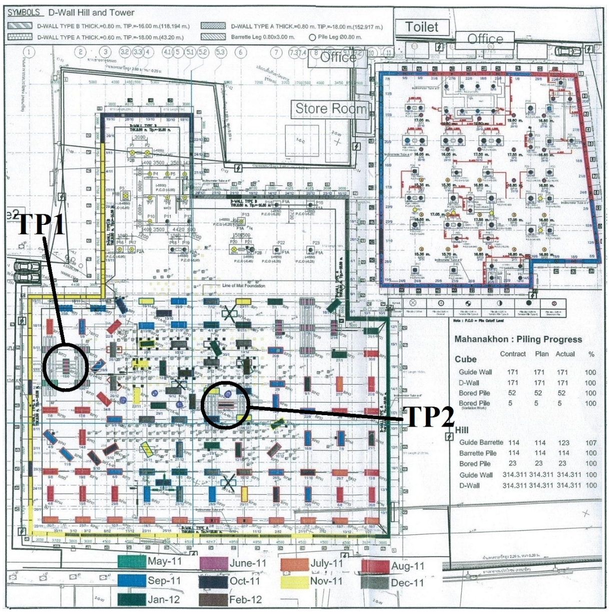

Figure 3 Location of barrette pile TP1 and TP2 in high-rise building project

Figure 4 Photo of barrette pile test

Figure 5 Photo of barrette pile test

Table 1 Description of barrette pile

Item	**Barrette Pile Number and Description**	
	TP1	**TP2**
Size	1.20x3.00 m.	1.20x3.00 m.
Pile Type	Cast in place, Barrette pile	Cast in place, Barrette pile
Date of Concreting	Jun 12th, 2011	Oct 8th, 2011
Date of Testing	Jul 25th-27th, 2011	Dec 20th-22nd, 2011
Curing Time (days)	43	72
Ground Level	+/- 0.00 m.	+/- 0.00 m.
Cut off Level	-8.90 m.	-0.26 m.
D-Wall tip Elev.	-66.00 m.	-65.76 m.
Instrumentation	14 Nos. of VWSG 1 Rod Extensometer	14 Nos. of VWSG 1 Rod Extensometer
Pile Toe Treatment	None	None
Stabilizing Slurry	Bentonite 5% Volume	Bentonite 5% Volume
Viscosity (sec)	39.5	32
Density (g/cm^3)	1.06	1.06
Sand Content (%)	2.50	1.00
pH Value	9.5	10.0

Table 2 Construction time of barrette pile

Item	**Barrette Pile Number and Description**	
	TP1	**TP2**
Excavation		
Start	8.00 – 10 Jun 2011	6.30 – 5 Oct 2011
Finish	22.00 – 11 Jun 2011	11.00 – 7 Oct 2011
Time Consumed	20.00 Hours	20.00 Hours
Desanding		
Start	22.30 – 11 Jun 2011	19.00 – 7 Oct 2011
Finish	9.30 – 12 Jun 2011	2.50 – 8 Oct 2011
Time Consumed	11.00 Hours	7.50 Hours
Reinforcement		
Start	11.00 – 12 Jun 2011	15.30 – 8 Oct 2011
Finish	15.30 – 12 Jun 2011	19.30 – 8 Oct 2011
Time Consumed	4.30 Hours	4.00 Hours
Concreting		
Start	17.00 – 12 Jun 2011	21.40 – 8 Oct 2011
Finish	19.32 – 12 Jun 2011	4.20 – 9 Oct 2011
Time Consumed	2.32 Hours	6.40 Hours
Design Volume	207.96 m^3	234.00 m^3
Actual Volume	251.00 m^3	255.00 m^3

Although all variables except curing time were tried to keep similar for both piles, there are some parameters which were out of control; for example, viscosity, pH value and sand content of bentonite slurry. However, Thasnanipan et al. (1998) reported that viscosity of bentonite does not have influence on axial capacity. Those pH values of two barrettes can be considered similar as they were not much different. Sand content is generally proportion of sand particle in slurry volume during drilling process; therefore, this parameter would not have significant effect on barrette capacity compared to curing time.

In the same way, construction time of barrette was carefully monitored. Excavation time was exactly the same between the two barrettes. Time needed for reinforcement installation and actual volume of concrete used of barrettes can be considered similar to each other. Desanding process aimed to clean the bored hole; therefore how long it took does not affect performance of pile. Time consumed in concreting TP1 was much longer than TP2 due to traffic management; however, the duration is still acceptable as it was not longer than concrete setting time.

5.2 Description of barrette pile test

Barrette pile testing was conform to ASTM D 1143/D 1143M-07 item 8.1.3 procedure B (maintain test) and Item 8.1.4 procedure C (loading in excess of maintained test) (ASTM, 2007). Four surrounding barrettes were used as a reaction pile in order to withstand the maximum test load. Steel girder was laid across test pile and fasten to these surrounding anchor piles. Load was applied to the test pile against reaction girders by using hydraulics jacks with capacity of 5 MN (Figure 6). Ball bearing was inserted in the middle of girder and hydraulic jack to eliminate moment transfer.

There were two cycles of pile load test performed on each pile. The first cycle was normal loading up to 2 times of design load or 49.70 MN while the second was quick load test of which maximum test load is 70.00 MN or failure. In cycle 1, 12.5% of two-timed design load is maintained in each step and is to be increased after one hour until reaches maximum. It is noted that if the rate of settlement exceeds 0.25 mm./hr., load has to be kept constant for two hours. Maximum load is controlled for 12 hours. If settlement is not over 0.25 mm./hr., load is to be released every one hours until recovery rebound stop otherwise maximum load is to be prolonged to 24 hours before unloading. For the second cycle, each load and unload is maintained for 20 minutes. Table 3 and Table 4 show loading sequence of TP1 and TP2 respectively. Figure 7 and Figure 8 present loading sequence with time.

The barrette pile number TP1 was tested at 43 days after pile construction while the barrette pile number TP2 was tested at 72 days after pile completion. Base or initial value of data was monitored one day prior to testing. During testing, each of instrumentation was recorded at each of the load increments and decrements.

Figure 6 Photo of hydraulic jacks

Table 3 TP1 Loading sequence

Item	Loading Sequence (MN)
Cycle 1	0 → 6.2125 → 12.4250 → 18.6375 → 24.8500 → 31.0625 → 37.2750 → 41.3500 → 43.4875 → **49.7000** → 43.4875 → 37.2750 → 31.0675 → 24.8500 → 18.6375 → 12.4250 → 6.2125 → 0
Cycle 2 (Quick Test)	0 → 12.425 → 24.850 → 37.275 → 49.700 → 49.700 → 50.950 → 52.000 → 53.000 → 55.490 → 56.000 → 56.800 → 57.150 → 57.800 → 58.300 → 58.600 →59.000 → 59.500 → 59.700 → **60.000** → 45.000 → 30.000 → 15.000 → 0

Table 4 TP2 Loading Sequence

Item	Loading Sequence (MN)
Cycle 1	0 → 6.2125 → 12.4250 → 18.6375 → 24.8500 → 31.0625 → 37.2750 → 43.4875 → **49.7000** → 43.4875 → 37.2750 → 31.0675 → 24.8500 → 18.6375 → 12.4250 → 6.2125 → 0
Cycle 2 (Quick Test)	0 → 12.425 → 24.850 → 37.275 → 49.700 → 52.185 → 54.670 → 57.155 → 59.640 → 62.125 → 64.610 → **66.660** → 50.250 → 33.500 → 16.750 → 0

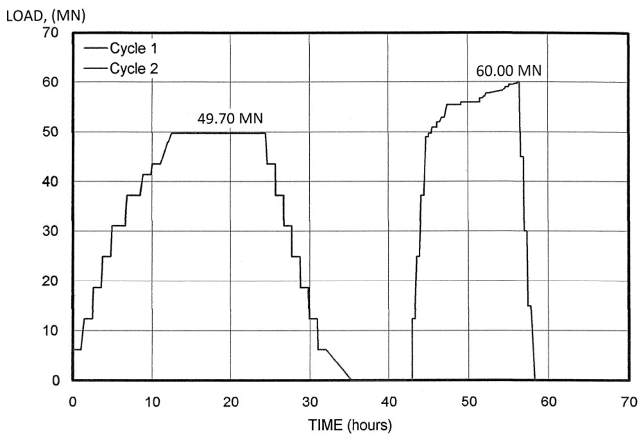

Figure 7 Loading sequence with time of tested barrette pile number TP1

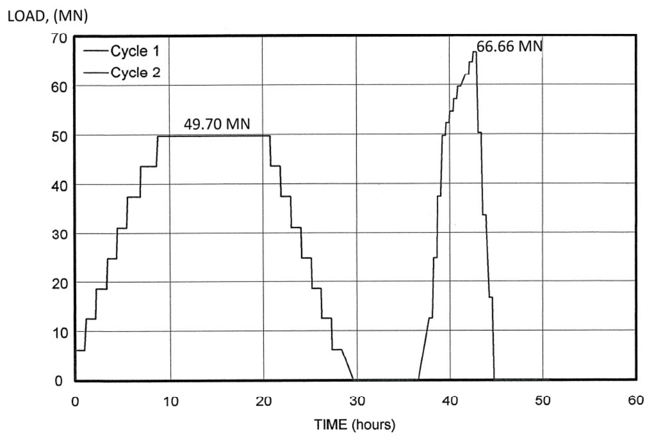

Figure 8 Loading sequence with time of tested barrette pile number TP2

5.3 Barrette pile test results

By following test procedure described previously, the test result of tested barrette pile number TP1 in terms of load and settlement is presented in Figure 9. The loads and settlements of cycle 1 and cycle 2 are shown in Table 5. TP1 at first was in elastic zone during starting to approximately 40.00 MN of loading. After that, the rate of settlement was larger and curve of load-settlement is much steeper indicating that TP1 was in elsato-plastic zone. Pile load test cycle 1 can still be carried out until load reached 49.70 MN (two times of design load). Pile behavior in cycle 2 was similar to the previous one until the loading reached 49.00 MN. After that, higher settlement rate was recorded until 60.00 MN of load was applied. Finally, total settlement of 302.01 mm. was monitored before rebounding step. Low value of recovery rebound was just about 17% in the first cycle and not even 10% in the second leaving very large value of permanent settlement.

The load–settlement curve and the test result of barrette pile number TP2 are presented in Figure 10 and Table 6 accordingly. The behavior of test pile under both cycle 1 and cycle 2 was in elastic zone from starting to finish. There was no sharp change in curve or in rate of displacement and rebound was high. Since yield strength has not been reached, very low settlement compared to TP1 of 14.82 mm. and 24.55 mm. was recorded. Additionally, recovery rebound was nearly 100% in the maintain load test and up to 86.8% in second cycle.

The maximum load test of TP1 was 60.0 MN lower than TP2. Nonetheless, the pile behavior is totally different. Much greater settlement was observed in TP1 compared to TP2 as yield point was reached at very first stage about 50.00 MN of loading in TP1. For TP1, 98.06 mm. and 302.01 mm. were induced in the first cycle and in quick load test, accordingly. This high value of settlement is very uncommon. In real situation, if barrette pile or any pile suffer from large value of settlement (more than 10% of pile size), it would be considered as failure. Dissimilarly, only 14.82 mm. and 24.55 mm. were detected in TP2.

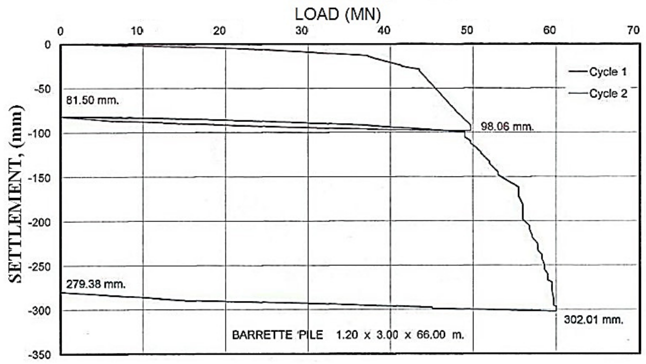

Figure 9 Load-settlement of tested barrette pile number TP1

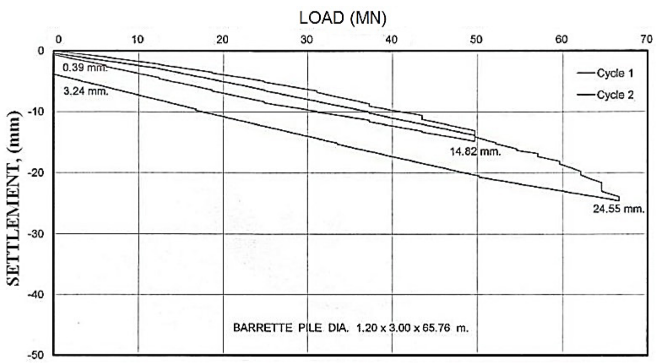

Figure 10 Load-settlement of tested barrette pile number TP2

Table 5 Test results of barrette pile number TP1

Description	**Explanation**	
	Cycle 1 Max. Test Load 49.70 MN	**Cycle 2 Max. Test Load 60.00 MN**
Gross Settlement	98.06	302.01
Permanent Settlement	81.50	279.38
Recovery Rebound	16.56	22.63

Table 6 Test results of barrette pile number TP2

Description	**Explanation**	
	Cycle 1 Max. Test Load 49.70 MN	**Cycle 2 Max. Test Load 66.66 MN**
Gross Settlement	14.82	24.55
Permanent Settlement	0.39	3.24
Recovery Rebound	14.43	21.31

6. INTERPRETATION OF BARRETTE PILE BEHAVIOUR

The barrette pile number TP1 was tested up to 60.00 MN. Raw data from strain gauges attached to barrette piles in each sequence of loading can be used for back analysis for load distribution along pile shaft as presented in the Figure 11 in case of TP1. It can be seen that end bearing has been put in action since the very first stage of load testing at about 40.00 MN. With load-settlement curve, it can be implied that large vertical displacement was induced during mobilization of end bearing capacity. This phenomena of end bearing mobilization is very unusual because high settlement will be induced and leads to failure of pile. From load distribution figure, mobilized skin friction curve in each strain gauge level can be plotted as illustrated in Figure 12. There are four curves in the figure. Each curve describes skin friction development behavior. It can be seen clearly that all lines reached yield strength at very initial stage. After that, large displacements were monitored in three sets of data (-2m. to -14.5m., -14.5 to -40m. and -50m. to 65m.) without any significant further friction mobilized or even decrease in some cases. There is only one line which shows an increase in skin friction after yielding but again with large displacement. Skin friction and end bearing are summarized in Table 7.

According to this shaft friction mobilization, adhesion factor (α) and friction factor (β) parameters suggested by Reese and O'Neill (1988) which used for barrette pile capacity estimation can be determined. Eq. (6) is employed to determine the value of friction factor in sand layer and effective overburden pressure can be estimated directly. The drawdown effective overburden stress is used as presented in Figure 2. Adhesion factor can be calculated using Eq. (5) for clay layer. The correlation equation from N-value of SPT test to shear strength (Su) for Bangkok subsoils follows Eq(7). These design parameters are then compared with typical value of Bangkok subsoil from Teparaksa (2001 and 2011) as shown in Figure 13 and Figure 14. Both values of TP1 agree with typical value of Bangkok subsoils. In addition, β values lay on non-grouted curve.

However, the barrette pile number TP2 showed much better performance. It was tested up to 66.66 MN. The load distribution is shown in Figure 15. Unlike TP1, there was no significant end bearing developed even at final load of 66.66 MN. From the graph, it can be implied that almost all of load was supported by skin friction. Mobilized skin Friction of TP2 in each level is illustrated in Figure 16. There are six lines of data in total. In the different way, only strain gauges in shallow layer show yielding (-2.76m. to -14.26 m. and -14.26m. to -23.76m.). Yielding is not reached in the other curves. Table 8 presents skin friction and end bearing evaluate from load distribution. It is obvious that much more end bearing has mobilized in TP1.

Relation between settlement and loading of TP1 and TP2 are displayed in Figure 17 and Figure 18. Load shown in those figures is the applied load at top of barrette pile. It can be obviously seen that much more toe settlement has been developed in TP1 indicating that end bearing has been mobilized starting at approximately 40MN of loading. The major settlement of TP1 was governed by end bearing. In TP2, very little toe settlement has been induced and end bearing mobilization was small as presented in Figure 15.

Using the same step as in TP1 calculation, Adhesion factor and friction factor of TP2 can be plotted as in Figure 13 and 14, respectively. The β values agree well with base-grouted curve.

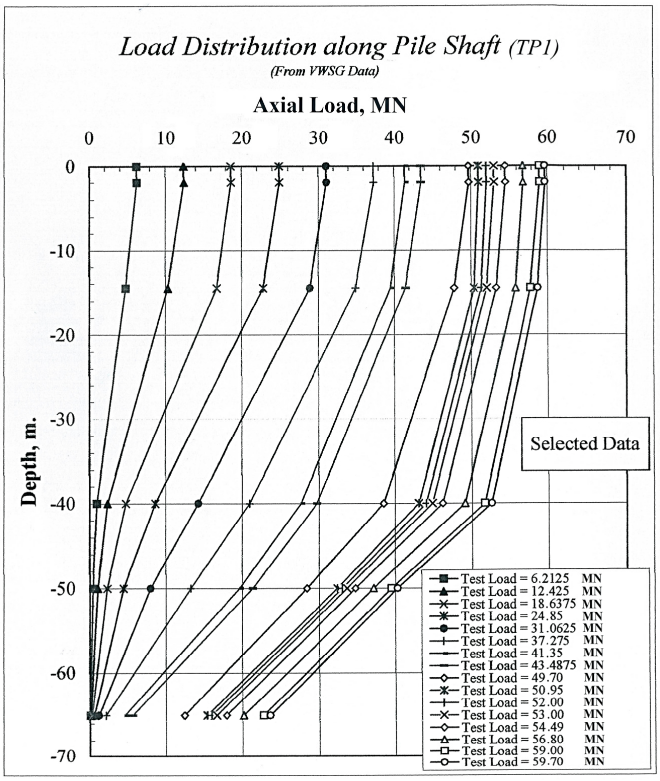

Figure 11 Load distribution of tested barrette pile number TP1

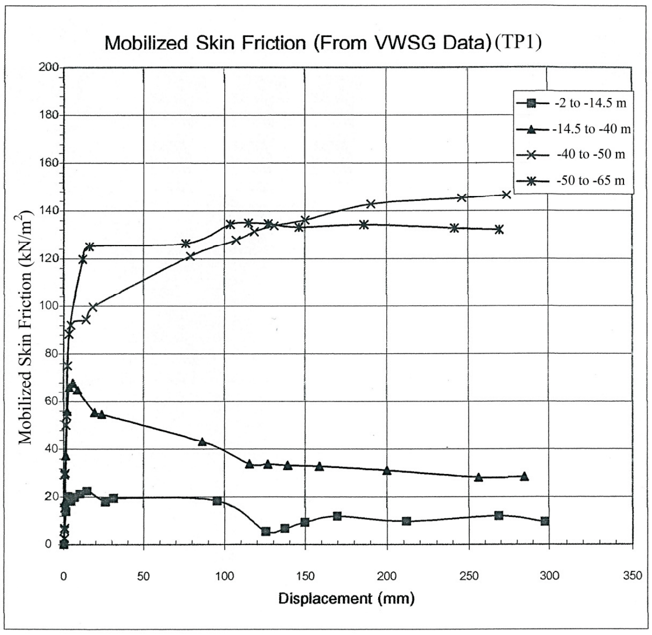

Figure 12 Mobilized skin friction of tested barrette pile number TP1

Table 7 Skin friction and end bearing of TP1

Load (MN)	Load Description	Skin Friction		End Bearing	
		Load (MN)	Unit Skin Friction (kN/m^2)	Load (MN)	Unit Skin Friction (kN/m^2)
24.8500	Design Load	24.2441	43.7	0.6059	168.3
31.0625	F.S. 1.25	29.9052	53.9	1.1573	321.5
37.2750	F.S. 1.50	35.1711	63.4	2.1039	584.4
41.3500	F.S. 1.66	36.7856	66.4	4.5644	1267.9
49.7000	F.S. 2.00	37.2604	67.2	12.4396	3455.4
56.8000	Settlement of 15% pile width	36.6261	66.1	20.1739	5603.9
59.7000	Maximum Test Load	36.0943	65.1	23.6057	6557.1

Table 8 Skin friction and end bearing of TP2

Load (MN)	Load Description	Skin Friction		End Bearing	
		Load (MN)	Unit Skin Friction (kN/m^2)	Load (MN)	Unit Skin Friction (kN/m^2)
24.85	Design Load	24.52	45.1	0.33	92.2
49.70	F.S. 2.0	48.76	89.6	0.94	261.3
62.13	F.S. 2.5	60.03	110.4	2.09	581.8
66.66	Maximum Test Load	63.53	116.8	3.13	868.4 (not fully mobilized)

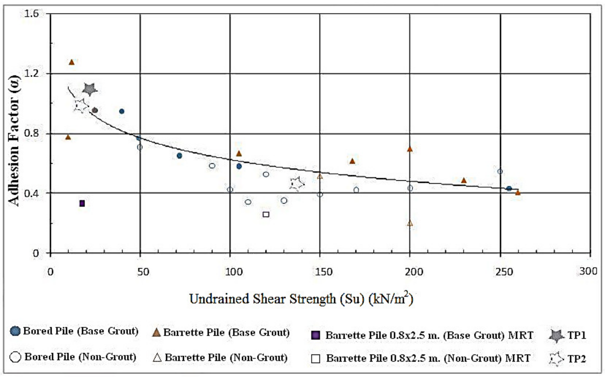

Figure 13 Adhesion factor (α) and undrained shear strength (S_u) for barrette pile using bentonite slurry (after Teparaksa, 2011)

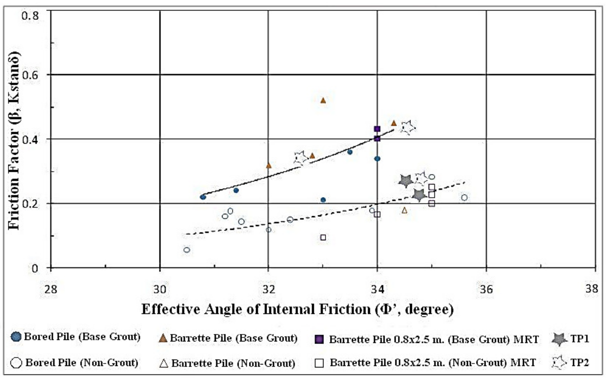

Figure 14 Friction factor (β) and effective angle of internal friction (Φ') for barrette pile using bentonite slurry with and without base grouting (after Teparaksa, 2011)

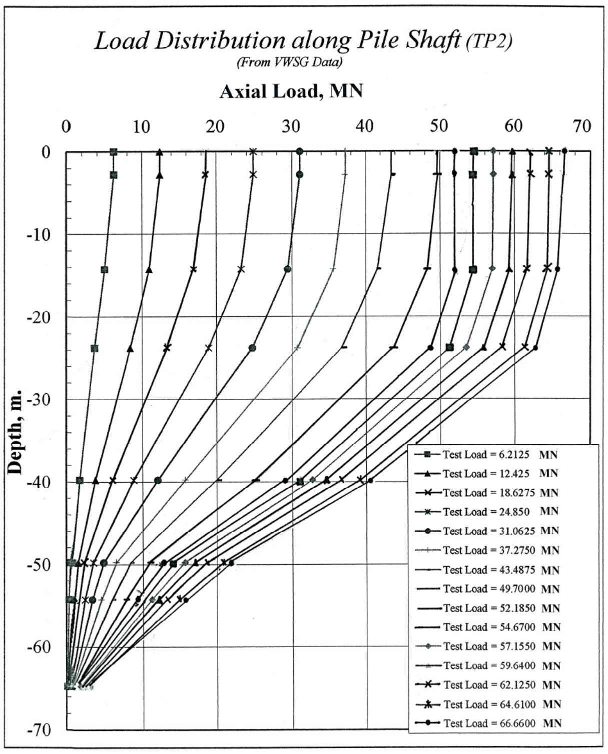

Figure 15 Load distribution of tested barrette pile number TP2

Normally in Bangkok, pile load test is to be conducted right after 28 days of pile casting due to cost of waiting. In some cases, there are just few days delay (no more than 40 days in total) but there would not be much different in test result. In this case study, if aging effect is taken into account, test result of TP1 can be considered usual as it has shorter curing time. However, there might be some parameters could be out of control; for instance, desanding process which governs cleanness of pile toe. This issue can also lead to end bearing mobilization and large toe settlement.

7. DISCUSSION OF TEST RESULT

7.1 Adhesion factor (α)

In Figure 13, there is one representative of each pile in the range of shear strength from 10 – 30 kN/m^2 as showed in a star symbol. The adhesion factor of TP1 stayed slightly above that of TP2. Comparing these results between TP1 and TP2, although the value of α parameter for barrette pile number TP1 is greater than that of barrette pile number TP2, the ultimate pile capacity of barrette pile number TP2 can be higher because the α parameter has less influence on capacity compared to parameter β. Another result of TP2 is lower than typical curve. Anyhow, both adhesion factor of TP1 and TP2 agree well with typical value of Bangkok subsoil. It can be concluded that curing time no significant different in this parameter.

7.2 Friction Factor

Figure 14 presents result of friction factor from back analysis. Fiction factors of barrette pile number TP1 lie on non-grouted barrette pile curve but TP2 result is scatter. It can be seen that most of Beta value of TP2 agree with grouted barrette pile line. Furthermore, there is also one point of data which located slightly above non-grouted line. Nevertheless, based on this set of data, it can be concluded that higher value of friction factor from TP1 or barrette which underwent more curing time agrees with typical value of grouted pile though the barrette have not been toe grout.

For the whole picture, the higher value of β can help promoting higher pile shaft friction resistance resulting in higher ultimate pile capacity. Hence, the barrette pile number TP2 can be tested at larger test load than the other one. This is might due to the longer curing time of the barrette pile with ageing effect (Jardine et al., 2005).

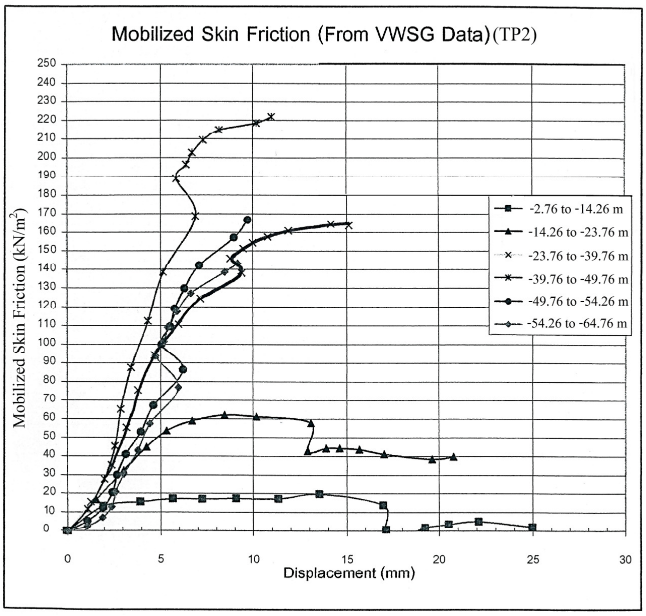

Figure 16 Mobilized skin friction of tested barrette pile number TP2

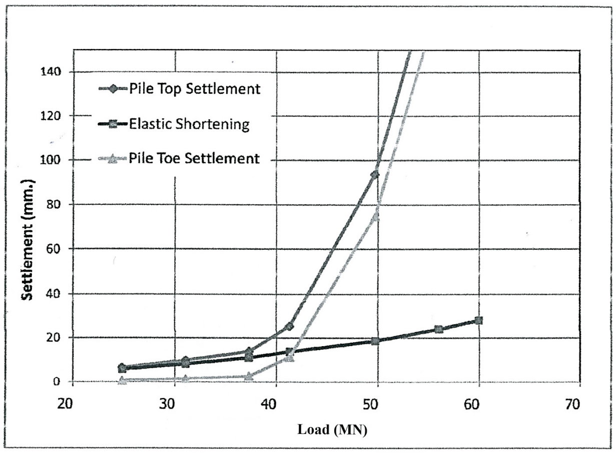

Figure 17 The relationship between settlement and loading of TP1

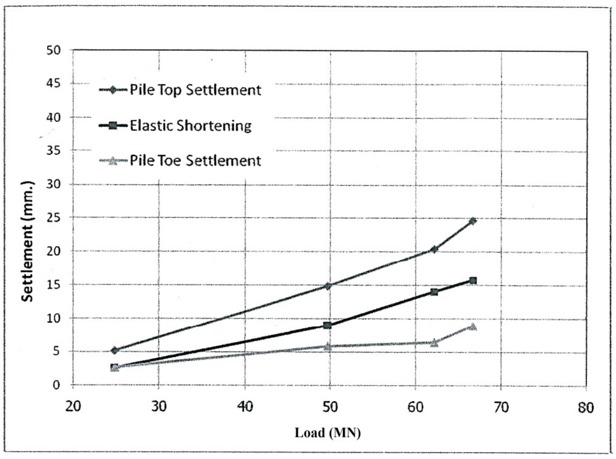

Figure 18 The relationship between settlement and loading of TP2

In the past work of Teparaksa (2008), it was found that using bentonite as a hole stabilizing fluid in circular bored pile results in forming approximately 1 cm. thick bentonite film or cake film at the soil and pile interface but just only in sand layer. Bentonite slurry itself is colloid and thus leads to sedimentation and promotes soft base problem as well as thick bentonite film in sand layer as presented in Figure 19. Physical model test was carried out in order to determine thickness of the interface layer. Figure 20 and Figure 21 show 1 cm. thick cake film from the research. The problem of cake film was also observed in many researches such as Fleming and Sliwinski (1977) and Brown (2002). Both issues cause loss in shaft skin friction and bearing capacity. Although soft base problem can be solved by base grouting, there is no way to dealing with cake film in case of bentonite slurry. However, polymer slurry can be used instead of bentonite in order to eliminate cake filter (Brown et al., 2002).

In similar way, using bentonite slurry in barrette also creates cake film. Polymer slurry here cannot be used to solve to problem as it would rise corner effect and thus lead to failure. In this case of pile testing in high rise building project, bentonite was also used. The result of the test is that gaining in skin friction only occurred for sand layer. By combining those past works in the topic of cake film that the cake filter is formed only in sand layer with this research, the conclusion that the strength of cake film is generated overtime can be implied. Additional supporting reason is that the α values of both TP1 and TP2 are not significantly different since there cake film does not appear in clay layer. Thus, gaining in strength over time can be noticed in only sand layer.

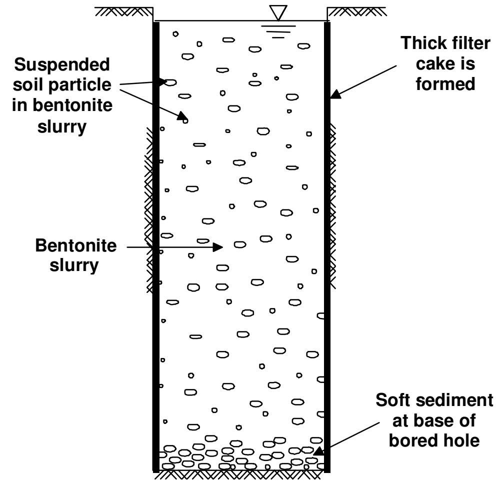

Figure 19 Cake film and soft base forming mechanism (Teparaksa, 2008)

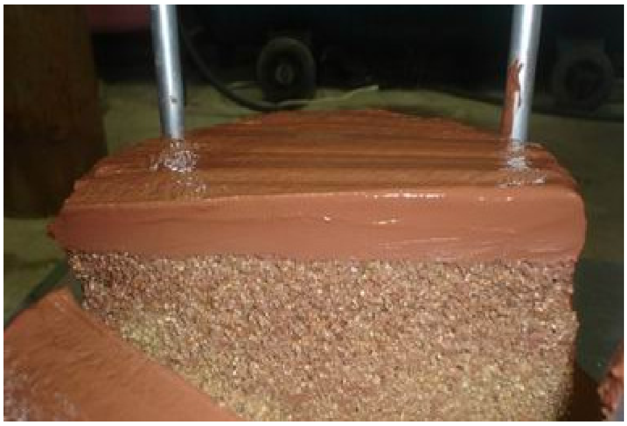

Figure 20 Photo of cake film from model test (Teparaksa, 2008)

Figure 21 Photo of cake film from model test (Teparaksa, 2008)

8. CONCLUSION

Two barrette pile tests were carried out at the highest high-rise building project in Bangkok subsoils. Barrette pile sizes were 1.20x3.0x66m. Full set of instrumentation was installed in the tested barrette piles. The maximum test load of each pile was different. The barrette pile no.TP2 showed higher maximum test load with less settlement compared to the second barrette pile. This might be due to the aging effect which promotes an increase in shear strength of surrounding soil around pile shaft since the barrette pile no.TP2 had longer curing time than TP1. Monitoring data from attached strain gauges gives load-displacement, load distribution and mobilized skin friction graph. This allows the design parameters; α and β, to be evaluated. These factors are then compared with typical value from past researches in Bangkok. Great agreement is clearly visible for adhesion factor. There is no significant difference in adhesion factor of TP1 and TP2. In addition, friction factor of TP1 and TP2 are corresponding to typical value of non-grouted barrette pile and base grouted barrette pile respectively. It may be implied from past works that this aging effect might influence friction factor and might affect strength of cake bentonite film.

9. ACKNOWLEDGEMENT

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