PERFORMANCE OF LARGE-DIAMETER BORED PILE WITH SHORT SOCKETED LENGTH IN IGNEOUS ROCK IN THAILAND

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ABSTRACT

Large-diameter bored piles have been utilized extensively in soft soil for more than four decades in Thailand, whereas there are only limited numbers of rock-socketed bored pile constructed. To date, load transfer and settlement behavior of piling in rock in Thailand is still not fully understood, especially when the socketed-length is less than one time the pile diameter. The objectives of this paper are to analyze load distribution and to investigate load-settlement relationship of bored pile in rock. Collected data in this study was from pile load test results of more than 90 large-diameter bored piles, socketed in igneous rock such as granite and andesite. Bored piles were constructed using rotary drilling method, having socketed length varying from 0.3 to more than 3 times the pile diameter. Factors that influenced pile settlement behavior, consisting of socketed length and construction method were analyzed. The effects of socketed length on load-settlement relationship and ultimate load bearing capacity were investigated. Load-settlement behavior, unit shaft resistance and unit end bearing is presented and discussed. Also, guidelines of estimation of pile settlement in rock is proposed. According to measured and calculated data, settlement of pile was mainly due to elastic shortening. This is because shaft friction in granite was the major component for pile resistance. For piles with socketed length per diameter ratio (S/D) of greater equal one, the mobilized shaft friction and end bearing are significantly larger than those with S/D of less than one. This is because socketed length is sufficient in rock to provide fully mobilized end bearing, resulting in relatively large stiffness at pile base.

KEYWORDS:

Load transfer mechanism, Socketed length, Design parameters, Igneous rock

INTRODUCTION

In Thailand, major deep foundation projects are located in Bangkok area where soft ground such as soft marine clay and alluvial soils are encountered (Boonyarak et al., 2016). Due to demand in urbanization of the country, deep foundation in rock gradually increase as the construction projects extent to other cities where rock depth is relatively shallow. Minimum socketed length in rock adopted from some codes of practice, for example Building Department of Hong Kong SAR, specifies 0.5 m in fresh or slightly weathered igneous rock (BD, 2009). However, due to lack of experience and suitable rock drilling tools, the ability to excavate in rock of many piling contractors in Thailand is about 0.3 m only in moderately weathered granite. Thus, behavior of bored pile in igneous rock, especially when socketed-length per diameter ratio (S/D) is less than one, is not fully understood. Thus, further research is still required to minimize variation and to obtain guideline in design of piling in igneous rock in Thailand.

This paper presents interpretation of load testing results of more than 90 piles, socketed in granite and andesite. Analysis results of load-settlement, pile settlement estimation, factor for calculating shaft friction and end bearing are reported. A case study of pile behavior where the S/D is less than one with possibilities of large and differential settlement is described. Discussion and explanation of pile behavior in igneous rock are provided.

BORED PILE CONSTRUCTION IN ROCK

Geology

Figure 1 shows Geological map of Thailand (Department of Mineral Resources, 1999). Major urban development has taken place in Greater Bangkok area, where soil conditions consist of soft marine deposit at the top 10-20 m follow by alternating layers of sand and clay. Igneous rocks are mainly located in the North, East and South. The most common rock encountered during bored pile construction in these regions is granite. Typical unconfined compressive strength of granite from several projects the authors have collected is ranging from 40-150 MPa.

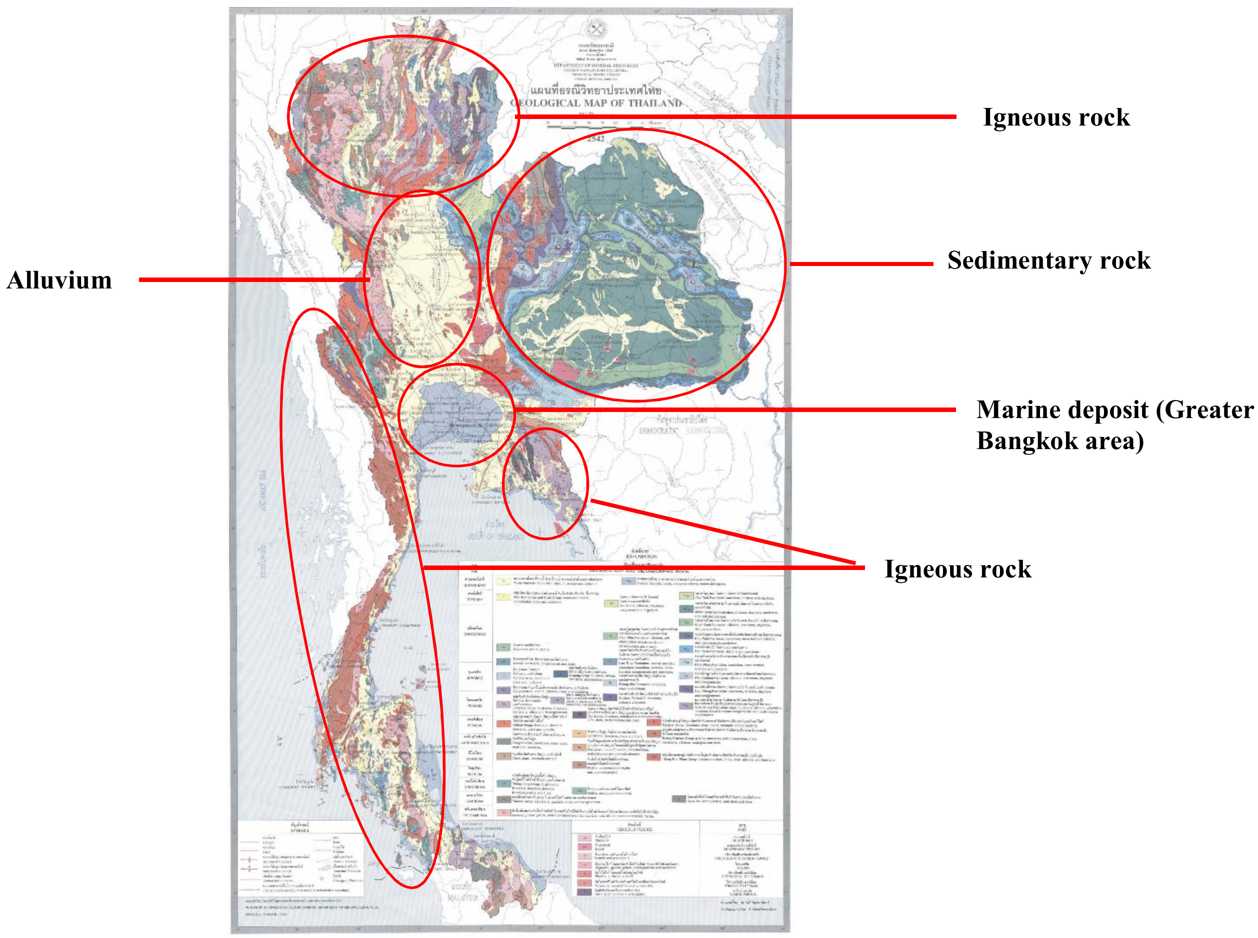
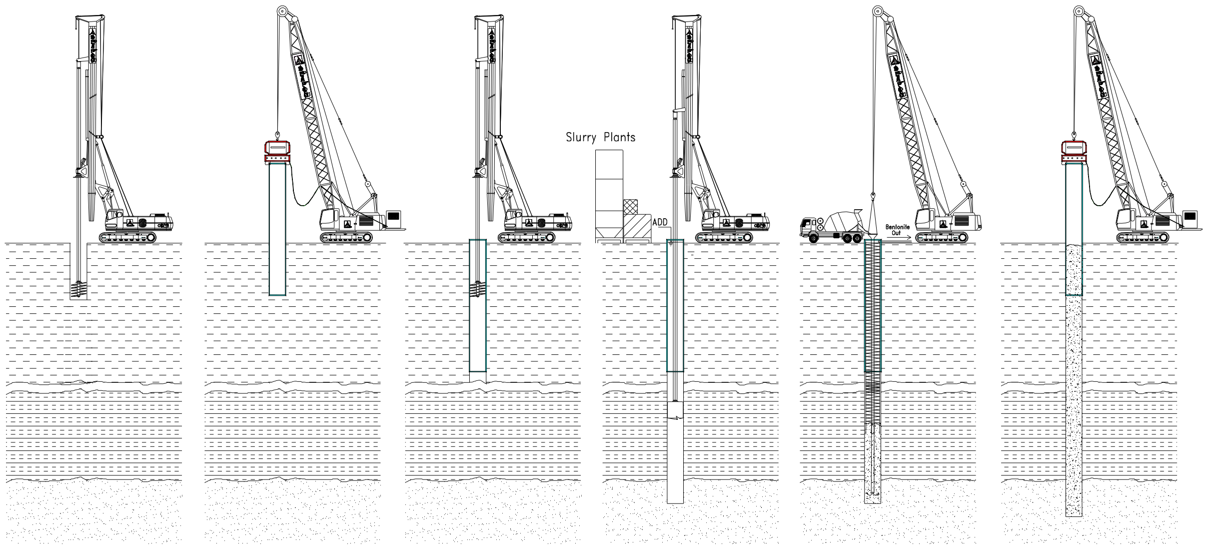


Fig. 1 Geological map of Thailand (Department of Mineral Resources, 1999)

Bored pile construction sequences and quality control

Typical construction sequences of bored pile in rock are shown in Figure 2. The sequences are as follows: (a) First, pre-boring of hard soil at the surface is carried out. (b) Temporary steel casing is driven using vibro hammer to protect borehole collapse in soft soil layers (i.e., loose sand and soft clay). (c) Excavation is carried out using flight auger inside the casing or in hard clay layer. (d) Prior to encounter unstable soil layer beneath the depth of the casing, bentonite slurry is poured into the borehole for stabilization. Drilling bucket is used for excavation under the slurry until reaching rock layer. Slurry circulation using pump and desander is used for cleaning of slurry in the borehole. (e) Reinforcement cages are lowered into the borehole one after another. Afterwards, concrete is placed through a tremie pipe to replace the drilling fluid until reach the pile cut-off level. (f) Finally, the temporary casing is extracted before the concrete sets.



(a) (b) (c) (d) (e) (f)

Fig. 2 Typical construction sequences for bored pile

Drilling in rock

To drill in completely and highly decomposed granite, bullet teeth installed on an auger is adopted (as shown in Fig. 3a). The samples taken using rock auger is shown in Figure 4a. However, in slightly weathered to fresh rock, auger with bullet teeth cannot be efficiently drilled into these layers. Maximum socketed length in hard layer using rock auger is from 0.3 to 0.5 m for moderately weathered granite. To penetrate further into slightly weathered granite, core barrel with bullet teeth is used as shown in Fig. 3b. In order to penetrate into the slightly weathered or fresh granite to depth more than 1 m, roller bit installed on core barrel as shown in Fig. 3c is adopted. By using roller bit installed core barrel, rock sample can be collected as a piece as shown in Fig. 4b.

(a) (b) (c)

Fig. 3 Tools for excavation in rock (a) rock auger with bullet teeth; (b) core barrel with bullet teeth; (c) core barrel with roller bits

(a)

(b)

Fig. 4 Rock material excavated by (a) rock auger with bullet teeth; (b) core barrel with roller bits

Quality control in construction

Deviation and inclination of the borehole are controlled to be within 75 mm at ground level and 1:100, respectively. The methods for controlling deviation and inclination of borehole are conventional survey technique, visual inspection of verticality of kelly bar and drilling monitor apparatus (if specified in the project). To ensure the quality of the drilling fluid, routine testing is carried out prior to excavation and concreting (FHWA, 2010). Prior to reinforcement cage lowering, rock samples at the pile base are identified and sediment measurement using sounding method by measuring tape is carried out. Concrete slump and setting time for wet-process bored pile are 150-200 mm and 4-6 hours, respectively. For Tremie concrete, apart from flow ability, stability to prevent bleeding and channeling is also required (EFFC/DFI, 2018).

LOAD TESTING AND DATA ANALYSIS

To validate load bearing capacity of the constructed pile, pile load test is carried out. For most of infrastructure projects, static pile load test is performed. Apart from static load test, high strain dynamic load test is carried out to verify the integrity and capacity of piles to save cost and time of testing.

Static pile load test

Axial compression static load tests according to ASTM D1143 standard are carried out on working piles of the project. If friction is adequate, four anchor piles for each test pile are used as a reaction system. In a case that friction is not adequate for anchor piles, dead load on kentledge is adopted as the reaction system. Maintained loading procedures up to 2.0 or 2.5 times the design working load are adopted. Table 1 summarizes details of test piles using static load test method to determine load-settlement curve and verify the capacity of each pile in this paper.

Dynamic pile load test

Apart from pile testing with static load method, which is time consuming and required costly preparation, high strain dynamic load test (ASTM 4945) can be adopted as supplementary tests. In many projects, static load tests are carried out on 2-3 pilot piles and dynamic load tests are carried out on working pile to randomly investigate the integrity and load bearing capacity of piles in each project. In this paper, results from 80 test piles using high strain dynamic load test were analyzed (details are not shown in table for brevity of paper).

Table 1. Details of test piles using static load test method

Province, Region	Pile ID	Pile diameter (m)	Socket length (m)	S/D*	Pile tip (m)	Max. test load (kN)	Rock type
Chonburi, East	SLT1	1.5	2.8	1.9	20.3	20,250	Granite
Chonburi, East	SLT2	0.8	3.0	3.8	18.0	4,400	Granite
Chonburi, East	SLT3	0.8	2.0	2.5	15.5	4,400	Granite
Chonburi, East	SLT4	0.8	1.0	1.3	14.0	4,400	Granite
Chonburi, East	P46	1.0	3.0	3.0	24.0	13,750	Andesite
Rayong, East	T1	0.8	1.7	2.1	13.7	4,400	Granite
Rayong, East	T2	1.5	3.0	2.0	15.7	18,000	Granite
Rayong, East	T3	1.8	3.0	1.7	14.4	26,000	Granite
Saraburi, Northeast	T120	1.2	18.0	15.0	26.0	16,500	Andesite
Saraburi, Northeast	T150	1.5	18.0	12.0	26.0	18,000	Andesite
Saraburi, Northeast	T180	1.8	18.0	10.0	26.0	26,000	Andesite
Songkla, South	TP	1.5	0.5	0.3	17.4	20,000	Granite
Phuket, South	BP43	0.6	0.3	0.5	19.7	2,700	Granite
Phuket, South	DP29	0.6	0.3	0.5	8.6	2,700	Granite
Phuket, South	KP18	0.6	0.3	0.5	11.4	2,700	Granite
Phuket, South	IP14	0.6	0.3	0.5	9.6	2,700	Granite

Note: * S/D is socketed length per diameter ratio

INTERPRETATION OF RESULTS

Load-settlement relationship

Figure 5 shows normalized load-settlement relationships of bored pile socketed in rock. To compare results from piles with different depths, diameters and applied loads, the applied load and settlement are normalized with section area and pile diameter, respectively. Load-settlement data was divided into two groups, socketed-length per diameter (S/D) ratio of less than one and greater equal one. In most tests, there are two cycles of loading, first cycle testing is up to the safe working load (SWL) following by unloading and second cycle loading to two times the SWL and rebound. In some tests, maximum test load can be up to 2.5 or 3 times the SWL.

For the piles with S/D greater equal one, normalized pile settlement at SWL is less than 0.5%, which is significantly less than fully mobilized friction condition proposed by Boonyarak et al. (2016). At two or more times the SWL, normalized pile settlement of the group of S/D greater equal one is relatively smaller than 2%, suggesting that friction is not fully mobilized and so is the end bearing. This is because of sufficient socketing in rock that provided high stiffness at the pile base and contributed to large capacity for bored pile.

On the contrary, for piles with S/D less than one, large variation of normalized settlements at SWL from 0.3% to 1.9% were observed. It suggests that, differential settlement may occur if these piles are supporting the same structure. At two times the SWL, the variation in settlement became larger, the maximum settlement was close to 5% which is the ultimate capacity suggested by Boonyarak et al. (2016). For short-socketed pile, when load transfer to toe, large pile settlement occurs. This can be also affected by the workmanship problem of inappropriate pile base cleaning. The piles with large settlement are in only one project in Phuket (summarized in Table 1). These piles required modification of foundation design and rectification measure to minimize differential settlement in each building. This measure is discussed in the Section "Rectification measure to minimize differential settlement".

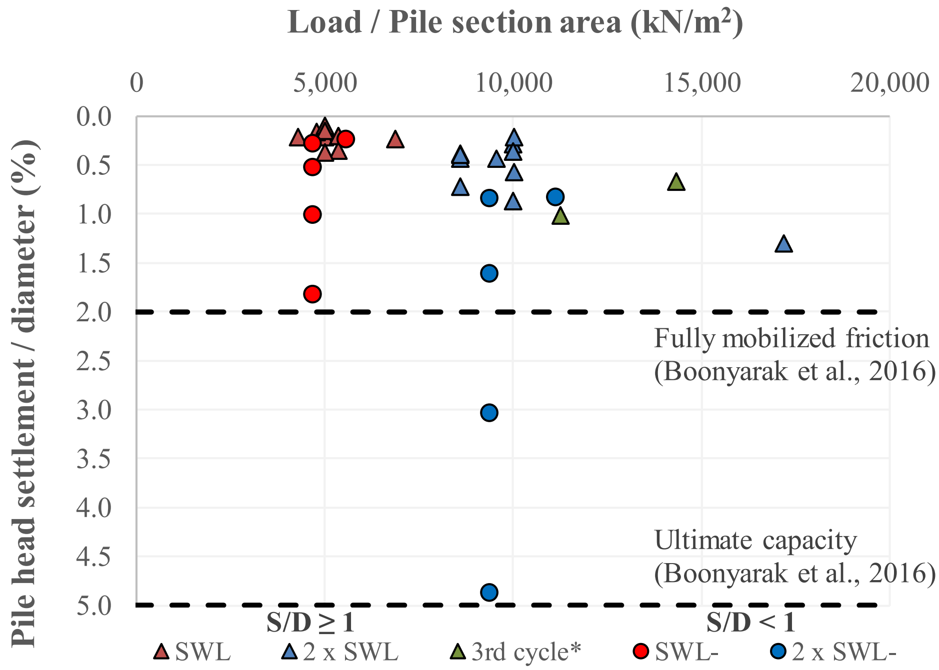


Fig. 5 Load-settlement curves of test piles

Estimation of pile settlement

Figure 6 shows comparison between measured and computed pile settlement at the maximum load of each test for pile with S/D greater equal one. Pile settlement can be estimated using Equation 1 proposed by Tomlinson (1995) as shown below.

$$\delta = \frac{(W_s+2W_b)L}{2A_sE_p} + \frac{\pi W_b B(1-\nu^2)I_p}{4A_bE_s} \qquad [1]$$

The first and second terms in this equation refer to elastic shortening of the pile and settlement of soil at pile base, respectively. Symbols in Equation 1; δ, W_s, W_b, L, B, A_s, A_b, ν, I_p and E_s denote settlement at pile head, load carry by shaft, load carry by base, pile length, pile width (diameter), shaft area, base area, Poisson's ratio, friction distribution factor and Young's Modulus of soil at pile base, respectively. The modulus of elasticity of rock adopted is 50 times unconfined compressive strength (UCS). The modulus of elasticity of reinforced concrete of each pile (E_p) is assumed to be 34 GPa.

From the comparison, calculated settlement reasonably agrees with the measured results. However, for some piles embedded in granite, substantial difference between measured and calculated results is observed at measured settlement of 10-12 mm. A possible reason for this discrepancy is variation in degree of weathering that was not considered in the estimation causing inaccuracy of estimation of elastic modulus of rock at pile base.

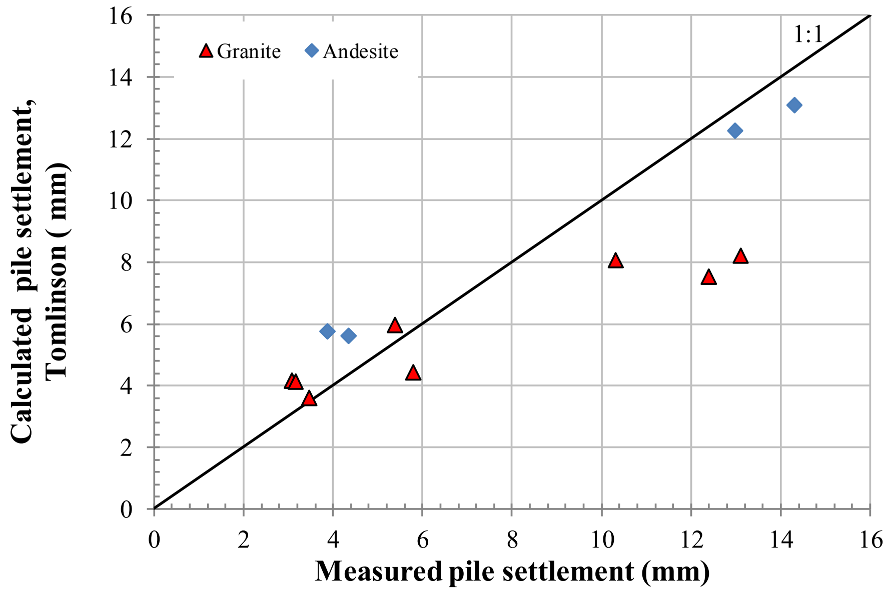


Fig. 6 Comparison between measured and calculated pile head settlement

BACK ANALYSIS OF PILE RESISTANCE

Results from high strain dynamic load test are able to estimate total shaft friction and end bearing of each pile. This estimation was obtained from input and output signal matching. Note that the maximum load activated on each test pile may not be the maximum load bearing capacity as the test load was designed to be slightly larger than the specified test load. Thus, the maximum resistance of pile is expected to be larger than those shown in this paper.

Mobilized shaft friction

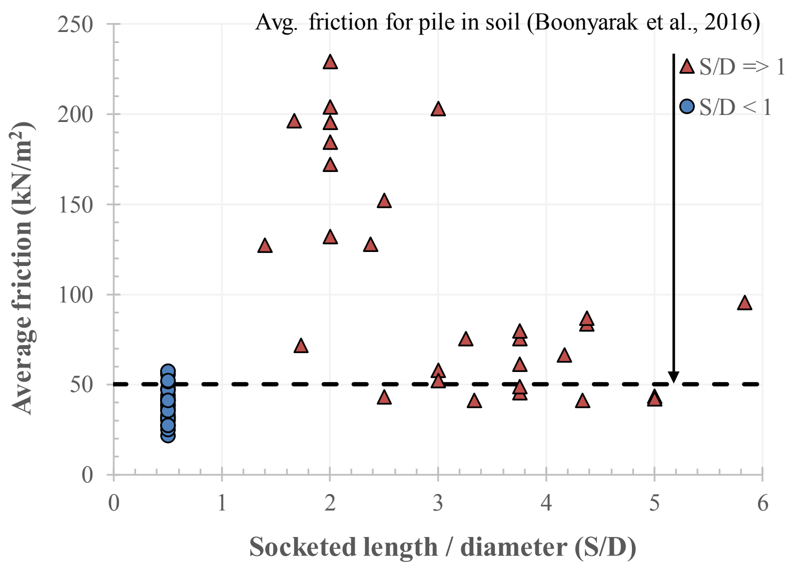
Figure 7a shows mobilized average shaft friction of pile with different S/Ds. The estimation of shaft friction reported in this paper is obtained for the entire pile length in soil and rock. For the piles with S/D less than one, average mobilized shaft friction was ranging from 20 to 60 kN/m^2. This is because of the end bearing in rock cannot be fully developed and pile settlement was relatively large (refer to Fig. 5). The large pile settlement caused smaller pile stiffness as stiffness of soil reduced with increasing shear strain.

For mobilized friction for piles with S/D ranging from 1 to 3, average friction was the largest (120 – 230 kN/m^2). This is because the pile is socketed in slightly weathered granite and the pile base stiffness was the largest. It results in largest friction developed along the pile shaft. The pile with S/D more than 3 has the mobilized friction between 40 – 100 kN/m^2. These range of friction was larger than those in soil reported by Boonyarak et al. (2016). It should be noted that, the average shaft friction does not increase proportionally with S/D. This maybe because for weaker rock or higher degree of weathering, the socketed length is required to be larger in order to support the design load.

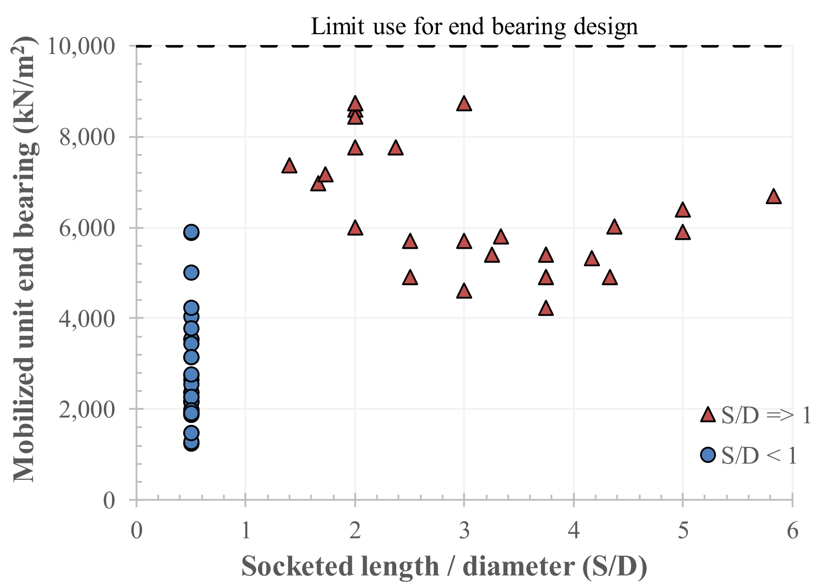
Mobilized unit end bearing in rock

In Fig. 7b, mobilized unit end bearing with different S/Ds is illustrated. Large variation and low end bearing was observed for pile with S/D of 0.5. This observation may be caused by poor rock material at pile base or poor base cleaning of pile. In addition, the socketed length was not sufficient to develop firm base behavior in rock resulting in large pile settlement (refer to Fig. 5). The mobilized unit end bearing for pile with S/D of 0.5 is ranging from 1,500 to 6,000 kN/m^2. This end bearing is only slightly larger than those pile constructed in soil where unit end bearing can be developed from 2,000 to 3,000 kN/m^2.

For piles with S/D more than one, the unit end bearings were between 4,500 and 9,000 kN/m^2. There is no clear trend between S/D and unit end bearing. It suggests that minimum S/D of one may be adequate to develop sufficient end bearing. Note that the mobilized unit end bearing does not reach 10,000 kN/m^2 which is the limit of commonly design value. This is because load activated by high strain dynamic load test was not large enough to fully mobilize the end bearing in rock.



(a)



(b)

Fig. 7 Pile resistance provided by (a) friction; (b) end bearing

RECTIFYING MEASURE FOR MINIMIZING DIFFERENTIAL SETTLEMENT

As stated earlier that pile settlement in one project with S/D of 0.5 has large variation in pile settlement (refer to Fig. 5), rectifying measures for reducing differential settlement was carried out. There are total 10 five-story buildings in this project and each building is supported by about 50 to 60 piles. The pile diameter is 600 mm and it was specified that only 300 mm is socketed in slightly weathered granite. However, according to test results, settlement in some piles were larger than 30 mm (5% of pile diameter) at two times the safe working load. To minimize the possibilities of differential displacement, spring stiffness of each pile group was evaluated based on load testing results. Unlike other project that spring stiffness for each pile diameter is a constant, the spring stiffness of pile group in this project vary area by area even the pile diameter is the same.

Pile re-striking with drop hammer to reduce settlement

After determination of pile spring stiffness was completed, piles with significantly low spring stiffness were selected. The head of these piles were prepared to be flat and smooth. Figure 8 shows the eight ton drop hammer for re-striking of bored pile with relatively low spring stiffness. Drop height, pile driving formula and factor of safety adopted were 0.6 m, Danish formula and 3, respectively. Settlements per blow in each pile were recorded and checked against the driving criteria given to achieved the working load of 1,350 kN per pile.

Monitoring of building settlement during construction

During construction of each building, four settlement markers were installed on the column of all four corners of the building. The record of building settlement was taken every month. According to the record, the maximum building settlement is ranging from 2 to 4 mm and differential settlement was within 1:1000, which is within the acceptance criteria.

Fig. 8 Drop hammer for re-striking of bored pile

SUMMARY AND CONCLUSIONS

Interpreted results from static and dynamic pile load tests are reported in this paper. Bored piles were constructed using rotary drilling and socketed into granite and andesite in Thailand. Load-settlement behavior and pile resistance were analyzed and discussed. A case study where socketed length per pile diameter ratio (S/D) of less than one was described and explained. According to the interpretation of data, following conclusions may be drawn:

- Settlement of pile with S/D greater equal one is less than 0.5% the pile diameter (D) at safe working load and less than 2%D at test load up to three times the safe working load. It suggests that, main component of pile settlement is contributed by shaft shortening.
- Calculation of pile settlement using analytical equation such as Tomlinson's provide a reasonable estimation. Young's modulus of rock at pile base is estimated to be 50 times the unconfined compressive strength of the rock sample.
- For piles with S/D of about 0.5, mobilized skin friction and end bearing behavior was only slightly larger than the pile constructed in soil. This is because pile base stiffness was not sufficient, resulting in relatively low pile resistance. For pile with S/D greater equal one, mobilized skin friction and end bearing are relatively high as the pile base is firm. There is still no clear sign of increasing in end bearing with increasing S/D. This is because S/D of greater equal one is sufficient to fully mobilize the end bearing in rock.
- To minimize the possibilities of differential settlement, some measures can be effectively adopted. These measures consist of determine spring stiffness of pile in each zone or re-striking of pile (if required) to minimize settlement at pile base.

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