Settlement Calculations of Large-diameter Bored Piles Socketed in Weak Rocks in the Area of a Double Track Railway Project (Chira Junction to Khon Kaen Station)

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Abstract

At the present, many public utilities transportation of Thailand have been developed in accordance with the government's policy that manages to connect urban and rural areas, for example, SRT Double Track Railway – Nakhon Ratchasima (Chira Junction) to Khon Kaen, Bang Pa-In – Nakhon Ratchasima Intercity Motorway Project (M6), etc. The runway structures of those projects are usually placed on large-diameter bored piles which run along the outskirts. Those areas have rarely been under engineering construction. Therefore, the engineering data is very limited. In this article, the calculations of settlement values of large-diameter bored piles socketed in weak rocks in the area of SRT Double Track Railway – Nakhon Ratchasima (Chira Junction) to Khon Kaen) are explained by traditional methods, a computer program method and comparisons with the data obtained from static load tests. The comparative results show that experimental pile shortening values are approximately 0.55-1.35 times of those of methods.

Keywords: Weak rocks, Pile settlement, Large-diameter bored pile, Static load test

1. Introduction

From the strategic plan for the development of Thailand's transportation infrastructure 2015-2022, infrastructure development is an important part of setting the foundation of economic development to enhance competitiveness and to attract investment. It also helps to improve the quality of life of the people and helps people to travel and trade at a lower cost. Therefore, the government has a policy and urges to push forward the construction of this project.

Currently, this project has been completed and has been launched since 2019. The foundations of superstructures in the project such as an elevated railway, U-turn bridges and overpass bridges are standing on large-diameter bored piles which are placed on the slightly–highly weathered sandstone and siltstone in order to procure adequate bearing capacity with the lowest pile settlement value. This area has never been constructed by using large bored piles before and engineering information is limited. It is commonly that practicing engineers use design parameters developed for ordinary soils instead of rocks to design the pile foundation, resulting in a relatively conservative design. Static load tests were therefore performed on 16 test piles to check their bearing capacities. Strain gauges and extensometers were installed on rebar cages of all test piles to find out measured axial force acting along piles and pile shortening. The results indicated that all piles did not reach the failure criterion. Consequently, design pile parameters under fully mobilized loading conditions using extrapolation methods were adopted and proposed by Mahakhotchasenichai et al. (2018). So, this research aims to study and predict pile settlement under extrapolation failure loads. This could to help the pile design of various projects in these areas to be closer to the reasonable value, resulting in a significant reduction in construction costs and time.

2. General information of the project

2.1 Location

The project is in Nakhon Ratchasima and Khon Kaen Provinces. The project starts at Thanon Chira Junction Railway Station, heading north parallel to Highway No. 2. The end of the project is at Khon Kaen Railway Station. The new track has a total length of approximately 187 kilometers, running parallel to the original track.

2.2 Soil and rock properties

From the geological maps shown in Fig. 1 to Fig. 2, it can be clearly seen that the majority parts of the project are placed on sedimentary rocks in the Maha Sarakham Formation which is non-marine red beds (light green color) and alluvial deposits (light yellow color). The most common rocks encountered are reddish-brown sandstone and siltstone. Rock salt is occasionally encountered in some areas. The soils encountered were generated from alluvial deposition and decomposition of the existing rocks, comprosing clay, silt and silty sand. In this project, there was an investigation by collecting soil and rock samples from 158 boreholes throughout the existing railway track. Details of soil and rock properties are discussed briefly as follows:

(1) At the beginning point of the project (NBH-001 at km. 268) to NBH-025 at km.290, most soils encountered comprise clay or sand. The deepest investigated borehole is approximately 60 meters. No rock layers were found in these areas.

(2) From NBH-025 at km. 290 to the end of the project (NBH-151 at km. 451), the top soil comprises clay and silt, alternating with sand, underlain by either sandstone or siltstone layers. The thickness of the soil layer is thinner compared to those encountered in the first section of the project. A rock layer was encountered at depths ranging from 3 to 50 m.

The rocks samples were tested to find out Uniaxial Compressive Strength of Intact Rock (UCS or σ_i) in accordance with ASTM Standard D-3148. The UCS values that acquired from the tests were inconsistent. In summary, the average UCS values are equal to 21.95 MPa for sandstone, 23.71 MPa for siltstone and 20.47 MPa for rock salt. Nevertheless, the strength parameters of the rocks acquired from the project investigation are not adequate to predict the pile settlement. Mahakhotchasenichai et al. (2018) performed an additional test to provide modulus of elastic (E_{50}) and Poisson's ratio (v) of rocks by mounted strain gauges on rock samples during uniaxial compressive strength tests. The results are demonstrated shown in the Table 1 to Table 2.

UCS values from the additional tests are less than those from the project investigation significantly. The average UCS values are equal to 11.16 MPa for sandstone, 15.37 MPa for siltstone and 12.77 MPa for both. There are various definitions used to describe weak rock: Robin (1992) defined rock strength class of mudrocks that are in the weak rock class when their UCS are less than or equal to 25 MPa, Hoek E. (2006) stated that some typical weak rocks are shale, mudstone, siltstone, phyllite and tuff and Kulhawy (1991) classified weak rocks by using UCS which is ranging from 0.5 to 20 MPa. Therefore, the rocks in this project are likely to be weak rock types compared to those definitions. Moreover, the rocks in this project are highly susceptible, more compressible and highly fractured with RQD ranging from 0 to 100. During construction, when the rocks came in contact with bentonite slurry, they were slaking and easy to disintegrate.

The rock data obtained from UCS test mounted with strain gauges will be adopted for further settlement calculation in the next section.

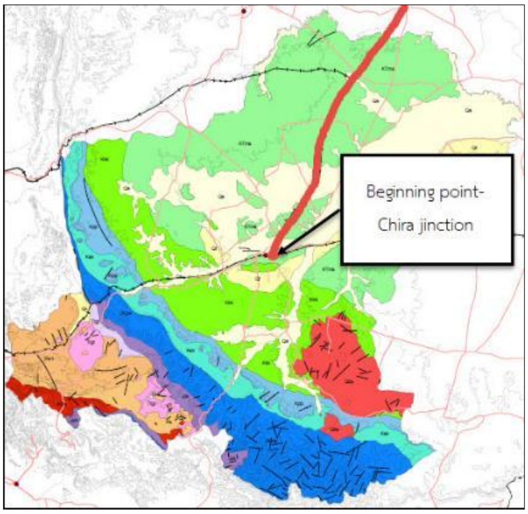

Fig.1 Geological map of Nakhon Ratchasima (Geological map of Thailand, 2007)

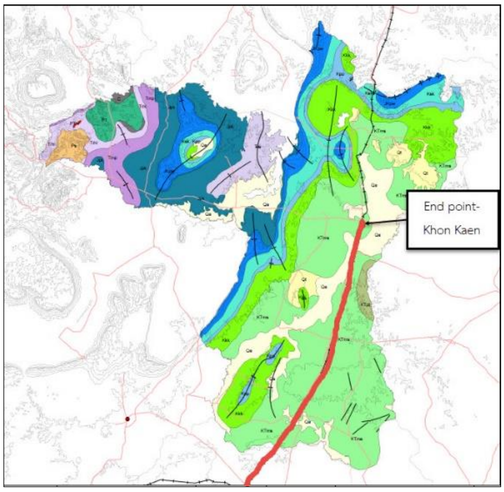

Fig.2 Geological map of Khon kaen Province (Geological map of Thailand, 2007)

Table 1 Results of UCS test mounted with strain gauges (Modified from Mahakhotchasenichai et al. 2018)

NBH	Type of Rock	Depth of sample (m)	σi (MPa)		Elastic modulus, E50 (GPa)		Poisson's ratio, v	
			Sample No.		Sample No.		Sample No.	
			1	2	1	2	1	2
27	Si(1)	17.5	20.79		4.98		0.20	
44	Sa(2)	21.0	7.57		1.42		0.33	
44	Sa	26.0	8.28	8.05	1.48	2.33	0.20	0.13
52	Si	19.0	26.71		7.67		0.19	
78	Sa	13.5	5.02		1.21		0.50	
78	Sa	18.0	5.30		1.59		0.50	
99	Si	14.0	7.37	7.58	1.32	1.30	0.16	0.19
99	Si	16.5	10.02		7.78		N/A	
130	Sa	18.0	23.85	21.47	3.71	3.73	0.31	0.42
132	Sa	20.5	16.83	11.92	8.34	5.31	0.18	0.19
134	Sa	14.0	9.72	7.23	1.29	1.53	0.49	0.39
134	Sa	19.0	9.92	9.92	3.79	1.00	0.38	0.29
148	Si	26.0	10.19		1.42		0.12	
148	Si	30.5	15.43		2.10		0.15	
148	Si	38.5	N/A		N/A		N/A	
148	Si	43.5	24.90		2.67		0.24	

(1)Siltstone, (2)Sandstone

Table 2 Summary of the results of UCS test mounted with strain gauges (Modified from Mahakhotchasenichai et al. 2018)

Type of rock	σi (MPa)			E50 (GPa)			Poisson's ratio, v		
	Min	Max	Av.	Min	Max	Av.	Min	Max	Av.
Si	7.37	26.71	15.37	1.30	7.78	3.66	0.12	0.24	0.18
Sa	5.02	23.85	11.16	1.00	8.34	2.83	0.13	0.50	0.33
All	5.02	26.71	12.77	1.00	8.34	3.14	0.12	0.50	0.28

3. Methodology

3.1 Static load test

For deep foundation design in Thailand, the best way to check actual pile bearing capacities of bored piles is to test piles using the static load test method. For this project, there is a plan to find out pile bearing capacities by performing static load tests at every location of U-turn and overpass bridges. There are also installations of strain gauges to each test piles to find out actual skin friction of soil and rock layers and extensometer tubes to find out pile shortenings. Static load tests were already conducted on 16 test piles. The summary data of those test piles are illustrated shown in Table 3. The test piles comprise the piles with diameters of 1.2 and 1.5m. They were tested with the maximum loads at 2.5 of safe working loads. The soils or rocks beneath the pile tips of the test piles consists of either siltstone, sandstone, clay or sand with the socket lengths in the rocks ranging around 0.3 – 9.8 m. depending on the bearing capacity of those sockets.

The rocks of this project are considered as weak rocks due to their properties. The average UCS value from the test was about 12.77 MPa which is classified as weak rocks according to Robin (1992) and Kulhawy (1991). Furthermore, the quality of rock needs to be considered. The rock at NBH-148 is visually classified as highly weathered. Their RQD are uncertain and range between 0-25% for the depth of 4.5-9.0 m., 60-65% for the depth of 9.0-11.5 m. and 20-35% for the depth of 11.5-14.5 m. Moreover, stiffness affects bearing capacity of rock. Osterberg and Gill (1973) gave theoretical load transfer curves for different depths of sockets. The relations express that the ratio of rock to concrete stiffness increases when the ratio of load carried by the scoket base decreases. For the rocks of this project, the average E_{50} is 3.14 GPa. Compared to Coon and Merritt (1970), when RQD was lower than 57%, the ratio of elastic modulus of rock mass (E_m) to elastic modulus of intact rock was about 0.15. Therefore, E_m of the rocks will be merely around 0.47 GPa which is significantly lower than that of concrete (≈25 GPa). In order to consider these conditions when designing, the strength of rocks need to be reduced, possibly using the approaches proposed by Kulhawy (1987), Zhang (2010) or AASHTO (1996).

Table 3 Details of Test Piles (Mahakhotchasenichai et al. 2018)

No.	NBH	Dia. (m.)	U.L.(1) (T)	W.L.(2) (T)	Pile Tip (m.)	Socket length (m.)	Soil or rock at the pile tip	D.O.W.(3) of rock at along pile shaft
1	027	1.2	1,750	700	9.297	0.297	Siltstone	F
2	030	1.2	1,750	700	36	-	Clay	-
3	034	1.2	1,750	700	14.5	4.0	Siltstone	Sli-Mo
4	041	1.2	1,750	700	35.338	-	Clay	-
5	044	1.2	1,750	700	19.8	3.3	Sandstone	Sli-Mo
6	052	1.2	1,750	700	20.3	8.3	Siltstone	Mo-Hi
7	078	1.2	1,750	700	18.6	7.6	Siltstone	Sli-Mo
8	099	1.2	1,750	700	15	4.5	Siltstone	Mo
9	101	1.2	1,750	700	15.5	-	Clay	-
10	132	1.2	1,750	700	16.5	2.5	Sandstone	Sli
11	134	1.2	1,750	700	15	1.0	Sandstone	F
12	137	1.2	1,750	700	18.5	-	Sand	-
13	138	1.2	1,750	700	35	-	Sand	-
14	146	1.2	1,625	650	22	-	Clay	-
15	148	1.2	1,875	750	14	9.5	Siltstone	Hi
16	148	1.5	2,500	1,000	14.3	9.8	Siltstone	Hi

(1)Ultimate load, (2)Working load, (3)Degree of weathering, F = Fresh, Sli = Slightly weathered, Mo = Moderately weathered, Hi = Highly weathered.

The relationships obviously shown that the uniaxial compressive strength ratios were directly proportional to RQD. Kulhawy and Prakoso (2001) also proposed the relationships of degree of weathering and intact properties of rock. UCS of rocks can decrease to 80% when degree of weathering is in highly weathered level. These conditions make the design of a pile more complex. Theoretically, they must be considered in order for a pile to gain sufficient bearing capacity.

3.2 Extrapolation method

The maximum loads applied in the static load tests might not appropriate for the piles to reach failure criterion. In order to calculate settlement, Mahakhotchasenichai et al. (2018) adopted extrapolation methods to estimate the ultimate loads of all piles using the methods of Brinch-Hansen (1963), Chin (1970), Ahmad and Pise (1997), Mazurkiewicz (1980), and Decourt (1999). The results were simply shown in Table 4.

Table 4 Summary of extrapolated ultimate failure (Mahakhotchasenichai et al. 2018)

No.	NBH	Ultimate load (T)	Decourt (1999)	Ahmad and Pise (1998)	Mazur. (1980)	Hansen (1963)	Av.
			Extrapolated ultimate failure load (ton)				
1	034	1,750	2,900	2,823	2,000	2,490	2,553
2	041	1,750	2,500	1,882	2,000	2,204	2,147
3	052	1,750	3,000	3,623	1,950	2,970	2,886
4	138	1,750	2,900	1,689	2,050	N/A	2,213
5	146	1,625	3,200	2,989	2,000	N/A	2,730
6	(1.2)148	1,875	N/A(1)	2,920	2,150	N/A	2,535
7	(1.5)148	2,500	3,700	2,550	2,750	3,138	3,034
8	044	1,750	N/A	N/A	1,950	N/A	1,950
9	030	1,750	2,000	2,103	1,900	1,790	1,948
10	027	1,750	2,100	1,872	2,150	1,964	2,021
11	078	1,750	N/A	N/A	2,000	N/A	2,000
12	134	1,750	2,100	1,987	1,900	1,927	1,979
13	137	1,750	3,000	2,266	2,000	2,770	2,509
14	132	1,750	3,000	2,094	2,000	2,966	2,515
15	099	1,750	2,300	1,919	2,000	2,030	2,062
16	101	1,750	3,300	2,548	2,050	3,959	2,964

(1)N/A is not applicable

3.3 Estimation for elastic modulus of rock mass (Em)

End bearing capacities of piles socketed in weak rocks depend on rock mass properties; the whole mass of rocks underneath the pile tips is subjected to the transferred loads. For this research, RQD of rocks affects the properties of intact rock by the relationship of E_m/E_i proposed by Zhang and Einstein (2004).

3.4 Pile Settlement Prediction

Because of the uncertainty of soils or rocks and insufficiency of data in design, the pile may have potentially excessive settlement. One of the key successes of pile design is to control the settlement of piles within the allowable value. Many researchers proposed equations to predict settlement of piles. In general, pile settlement are the sum of pile shortening and toe movement which is the deformation of soils or rocks beneath the pile bases.

$$\delta_h = \delta_s + \delta_t \quad (1)$$

where:

δ_h = settlement at pile head

δ_s = pile shortening

δ_t = pile toe movement

Consequently, the estimation of the magnitude of settlement should consider soil and rock properties surrounding pile shafts and at the tips of piles and properties of piles themselves. **Hooke's law** is a simple principle applied to explain the state of force needed to compress or extend a spring which can be applied to calculate pile settlement by assuming a pile reacting as a spring. The applied load starts pressing on the top of the pile head and transfers along the segments, as it is reduced by frictional resistance of soil/rock layers. After this, the remaining load is transferred to the lower segment and finally to the pile toe. In this state, each segment of the pile is compressed and shortened, the pile shortening can be calculated by:

$$\delta_s = P_i L_i / A_i E_{pi} \quad (2)$$

where

Pi = pile axial force of each segment

Li = length of each segment

Ai = area of pile at segment i

Epi = elastic modulus of pile at segment i

The pile load is carried and consumed by the friction resistance of each layer. After that, the remaining load is transmitted to a soil or rock layer at pile tip. The base load compresses the ground beneath the pile tip and creates movement. Pile toe movement can be calculated from:

$$\delta_t = P_t D_t / A_t E_s \quad (3)$$

where

Pt = transferred load at pile toe

Dt = pile diameter at pile toe

At = cross-section area of pile at pile toe

Es = elastic modulus of soil/rock at pile toe

Note that this approach considers that the pile and soil/rocks behave as elastic materials.

Bowles (1996) proposed a method to estimate pile settlement by dividing the magnitude of settlement into 2 parts; pile shortening of each segment of piles which is the same term with Hooke's law and the point settlement or pile toe movement.

$$\delta_h = \sum [P_i L_i / A_i E_{pi}] + \left(\Delta q D \left(1 - v^2\right) m l_S I_F F_1\right) / E_s \quad (4)$$

where:

Δq = bearing pressure at pile base = base load/At.

v = Poisson's ratio

mls = 1.0 (shape factor)

I_F = Fox embedment factor, as follow:

I_F = 0.55 if L/D ≤ 5

I_F = 0.50 if L/D > 5

F_1 = reduction factor, as follows:

0.25 if the axial skin resistance reduces the point load Pt ≤ 0

0.50 if the point load Pt > 0

0.75 if point bearing (there is always some skin resistance)

Vesic (1977) presented the calculation of pile settlement different from the others by dividing the settlement into three components. The first is the term of pile elastic shortening (δ_s) and the second and third term are the settlement at pile point caused by load transferred at the point (δ_t) and surrounding the pile shaft (δ_{ps}), respectively.

So, the three components can be written in a general formula as follow:

$$\delta_h = \delta_s + \delta_t + \delta_{ps} \quad (5)$$

The pile shortening can be determined by assuming the pile materials are in elastic behavior and strength of materials is well-known.

$$\delta_s = \left(Q_{wp} + \alpha_s Q_{ws}\right) L / A_p E_p \quad (6)$$

where

Q_{wp} = the actual point load transferred by pile in working stress

Q_{ws} = the actual skin friction load transferred by pile in working stress

α_s = the magnitude of this coefficient varies between 0.67-0.50 depends on type of load distribution

L = length of pile

A_p = area of pile

E_p = elastic modulus of pile

The pile point settlement for both components can be found by assuming that the soils or rocks around the pile shaft and tip of pile toe behave in elastic condition. The pile toe and pile shaft settlement consider pressure at the pile toe and average unit skin friction transferred along the pile shaft, respectively. The solutions can be written as follows:

$$\delta_t = q_{wp} D\left(1-v^2\right) I_{wp} / E_s \tag{7}$$

$$\delta_{ps} = Q_{ws} D\left(1-v^2\right) I_{wp} / pLE_s \tag{8}$$

where

q_{wp} = bearing pressure at pile base = Q_{wp}/At

D = width or diameter of pile

I_{wp} = influence factor ≈ 0.85

p = perimeter of pile

L = pile length

I_{ws} = influence factor = 2 + 0.35(L/D)0.5

Tomlinson (1995) proposed a method to calculate pile head settlement which is based on settlement at the pile shaft and base similar to the others. The solutions can be presented by:

$$\delta_h = \left(W_s + 2W_b\right) L / 2A_p E_p + \pi W_b D\left(1-v^2\right) I_p / 4A_t E_s \tag{9}$$

where

W_s = loads on the pile shaft

W_b = loads on the pile base

I_p = influence related to the ratio of L/B

It can be seen that all mentioned theories assume that piles, soils, and rocks behave as elastic materials and settlement would be calculated from the ultimate loads of piles. All piles of the project were tested at the maximum loads or 2.5 times of safe working load, which might not be the failure criterion, resulting in the settlement values may differ from the prediction. Moreover, the actual behavior of piles might not be in elastic. Therefore, to eliminate the variation, the finite element method is adopted together with the ordinary methods to estimate the pile settlement. In this research, for prediction of pile settlement from equation implementation, Young's modulus of clay and sand are determined to be 800S_u and 200N, respectively (adopted from [3]) and Poisson's ratio of soil is determined to be 0.25.

Finite Element in pile settlement analysis can be carried out by using state of art computer application PLAXIS 2D Connect edition from Bentley company. The analysis of a single pile is modeled in axis-symmetry mesh as presented shown in Fig. 3

Load testing from different pile types varied with underground condition. Pile has the same diameters of 1200 mm with toe depth between 9.3 m to 35.3 m. Pile toes are socketted into rock in certain pile length.

Plaxis 2D's default boundary condition is adopted. Deformation in any direction except y-axis are not allowed. Width of model is 30 m and depth is varied from 30 m to 50 m depend on pile toe. The boundary geometry is much greater than six times of required distance of pile structure dimension. This requirement is mainly to avoid deformation effects of model boundary.

Material model of pile is set to non-porous with linear elastic material. Material parameters is presented shown in Table 5.

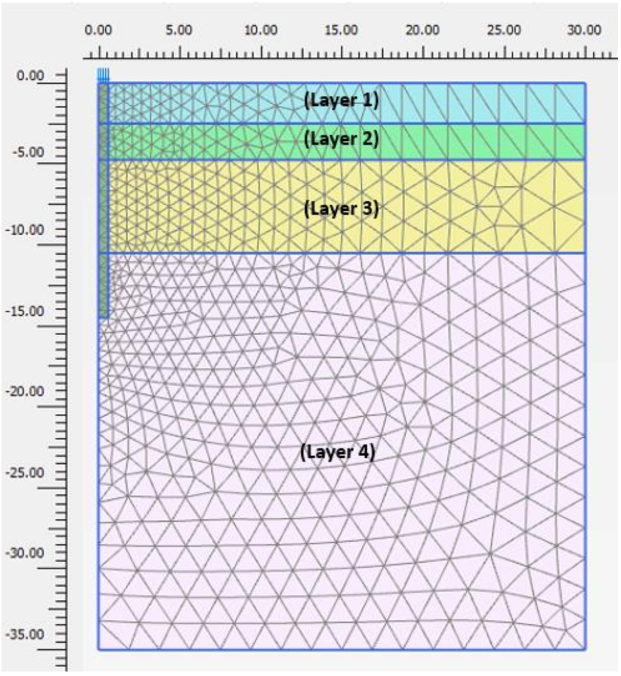

Fig.3 Numerical analysis in pile settlement

Table 5 Pile properties for numerical analysis

Type	γ (kN/m^3)	E (kN/m^2)	ν	Ineterface
Pile	23.5	25.69x10^6	0.15	0.75

Geotechnical material ie. soils and rocks were analyzed using hardening soil with small strain (HSsmall). This model presents strain-dependent stiffness where stiffness is higher while strain level is in small portion then, decrease when strain is higher. HSsmall accurately provide deformation comparing to simple linear-elastic model such as Mohr-Coulomb.

The hardening soil parameters used in this analysis were adopted from Thanyatorn B. 2016 that predict settlement of bored pile in Bangkok area and rock parameters were adopted from Mahakhotchasenichai et al. (2018). The soil stratum properties from uppermost layer 1 to lowermost layer 4 are, stiff clay with unit weight of 18.64 kN/m3 and SPT of 20 blow/ft then, dense sand with unit weight of 20.60 kN/m3 and SPT varied from 24 to 70 blow/ft then, hard clay with unit weight of 18.50 kN/m3 and SPT higher than 50 blow/ft then lowermost silt stone layer which has properties refers [9].

4. Results and discussions

4.1 Static load test

16 test piles of the project were already tested by using the static load test method. The results of those test piles are presented in the relationships of pile head settlement and applied load and shown in Fig 3. It can be evidently seen that most of the piles have pile head settlement between 5-10mm. for maximum test loads. In order to report and meet other standards, the pile head settlement and the applied loads are normalized by their diameters and safe working loads. The relationships were newly presented in Fig 4. It is explicit that the ratio of the settlement to diameters of the most piles is lower than 1%. Eurocode 7 presented that a pile attains a failure criteria when pile head settlement is equal to 10% of its diameter. Therefore, all piles in the project would not achieve the failure criterion. Furthermore, Ng et al. (2001) also proposed that the movement for mobilization of toe resistance capacity was specified to be 4.5% of pile diameter. Even though whole movements of all test piles of this project relating to failure were not reached according to Eurocode 7 and Ng et al. (2001) mentioned, soil and rock layers carrying skin friction were fully mobilized.

The figure also illustrates that the settlements of all piles suddenly increase at the ratio of applied load to safe working loads around 2.25. It indicates that the whole applied loads of the first range of the pile head settlement are carried by skin friction of those piles. After the applied loads increasing to 2.25 times of the safe working loads, the remaining loads start transferring to the bases of the piles, resulting in the increase in deformation of the soils and the rocks beneath the pile tips.

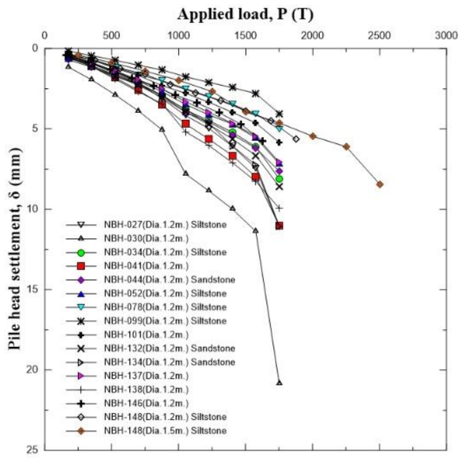

Fig. 4 Applied load vs pile settlement for 16 test piles

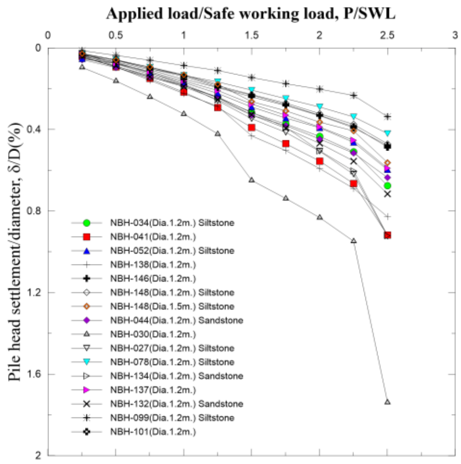

Fig. 5 Normalized applied load vs pile settlement for 16 test piles

Figure 6 to Fig. 7 show examples of load distribution acting on the piles over their length obtaining from strain gauges installed in rebar cages. It is explicit that the applied loads are only slightly transferred to the bases of the piles, which corresponds to Applied load vs Pile head settlement relationship in Figure 4 and 5. If pile head settlement at 10%D from Eurocode 7 or pile toe movement at 4.5%D from Ng et al. (2001) is set as settlement at an ultimate load bearing capacity, settlement of all test piles does not reach those criteria. According to Ministerial Regulation No.6 of Thailand mentioning about pile head settlement that if pile head settlement after maintaining a maximum applied load for 24 hours does not exceed 25mm., it is considered that a pile passes the test criterion.

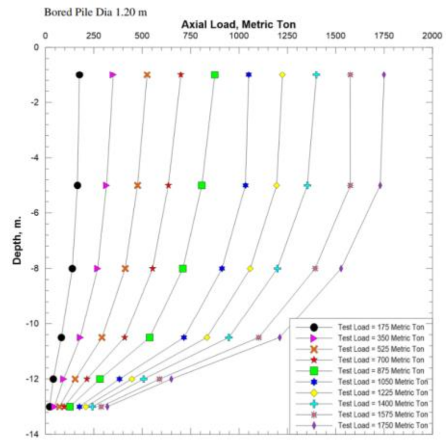

Fig. 6 Load distribution along pile shaft NBH-034

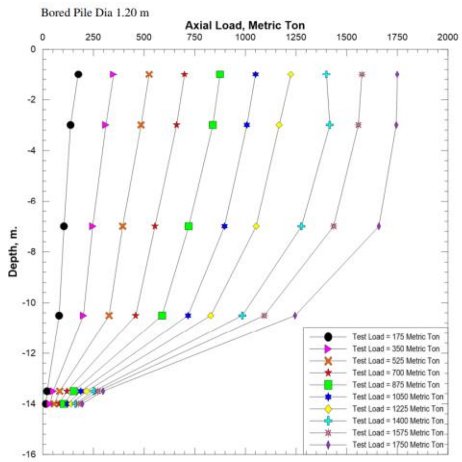

Fig. 7 Load distribution along pile shaft NBH-134

4.2 Analysis of pile movement

Figure 8 to Fig. 9 show example results of pile head settlement calculation using theories in section 3.4. the maximum loads from extrapolated loads in table 4, each applied load from static load tests and skin friction of each pile collected from strain gauges are applied to calculate pile head settlement values. Insummary, the settlement values are in the allowable range compared to Ministerial Regulation No.6 as shown in Table 6.

The pile head settlement in this study did not reach the failure state as mentioned even there are the extrapolation. It is therefore difficult to carry out pile toe movement. Nonetheless, pile shortening of the piles occurs at full mobilization of soil or rock surrounded pile shafts. Details of pile shortening analysis will explain further in the next sextion.

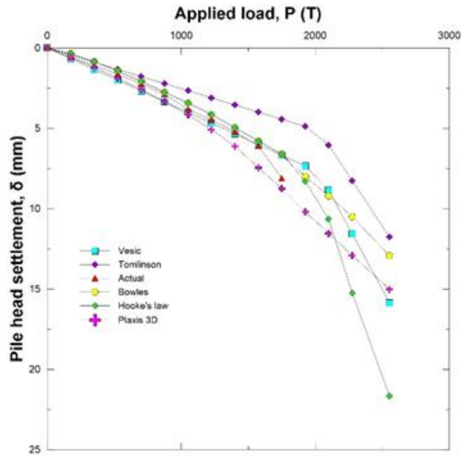

Fig. 8 Applied load vs pile settlement of NBH-034 from measurement and calculation.

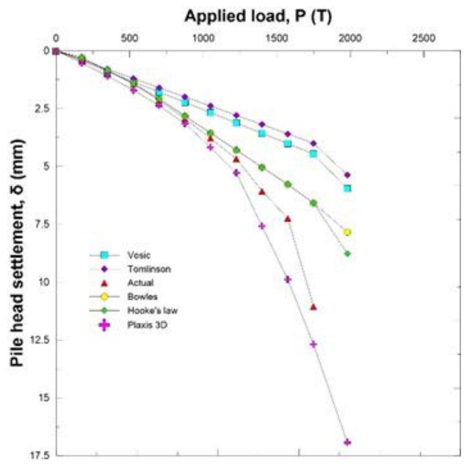

Fig. 9 Applied load vs pile settlement of NBH-134 from measurement and calculation.

Table 6 Summary of extrapolated ultimate failure

No.	NBH	Ultimate load (T) from Measurement	Extrapolated load (T)	Pile head settlement (mm)				
				Hooke	Bowles	Vesic	Tomlinson	2D
1	034	1,750	2,553	21.67	12.93	15.86	11.77	15.02
2	41	1,750	2,147	23.29	20.80	16.50	15.47	-
3	052	1,750	2,886	22.42	18.27	16.18	13.82	12.30
4	138	1,750	2,213	20.28	19.35	16.38	16.07	9.76
5	146	1,625	2,730	27.64	18.78	18.87	17.00	-
6	148(1.2)	1,875	2,535	30.58	15.26	21.43	15.27	10.38
7	148(1.5)	2,500	3,034	26.95	12.52	19.61	13.17	-
8	044	1,750	1,950	8.47	8.30	8.36	6.25	8.19
9	030	1,750	1,948	25.30	19.68	18.02	16.47	12.69
10	027	1,750	2,021	9.47	6.56	7.41	5.35	9.56
11	078	1,750	2,000	8.56	8.56	7.48	6.12	10.28
12	134	1,750	1,979	8.76	7.84	5.97	5.37	16.94
13	137	1,750	2,509	37.86	19.00	24.09	18.57	-
14	132	1,750	2,515	13.91	12.19	10.05	9.47	10.46
15	099	1,750	2,062	16.28	10.13	11.49	9.03	-
16	101	1,750	2,964	24.49	14.24	15.90	13.54	-

4.3 Analysis of pile shaft shortening

The pile settlement prediction methods are adopted to all test piles to calculate pile shortening. The results are showed shortly in Fig. 10. The figure show the comparison between pile shortening values from the tests and pile shortening values from calculation. It can be evidently seen that the values from the tests lower that those from the calculations. In order to shows the results more clearly, the theories in section 3.4 are presented in graphs. First of all, from the relationship of actual pile shortening values versus Hooke's Law and Bowles (1996) as shown in Fig. 11, all values from the tests are lower than those in design. They are proposed In the term of ratio, approximately 0.55, 0.70 and 0.90 for bored piles placed on soil, siltstone and sandstone, respectively. The next relationship as shown in Fig. 12 can not be adopted to the bored piles placed on the soil due to lack of necessary data to define the soil properties. The results seem as near as those from Hooke's Law and Bowles (1996). The actual pile shortening values are around 0.55 and 0.80 times of Plaxis2D values for siltstone and sandstone, respectively. The results from Tomlinson (1995) and Vesic (1977) in Fig.13 differ significantly from the others. Apparently, the actual pile shortening values are as close as the design values, ranging from 0.80X to 1.35X. This relationship is notably distinct from the others, as it considers the load on the pile base in the term of pile shortening calculation, resulting in a greater value.

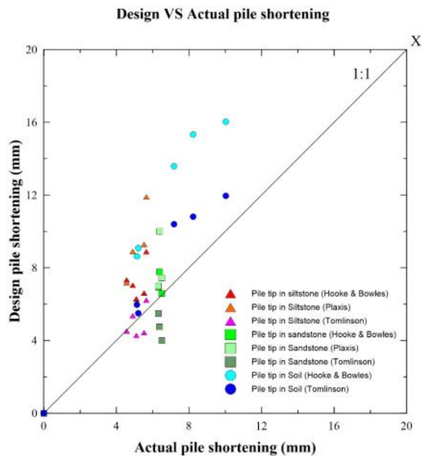

Fig. 10 Actual Pile Shortening VS Pile Shortening from Calculation in Summary.

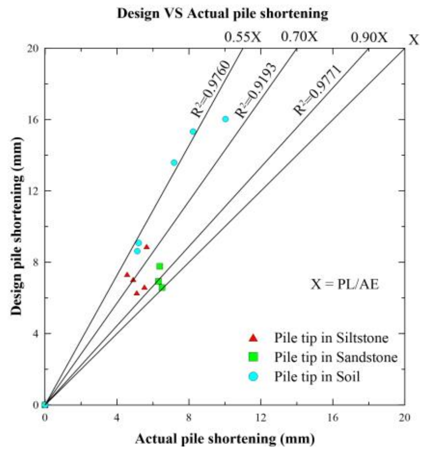

Fig. 11 Actual Pile Shortening VS Pile Shortening from Calculation by Hooke's Law and Bowles (1996) Method.

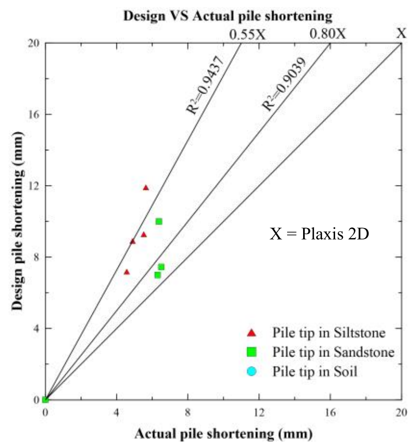

Fig. 12 Actual Pile Shortening VS Pile Shortening from Calculation by Plaxis 2D.

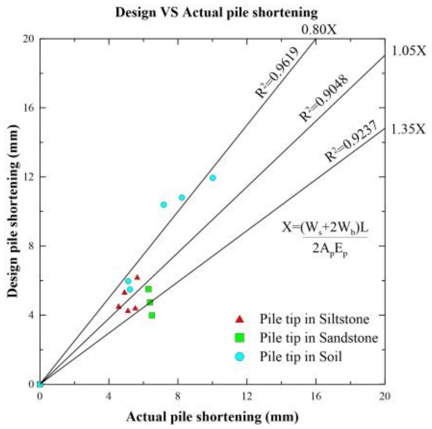

Fig. 13 Actual Pile Shortening VS Pile Shortening from Calculation Tomlinson (1995) and Vesic (1977) Method.

5. Conclusions

To sum up, all results are presented in a simple tabular format as shown in Table 7. It is implicit that most of the calculated pile shortening values are over than the actual pile shortening values except the values from Tomlinson's approach.

The estimation of pile shortening is very complicated. The magnitude of pile shortening mainly depends the mechanism of applied loads. The applied loads transferred along piles as skin friction, reacting to the piles causing them shorter, also depends on many factors such as soil properties, rock type, degree of weathering and fracture of rock, placed on soil, socketed in rock or construction details.

As mentioned in section 3.4, pile head settlement is equal to the summation of pile shortening and pile toe movement. Anywise, the bored piles in this study have not been tested until reaching the failure criterion despite performing extrapolation to estimate failure loads. It is therefore very difficult to figure out pile head settlement or pile toe movement in the failure state. However, there are many approaches presented out to carry out the pile head settlement or the pile toe movement. For example, Eurocode 7 : Geotechnical Design has presented that if it is difficult to define an ultimate state, pile head settlement equal to 10% of the pile diameter can be identified to be the pile head settlement at the failure criterion. Weltmen (1980) and ISSMFE (1986) also suggested that an applied load at pile head settlement which is equal to 10% of the pile diameter can be defined as the failure load as same as Eurocode 7. There are also suggestions to find out pile toe movement at the failure load such as pile toe movement at the failure load equal to 4-5% of the pile diameter from O'Neill (1988) or 4.5% of the pile diameter from Ng et al. (2001).

Table 7 Summary of Pile Shortening from Static Load Tests vs from Calculations.

Method	Fitting Equation	R Square	Soil/Rock at Pile Tip
Hooke's & Bowles $X=PL/AE$	0.55X(1)	0.9760	Soil
	0.70X	0.9193	Siltstone
	0.90X	0.9771	Sandstone
Plaxis 2D X	-	-	Soil
	0.55X	0.9437	Siltstone
	0.80X	0.9039	Sandstone
Tomlinson & Vesic $X=\frac{(W_s+2W_b)L}{2A_pE_p}$	0.80X	0.9619	Soil
	1.05X	0.9048	Siltstone
	1.35X	0.9237	Sandstone

(1) X refers to the equations for pile shortening calculation of each method in section 3.4.

6. Recommendations

In calculation of pile settlement values and bearing capacity for a pile of this research, the reduction of rock strength was determined by using approaches as mentioned in chapter 3. The relevant factors that affects to the strength of the rock was considered such as degree of weathering and fracturing state of rock. Therefore, before any implementation, authors propose that a pile designer should consider the aforementioned factors thoroughly to ensure data consistency.

With limitation of cost for a large-scale project in the future, a static load test should be performed on a small pile until reaching the failure criterion to obtain fully-mobilized movement of piles which is better to be used in further analysis

7. Acknowledgment

We would like to thank SEAFCO Public Limited Company for the information support.

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