The Vertical Bearing Capacity Behavior of Single Pile by Geotechnical Centrifuge

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Abstract

In 1960, Bangkok's groundwater level was at ground surface. Because of economic growth, the groundwater level decreased to a minimum due to an increase in groundwater pumping. Currently, the groundwater is rising back to match the ground surface as a result of groundwater control laws. In order to understand the pile's behavior, especially concerning the capacity of the pile foundations, calibration should be conducted before using a centrifuge model for testing. This research was carried out to compare the bearing capacity of the pile by testing the pile load through 1, using the centrifuge model, and 2, using hand calculations via Alpha and Beta methods. The results were consonant with the average unit skin friction with less than 25% difference. Unfortunately, the strain gauge setup in the sand layer malfunctioned, and data about its capacity was not obtained for comparison.

Keywords:

groundwater level; single pile; bearing capacity; centrifuge test

1. Introduction

The Bangkok area is located on the Chao Phraya river basin. The Department of Groundwater Resource [1] (2012) estimated that the groundwater level was at ground surface in 1960 (57 years ago). Because of the economic growth, groundwater pumping increased, groundwater level decreased, and land subsidence occurred. Then, in 1997 groundwater level decreased to the minimum level. Regulations from the water pumping control laws not only halted groundwater reduction, but also heralded groundwater recovery.

Many researchers have studied the possibility of pile bearing capacity reduction and pile movement due to groundwater changing. Wilkinson [2] (1984) discussed the problem in the London area and undrained shear strength when pore water pressure increased, the bearing capacity decreased by 50% due to the reduction of effective stress. Armishaw and Cox [3] (1979) studied the effect of the increasing of groundwater level to pile capacity of a driven pile in sand and gravel overlain by a peat and clay layer using static pile load tests. The results showed the loss of total pile capacity and indicate that the percentage loss of skin friction was more than the end bearing.

The aim of this study is to compare the bearing capacity of the pile, of the initial stage (groundwater level at ground surface) measuring from a centrifuge model and calculating from hand calculations in order to know the accuracy of the model.

2. Geology of Bangkok

Bangkok, especially its urban areas, is covered by thick marine clay and alluvial soil deposits. The first layer is top clay crust followed by soft clay with a thickness ranging from 5 to 15 meters and followed by 5 meters of medium stiff clay. Underneath this clay layer is the first sand layer that is 20 to 25 meters deep, followed by a layer of clay with medium stiff to stiff density. The second sand layer is located 45 to 65 meters from the surface. Finally the bedrock is found deeper than 450 meters from the surface. The soil profile is shown in Figure 1.

3. Scaling law for Centrifuge Modeling

There are some major problems concerning geotechnical testing. First, the behavior of soils depends on the stress field such as behavior of soil at great depth with high stress level, which is difficult to recreate in 1 g. Then the particle size and size of soil structure are also difficult to prepare in a laboratory because a particle's size in a model test is smaller than in a field test. Third, some problems include either increasing or replicating the stress field of the model to match those of the real field or full scale tests. Since the cost of the field tests is very expensive, compared with the value of the results, geotechnical centrifuge can suffice for such testing to solve the complex geotechnical problems. The general principles of centrifuge

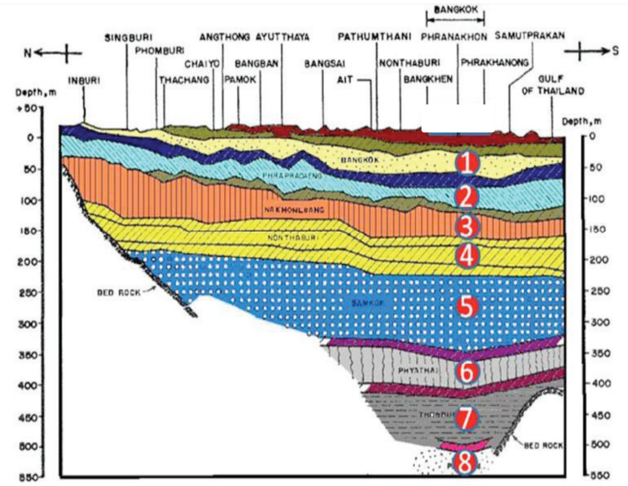


Figure 1 Geologic profile of the Chao Phraya river basin central plain of Thailand. (Aye Z. and Boonyarak T. [4].)

modeling include either increasing or recreating the stress field of the model similar to real field or full scale tests. The stress levels in geotechnical centrifuge model can be increased by applying the centrifugal force, which greatly increase the gravitational level above the normal gravitational level. When gravitational level increase in relation to the size of the centrifuge model, N time is reduced for every N gravitational increased, called "Scale Factor." The centrifuge test is an advance test that most researchers use to analyze and predict the behavior of soil in situations including building equilibrium, slope stability etc. Single pile capacity due to changing ground water consists of many factors such as stress level, groundwater condition, size of the structure and time factor that is difficult to control and prepare. So centrifuge tests are suitable in addressing this problem. The fundamental principle of centrifuge modeling will recreate the stress condition from prototype scale to model scale. When rotating the centrifuge by applying constant radius and constant angular velocity through the center of the centrifuge arm, the g level (g is the acceleration of gravity) will also increase.

Geotechnical materials such as soil have nonlinear behavior and mechanical properties that depend on stress histories of soil. The centrifuge increases the gravitational acceleration of the physical model in order to reproduce identical self-weight stresses in the model scale as prototype scale. The reproduction of stress level enhances the similarity of the physical models and makes it possible to obtain accurate data to solve the complex geotechnical problems. Centrifuge model testing provides data to improve our understanding of basic mechanism of deformation and failure, while providing benchmarks useful for verification of numerical model. Bucky [5] (1931) suggested a method that "To produce at corresponding points in a small scale model, the same unit stresses that exist in a full scale structure, the weight of materials in the model must be increased in the same ratio that the scale of the model is decreased with respect to the full scale structure. The effect of an increase in weight may be obtained by the centrifugal force, the model being placed in a suitable revolving apparatus." If a 1/N scale model of a prototype is spun at N g in the centrifuge, then behavior of model will be close to the behavior of prototype or full scale, which the scaling law summarizes in Table1. Most details can be seen in the scaling laws and centrifuge application by Schofield [6] (1980), Taylor [7] (1995) and Ng et al [8] (2006). The first step of preparation is to replicate a model, which is an exact and scaled version of the prototype. The scaling relations between the model and prototype should be satisfied. For example, a 30 m thick sand layer in a prototype can be modeled in a centrifuge model by using 300 mm thick sand accelerated at 100g, based on the stress similarity between model and corresponding prototype.

In this study the groundwater level increased to ground surface from unsaturated state to saturated state during preparation. So, the pore water pressure distribution in this test is very important. H. Nakajima and A. T. Stadler. [9] (2006) compare the pore water pressure distribution in centrifuge model and prototype or full scale test. The relationship between pore pressure distribution at hydrostatic condition in model scale and prototype scale is almost the same.

Physical quantity	Scaling factor (model /prototype)
Gravitational acceleration	N
Linear dimension	1/N
Area dimension	$1/N^2$
Volume dimension	$1/N^3$
Stress	1
Strain	1
Mass	$1/N^3$
Density	1
Unit weight	N
Force	$1/N^2$
Bending Moment / unit width	$1/N^2$
Flexural stiffness / unit width	$1/N^3$
Time(consolidation/diffusion)	$1/N^2$
Time(creep)	1
Velocity	1

Consolidation tests in full scale models may take a long period to finish the process including long term settlement of clayey soil. The major advantage of centrifuge model tests is that it requires less time than full scale model. Consolidation time in centrifuge model was reduced N2 times from prototype or full scale test.

4. Centrifuge model technique

The centrifuge model in this test was performed in the Geotechnical Centrifuge Facility (GCF) at the Hong Kong University of Science and Technology (HKSUT) as shown in Figure 2. The centrifuge has a rotating arm of 8.4 m in diameter. The maximum modeling capacity of the centrifuge is 400g-tons, while simulating an elevating gravity field over 150 times from the earth's gravitational field for static load test. The detail about centrifuge machine is represented in Table 2.

Table 2 Technical specifications for the centrifuge

Item	Detail
Payload capacity	400 g-tons.
Arm radius	4.2 m. to the base of the swinging platform.
Maximum acceleration	150g. (static tests) 75 g. (Dynamic tests)
Payload size	1.5m.x1.5m.x0.6x0.4m. for dynamic tests.

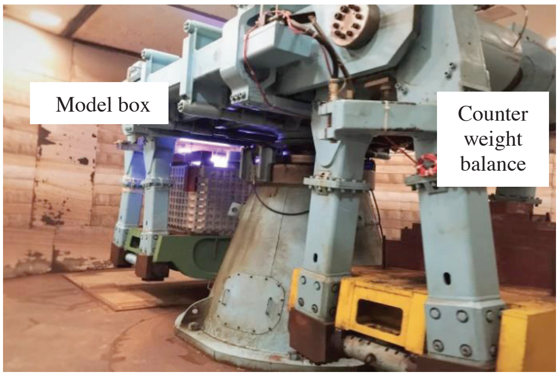


Figure 2 Geotechnical centrifuge

5. Model pile and machine

The single pile model was fabricated from aluminum tube as shown in Figure3. The dimensions of pile were derived using the scaling law. The requirement for pile model material is (EA)m equal to N-2(EA) p. where N is the scaling factor enhanced in a centrifuge test. Factor E is Young's modulus of material. Factor A is the cross section area of the pile.

The test has four types of instruments to measure stress and strain behavior (strain gauge), soil settlement and pile head settlement (linearly variable differential transformers LVDT), undrained shear strength (Vane shear test) and observation of pile movement (camera).

Measurement of stress and strain, due to the groundwater change may cause the stress and strain of pile and soil around the pile change. Four strain gauges were attached inside each layer of the pile model along the pile length to measure stress in every direction. The strain gauge results will convert to stress and force due to pile load tests in every step and compare each step together. The pile model has seven segments that install strain gauges inside the pile. The full Wheatstone bridge strain gauges were installed inside each segment of pile model to measure the axial force along the pile length. Figure3 shows the position of strain gauges from SG 1 near the pile head to SG 7 near pile tip. Strain gauges were protected by a thin layer of epoxy coating. Then, each segment is connected together by a thread and bound with leak tape for water protection. Before testing, the strain gauge must be calibrated to create the relationship between an applied load on pile and the corresponding reading of each full Wheatstone bridge.

Measurement of soil settlement, surface soil settlement due to groundwater level change and consolidation of soils was measured by LVDTs. All of LVDTs must be calibrated on the swinging platform in the centrifuge to obtain a displacement and output voltage relationship before using this test.

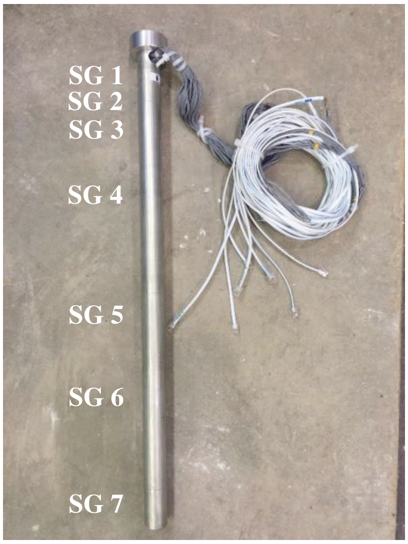


Figure 3 Aluminum pile

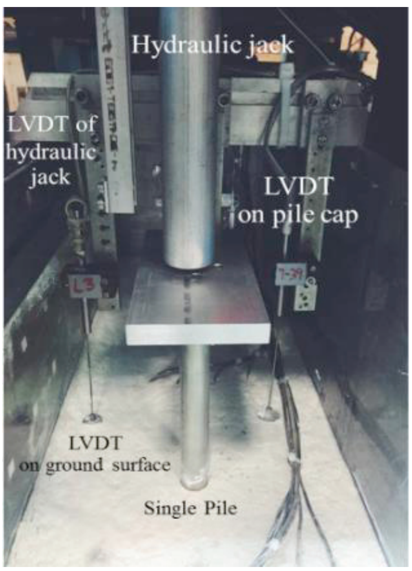


Figure 4 The instrumentation

6. Soil properties

Toyoura sand is a sub rounded to sub angular shape with uniform fine particles. The sand has an average diameter of 0.17 mm, a coefficient of uniformity of 1.7, a maximum and minimum void ratio equal to 0.977 and 0.597 respectively, a specific gravity 2.65 and a critical friction angle of 31^{o} (Verdugo and Ishihara, [10] 1996). Toyoura sand consists of 75% of quartz, 22% of feldspar and 3% of magnetite (Oda et al., [11] 1978). A summary of the sand properties is shown in Table3.

Kaolin clay that is used in this study is saturated clay. Around 80% of the particles are smaller than 0.02 micro-meters and the clay has specific gravity of 2.6, and a pH value of about 5.0 implying that the clay is acidic. A summary of the sand properties is shown in Table4.

Table 3 Properties of Toyoura sand

Properties of Toyoura sand	Value
Mean diameter, D_{50} (mm)	0.17
Uniformity coefficient, U_c	1.7
Maximum void ratio, e_{max}	0.977
Minimum void ratio, e_{min}	0.597
Specific gravity, Gs	2.65
Dry density, t/m3	1.73
Effective angle of friction at critical, ϕ’	31^{o}

Table 4 Properties of Kaolin clay

Properties of Kaolin clay	Value
Critical state angle of shearing resistance, ϕ'_{cr}	22°
Undrained shear strength, Su	8.8
Dry density, t/m3	1.73
Coefficient of earth pressure at rest, Ko	0.63
Saturated horizontal permeability, kh	3.49x109
Saturated vertical permeability, kv	2.58x109

7. Model preparation

The strong box must be sealed to prevent the leakage from the box during tesing. Internal dimension of the container is 350 x 1,245 x 850 mm. (length x width x height). The strong box was cleaned before model preparation. Each side of the strong box was coated with grease to reduce friction rexistant between soil and wall. The drainage System was installed at the bottom of the strong box as shown in Figure 5. Sand layer was placed by pluvial Toyoura sand from sand hopper into the model box with a constant rate at 50 mm per layer based on previous references to performed medium dense sand condition until 300 mm height (included filter layer). The sand layer was saturated by water reservoir, which the water head was maintained at the level equal to the top of the sand layer as shown in Figure 6. After saturation state, decrease water level in the model box to bottom of sand layer to compact clay layer. The clay layer was modeled using Speswhite Kaolin. The Kaolin powder was mixed with 27% water content. The compaction process was controlled by the constant density every 25 mm layer until 250 mm at dry density equal to 1.65 t/m^3. Then the model pile was installed at the middle of the box shown in Figure 7 (The model preparation set up for three tests but only Pile A was used in this study) and Figure 8. The bored pile was designed at 60 m depth (in prototype scale) below the ground surface. The test was designed to test in saturated clay during groundwater rising. After completing the model preparation and final check, cameras were set up to capture the photographs and video during testing. Two video cameras were installed to monitor the test. The data loggers was set to record data at frequency 1 Hz during spin down and spin up but record data at 0.1 Hz during perform load test. Upon centrifuge spin up, photos were taken at every 300 second and saved to the computer.

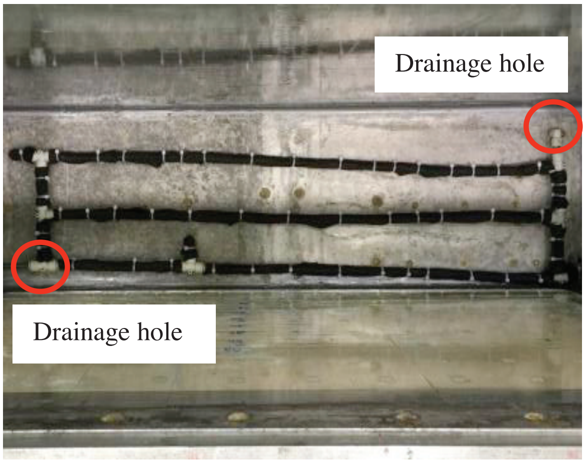


Figure 5 Standpipe on the bottom of the strong box

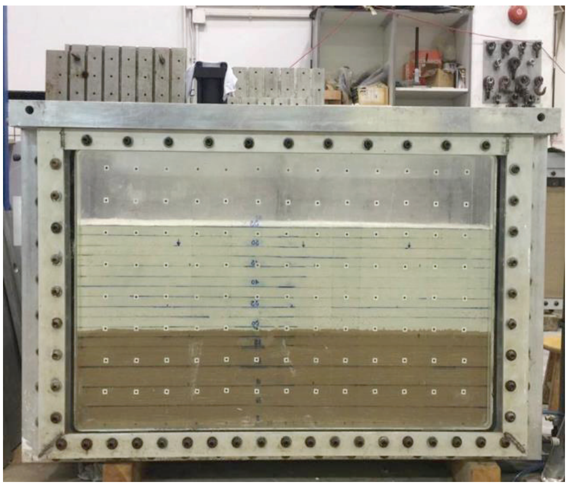


Figure 6 Sand and clay preparation.

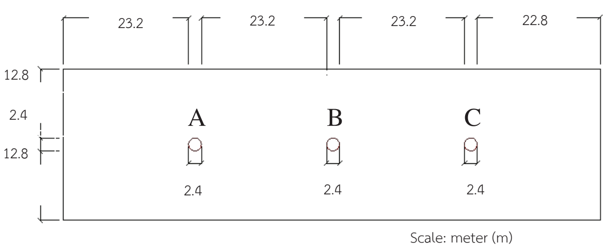
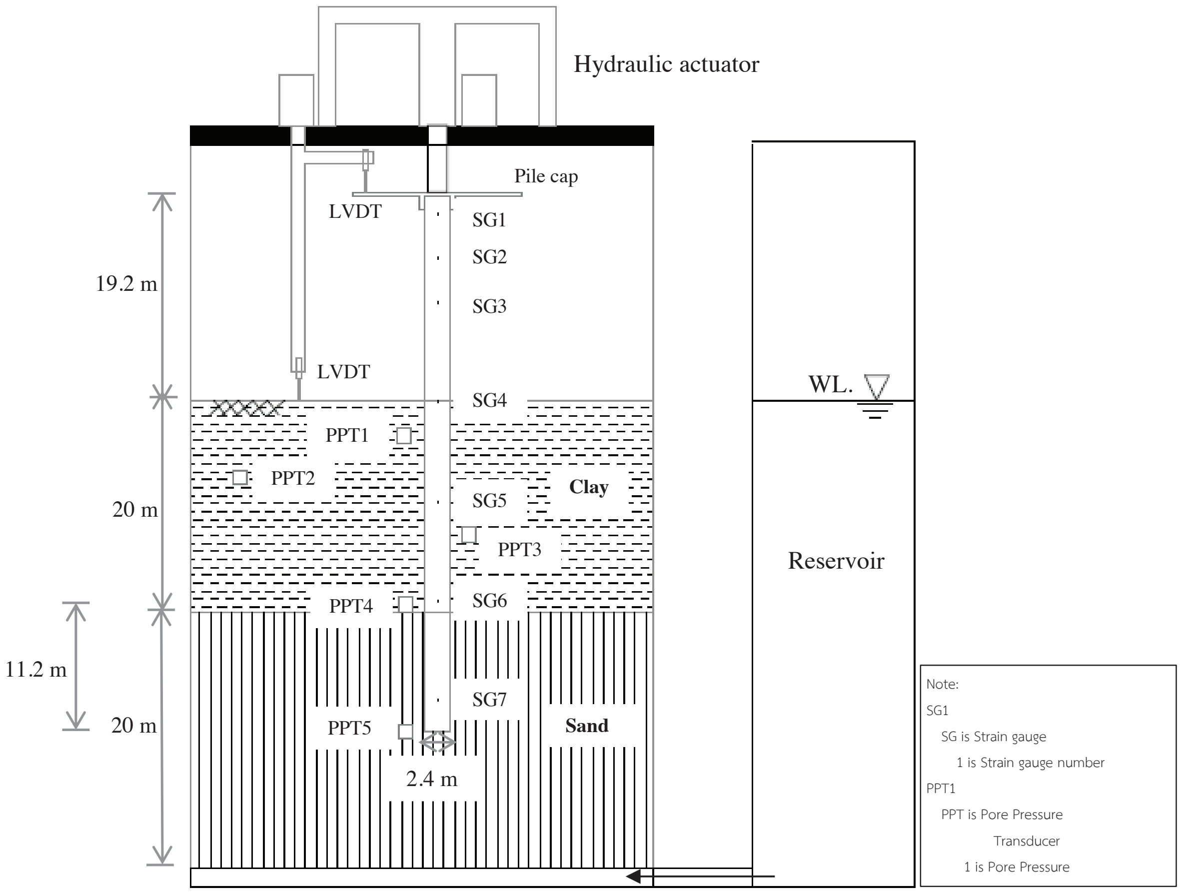


Figure 7 The top view of the test mode

8. Testing Procedures

The groundwater level was increased to the ground surface with the rate of 0.19 m/year. The dead load was applied before spin up. Afterwards, water was increased through Toyoura sand and Kaolin clay respectively, while waiting for the groundwater condition to reach a saturated state. When the acceleration of the centrifuge reached 80g and equilibrium condition was achieved, the camera system was changed to take photos every 30 seconds. The speed was increased to ensure enough time for the water to flow through the soil. After that, the first load test was performed on the head of pile until strain control failed (displacement control rate at 2 mm/min, reference to constant rate of penetration method for individual piles of the ASTM D 1148-81 [12] (1994)). The testing procedure is summarized in Figure9. After spin down, inspection of bearing capacity showed change due to consolidation and changes of groundwater level.



Drained Hole

Figure 8 The test model

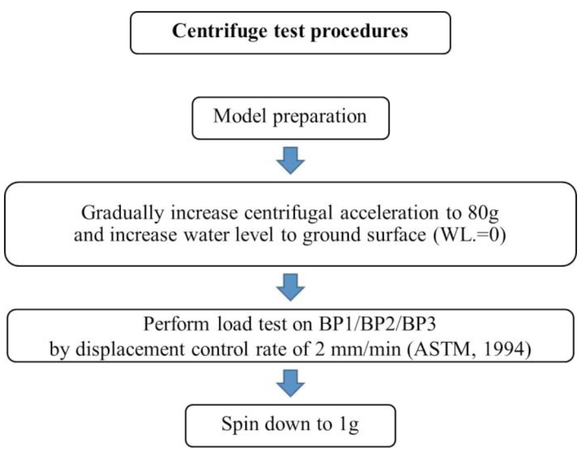


Figure 9 Geotechnical centrifuge

9. Results

1. Centrifuge Model

Figure 10 showed the changing of pore water pressure during model preparation to ensure saturated condition of the soils before doing a pile load test. Load tests were then performed on the pile head by displacement control method refer to ASTM D 1148-81 method. The relationship between load distribution and pile settlement during pile load test was presented in Figure 11. Unfortunately, there were only the strain gauges installed in stiff clay, SG4, SG5 SG6 that functioned well. So this study would show only three positions of strain gauges results shown in Figure 11. During the tests, soil settlements on ground surface rapidly increased with time.

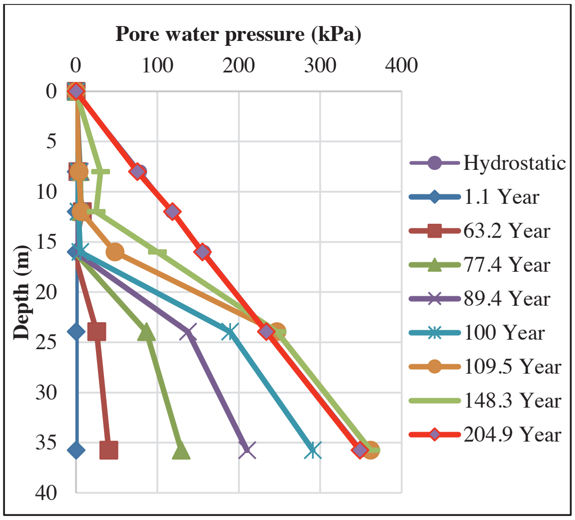


Figure 10 The relationship between pore water pressure and depth at water level

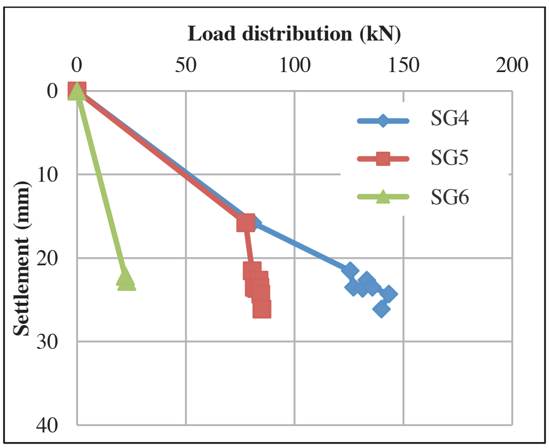


Figure 11 The strain gauge behavior during Pile load test

The soil settlements continuously increased since the spin up process due to self-weight and high gravitational field. The soil settlement curve still in primary consolidation stage because of behavior of settlements rate was not constant as shown in Figure 12. The settlement of pile cap rapidly increased during load test while ground surface settlement did not occur, showing that the settlement occurred only because of the additional load as shown in Figure13. After the test was completed, spin down was 1g. The reduction of gravitational field decreased the behavior of ground settlement. The permanent settlement occurred at 80 mm.

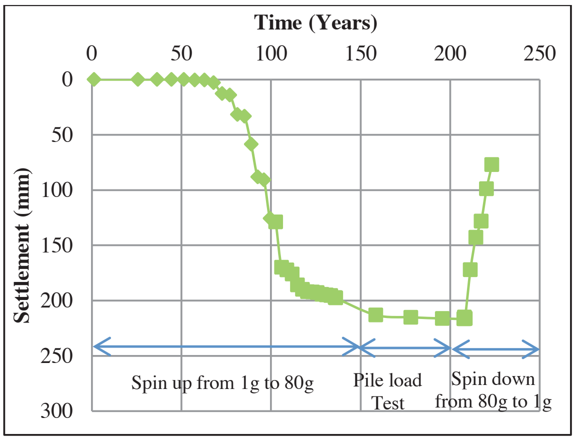


Figure 12 The settlement behavior of soil surface

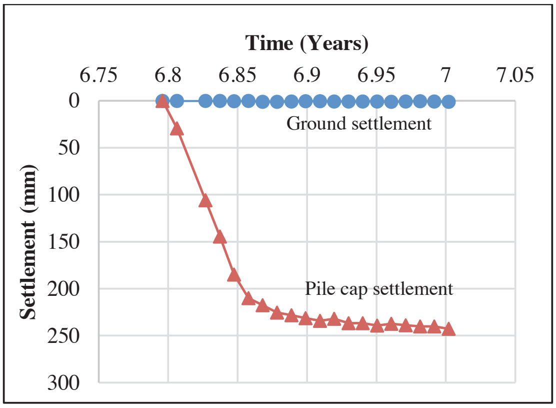


Figure 13 The settlement behavior during Pile load test

2. Hand calculation

Typically after a load test, T-bar test would be conducted to measure the strength of clay at any layers. However the clay was too stiff and thus the test wasn't conducted. Then the undrained shear strength was calculated using equations from previous studies from Steward [13] (1989), Springman [14] (1989), Phillip [15] (1987) and Garnier [16] (2002) as represented in Table 5. The undrained shear strength equation for each test depended on the over consolidation ratio (OCR) and the vertical effective stress. In this study OCR equal to 1 was used due to the Normally Consolidation state (NC). Figure 14 presented undrained shear strength of clay in each depth according to the calculations. The results for each studies were not similar. Then, these results can refer to Undrained shear strength of Bangkok clay from laboratory tests [17] from previous study found that the undrained shear strength value were not different.

Table 5 Undrained shear strength equation

Researcher	Undrained shear strength equation
Stewart, 1989	$S_u = 0.22\sigma'_v OCR^{0.57}$
Springman, 1989	$S_u = 0.19\sigma'_v OCR^{0.71}$
Phillip, 1987	$S_u = 0.19\sigma'_v OCR^{0.67}$
Garnier, 2002	$S_u = 0.19\sigma'_v OCR^{0.59}$
Average value	$S_u = 0.1975\sigma'_v OCR^{0.635}$

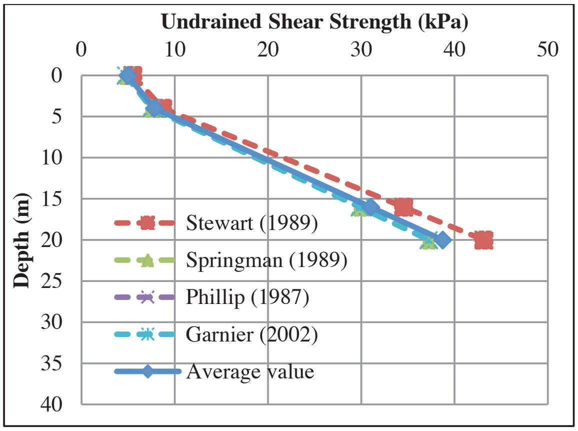


Figure 14 Estimate undrained shear strength from previous studies

3. Comparison

Figure 15 showed the comparison of the unit skin friction of centrifuge test results, hand calculation results and calculation from previous studies. In each method similar trends showed that strength gradually increased with depth and soil properties. The results were not perfect because the last strain gauge in the sand layer was not determined. Table 6 represented unit skin friction of each method showing that the results between the Centrifuge test and hand calculations were not different. The results from both methods showed a difference of less than 25 %.

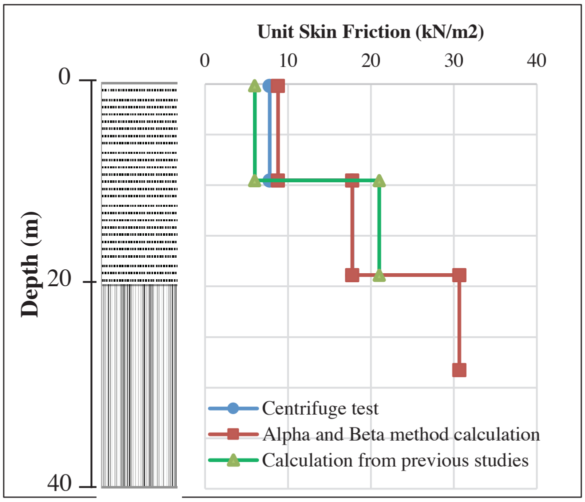


Figure 15 Unit skin friction developments

Table 6 Comparison of unit skin friction of soil in each method

Depth(m) /Method	The Unit skin friction (kPa)		
	Centrfuge test	Beta/Alpha Method	Calculation from previous studies
0 - 9.52	8	9	6
9.52 - 18.89	18	18	21
18.89 - 28.25	-	31	

10. Future work

Future studies of pile bearing capacity after the change of groundwater shall be conducted. Lastly, this research recommends that all instrumentations and equipment should be calibrated and protected from water, dust, and also from extreme gravitational field of centrifuge test, which can lead to erroneous results.

11. Conclusions

This study presented the comparison of bearing capacity of single pile by centrifuge test and hand calculation method to verify the accuracy of the model. The centrifuge model results showed that load distribution of single pile in centrifuge test was less than the hand calculation method but still in the same trend. The unit skin frictions in each depth of clay layer had different values. For sand layer, the strength results could not be presented, since the strain gauge in the sand layer malfunctioned. Nonetheless the centrifuge model provides an accurate bearing capacity compared with the hand calculation.

Acknowledgement

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