PERFORMANCE OF INSTRUMENTED BORED PILE IN KENNY HILL FORMATION

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Abstract

The Kenny Hill Formation forms part of the various geological formation and extends over a significant part of the Klang Valley of Peninsular Malaysia. Design of cast-in-place bored pile in Malaysia is usually based on empirical correlation with the Standard Penetration Tests (SPT-N). However, the current empirical correlation appears to indicate that the value may not be applicable for material with SPT-N>50. This paper presents the evaluation of measured shaft resistance, f_s and end bearing resistance, f_b of three instrumented cast-in-place bored piles constructed in the Kenny Hill Formation which were socketed into material with SPT-N value up to 300. The results are examined to validate the existing design method and to establish future guidelines for designing bored piles in this sedimentary formation. The correlation of f_{su} = 2N as suggested by Chang and Broms (1991) is achievable for material with SPT-N<50 blows/300mm. However, this correlation appeared to be unachievable for material with SPT-N>50 blows/300mm. It is recommended that lower value of K_{su} should be considered for SPT-N more than 50. In view of limited data for end bearing resistance, the correlation of f_b is almost reach to f_{bu} = 30N.

Keywords: Bored pile, kenny hill formation, standard penetration test, sedimentary, shaft resistance, end bearing resistance

Introduction

The development of pile design method to predict the bearing capacity of pile becomes a major concern of geotechnical engineer especially in large diameter bored piles. In Malaysia, the design of cast-in-place bored pile depends on the shaft friction of pile and usually ignores the contribution of the end bearing due to inconsistency of base cleaning quality. However, the design is mainly estimated based on the empirical methods that are correlated with Standard Penetration Test results (SPT-N).

The site of the test piles is located in Klang Valley area and is underlain by weathered meta-sedimentary of the Kenny Hill formation which generally consists of a monotonous sequence of interbedded phyllitic shale and thick bedded fine to medium grained sandstone that has undergone some degree of regional metamorphism. This formation also can be identified from the layers of clastic meta-sedimentary rocks, meta-argillite and meta-arenite. The overburden material of this

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formation generally consists of silt and sandy silt (Balakrishnan, 1999).

Three instrumented test piles are all cast-in-place bored piles of 1000mm and 1200mm diameter and were socketed into soil material with SPT-N values ranging from 4 to 300. Vibrating wire strain gauges and extensometers are installed in the test piles as instruments to assess the load transfer behaviour along the pile. These piles were load tested using kentledge and reaction beam as reaction systems to validate the design parameter adopted for the pile foundation. Kentledge system is usually adopted where there is no site constraint (Figure 1). However, if the maintained load test (MLT) is required in built-up areas, loading system using reaction beam would be adopted due to safety reason (Figure 2).

The test results interpretation of these preliminary instrumented bored piles are presented in this paper and provide evaluation of the measured shaft resistance and end bearing resistance obtained in the similar geological material and pile construction methodologies.

Overview of Bored Pile Foundation Design

The Standard Penetration Test is always carried out as part of the subsurface exploration and widely used to estimate the strength of varying soil strata. Semi-empirical correlations have been extensively developed relating both shaft resistance and end bearing resistance of bored piles to N-values from Standard Penetration Tests (SPT-N values). In the correlations established, the SPT-N values generally refer to uncorrected values before pile installation (Sew *et al*., 2003).

The design of bored piles in Malaysia is based on conventional design methods where the static ultimate pile capacity is usually estimated based on the following equation:

$$Q_u = Q_s + Q_b = f_{su}A_s + f_{bu}A_b \quad (1)$$

(Chang and Broms, 1991)

where

f_{su} = unit shaft resistance

f_{bu} = unit end bearing resistance

A_s = surface area of the pile

A_b = cross-sectional area of the pile

The commonly used correlation for unit shaft resistance and end bearing resistance of bored piles are based on coefficient factor of K_{su} (for unit shaft resistance) and K_{bu} (for unit end bearing) with the corresponding SPT-N values along the pile shaft and pile toe, respectively. Meyerhof (1976) suggested that the ultimate unit shaft resistance of bored piles can be estimated directly from the Standard Penetration Test value such as K_{su} = SPT-N. Studies carried out by Chang and Broms (1991) for instrumented piles in residual soil of Singapore suggested that K_{su} of 2 for bored piles with SPT-N≤150-180. Tan *et al*. (1998), presented K_{su} of 2.6 but limiting the f_{su} values to 200 kN/m^2 from the results of 13 nos. of fully instrumented bored piles in residual soils. Toh *et al*. (1989) also reported that the K_{su} values of 2.5 to 2.7 for weathered sedimentary formation in Malaysia appear to be reasonable for SPT-N≤120 and suggested to limit the SPT-N value to 200.

For base resistance, K_{bu} values reported by many researchers are varied depending on several factors such as consistent base cleaning during construction and mobilisation of end bearing resistance during load application. Chang and Broms (1991) suggested that K_{bu} equals to 30 to 45 can be considered satisfactory with critical displacement of 5 to 10%. Based on the results

Figure 1. Reaction system by kentledge

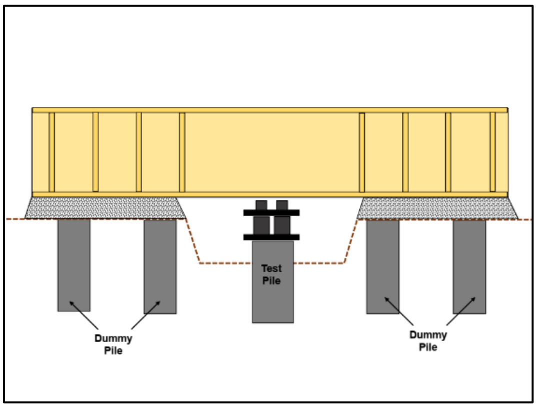

Figure 2. Reaction system by reaction beam

obtained from the two piles tested to failure, Toh *et al*. (1989) reports that K_{bu} falls between 27 and 60 with SPT-N values of 73 and 100 respectively.

The acceptance criteria for pile design usually related with the settlement of the pile carrying capacity. In Malaysia, this criterion has been published in the Guideline for Piling Works in 2017 by Ministry of Work (Public Work Department). The tested pile shall be deemed to have failed if the total settlement under design load (working load) exceeds 12.5 mm and the total settlement under twice the design load exceeds 10% of pile size (Jabatan Kerja Raya Malaysia, 2017).

Based on the above previous studies, this study was conducted to evaluate the validity of the established correlation of K_{su} value for unit shaft resistance and K_{bu} value for end bearing resistance to determine whether it is applicable to weathered meta-sedimentary material of the Kenny Hill formation.

Pile Installation and Testing

The subsoil profiles of the borehole carried out at the footprint of the test piles are shown in Figures 3 to 5. Test piles A and B have a diameter of 1,200 mm with penetration depth of 36.8 m and 48.1 m from piling platform level, respectively. Test pile C has a diameter of 1,000 mm with penetration depth of 35.1 m from piling platform level. Consistent soil overburden is encountered in all the test piles where the upper layer consists of alluvium material of silty sand and sandy silt, and the lower layer of weathered meta-sedimentary consist of sandy silt with SPT-N values more than 50 which is encountered at depth 12 m to 27 m below ground level.

The summary of the pile installation is shown in Table 1. All test piles were formed by auger drilling with polymer slurry for hole stability and concrete casting using tremie method. Temporary casing was driven for the top 5.5 m to 20.6 m of loose alluvial soils. Each of the pile was installed with 4 nos. of sonic logging tubes tied to the main steel reinforcement prior to concreting.

In order to assess the strain development and pile shortening behaviour during pile testing, at least six levels of vibrating wire strain gauge and extensometers are installed in the test pile. Each level consisted of four strain gauges and two extensometers which were welded to the main steel reinforcement as shown in Figure 6. The first level strain gauges were used as calibrating level, in which the load at this level was same as the applied load, to establish the actual pile stiffness. The subsequent levels of strain gauges were planned based on average SPT-N values in soil layers and at

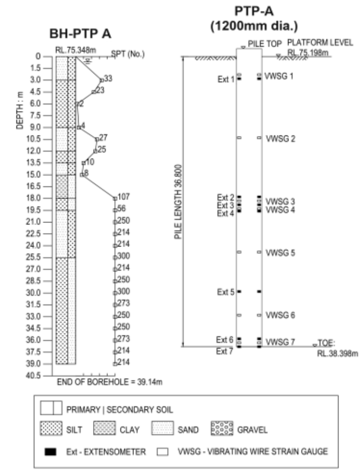

Figure 3. Test pile A

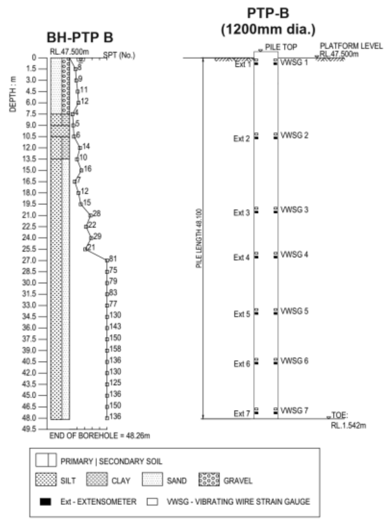

Figure 4. Test pile B

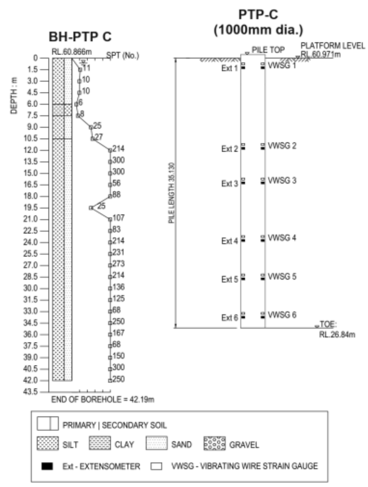

Figure 5. Test pile C

Table 1. Summary of pile installation records

Test Pile	Pile A	Pile B	Pile C
Pile Diameter	1,200 mm	1,200 mm	1,200 mm
Pile Length from Piling Platform	36.8 m	48.1m	35.1 m
Pile Working Load	9,800 kN	9,800 kN	7,200 kN
Drilling System	BG26	BG30	BG36
Stabilising Fluid	Polymer	Polymer	Polymer
Temporary Casing Length	20.6 m	8.8 m	5.5 m
Duration of Pile Drilling	7.5 h	10.5 h	8 h
Concrete Grade	40 MPa	40 MPa	40 MPa
Concreting Method	Tremie	Tremie	Tremie
Concrete Overbreak	27%	14.1%	18.9%

Figure 6. Test pile instrumentation

Figure 7. Load cells and pressure gauges

critical locations along the pile shaft. Additional devices such as calibrated load cells (four numbers) and pressure gauges attached to the hydraulic jacks were also used as primary load measurement and to verify the applied load to be in accordance with loading schedule (Figure 7).

Prior to carrying out the load test, Crosshole Sonic Logging test was carried out in accordance with method described in ASTM D6760 to verify the pile structural integrity in terms of concrete homogeneity, defect, and anomalies. The reason for the testing is to minimise the disturbance that may

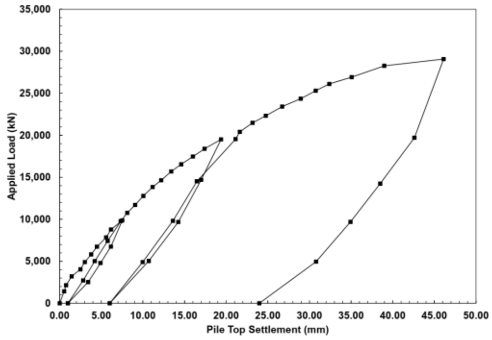

Figure 8. Load displacement curve for test pile A

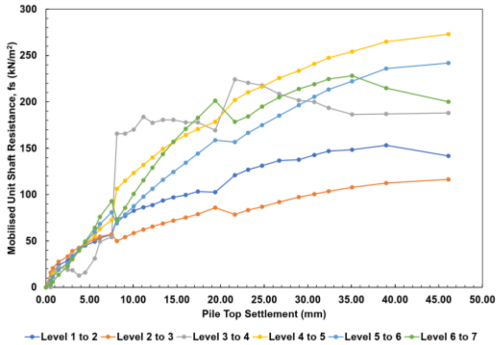

Figure 9. Mobilised unit shaft resistance for test pile A

affect the load transfer behaviour and interpretation of load testing results due to structural defect. Load tests were performed only after the piles had achieved minimum 28 days of compressive concrete strength (generally after 14 days) and upon satisfactory of the Sonic Logging test result. Generally, these piles were tested up to three (3) times of pile working load or to failure, which ever come first.

Results and Discussion

Test Pile A

The load displacement curve for Test Piles A is plotted in Figure 8. At 100% of pile working load, the measured pile top settlement was 7.5 mm. The observed pile top settlement in second and third cycles were 19.4 mm and 46 mm. Although the settlement is less than 10% of pile diameter, it can be seen from the curve that the pile has almost reach the ultimate condition. The shaft contributed about 90% (26,126 kN) while the base contributed about 10% (2,942 kN) at the maximum applied test load of 29,068 kN.

Based on Figure 9, the mobilised unit shaft resistance was ranging from 116.4 kN/m^2 to 224.2

Table 2. Summary of Mobilised Unit Shaft Resistance (Test Pile A)

Strain Gauges Level	Maximum Mobilised Shaft Resistance, f_s (kN/m^2)	Soil Layer	Average SPT-N of Subsoil Layer	Back Calculated K_{su} Value	
				Average N	Limit N to 150
Level 1 to 2	153.2	SILT	16	9.9	9.9
Level 2 to 3	116.4	Silty SAND, Sandy CLAY	18	6.7	6.7
Level 3 to 4	224.2	Sandy SILT	82	2.8	2.8
Level 4 to 5	273.0	Silty SAND	226	1.2	1.8
Level 5 to 6	241.9	Sandy SILT	267	0.9	1.6
Level 6 to 7	228.2	Sandy SILT	258	0.9	1.5
			Average	3.7	4.0

Table 3. Summary of Mobilised Unit Shaft Resistance (Test Pile B)

Strain Gauges Level	Maximum Mobilised Shaft Resistance, f_s (kN/m^2)	Soil Layer	Average SPT-N of Subsoil Layer	Back Calculated K_{su} Value	
				Average N	Limit N to 150
Level 1 to 2	39.7	Gravelly SAND. CLAY	8	5.0	5.0
Level 2 to 3	21.8	Clayey SILT, Sandy SILT	12	1.8	1.8
Level 3 to 4	281.9	Sandy SILT	25	11.3	11.3
Level 4 to 5	57.2	Sandy SILT	79	0.7	0.7
Level 5 to 6	389.4	Sandy SILT	145	2.7	2.7
Level 6 to 7	231.2	Sandy SILT	135	1.7	1.7
			Average	3.9	3.9

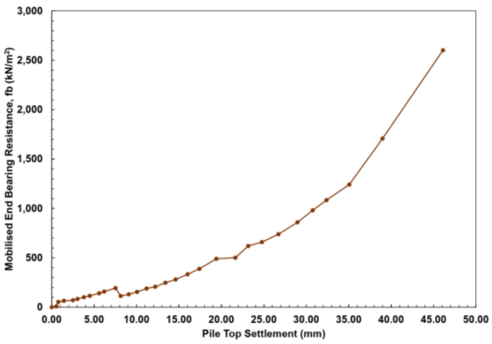

Figure 10. Mobilised end bearing resistance for test pile A

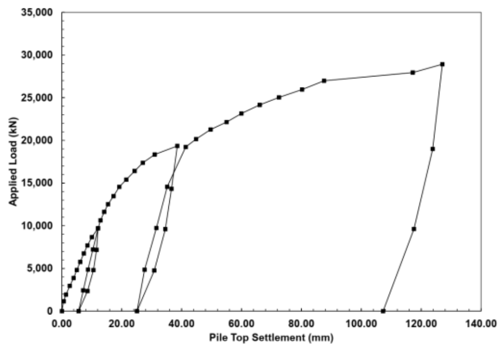

Figure 11. Load displacement curve for test pile B

kN/m^2 at upper layer and the maximum mobilised unit shaft resistance was 273 kN/m^2 at bottom layer of weathered meta-sedimentary with maximum pile top settlement of 46.1 mm. The maximum mobilised shaft resistance at each level of strain gauges is summarised in Table 2. By limiting the SPT-N value to 150, the back calculated K_{su} values shows that it ranges from 1.5 to 9.9 with an overall average of 4.0. It is observed that the top 18m with average SPT-N value of 16 to 82 has higher K_{su} values compared with bottom layer with average SPT-N value of more than 200.

The maximum mobilised end bearing resistance was 2,602 kN/m^2 and Figure 10 shows that the mobilisation curve of the end bearing has not reach its ultimate condition as most of the load had been transferred to the pile shaft. The back calculated K_{bu} is 17 with SPT-N value of 214 (limited to 150) at pile toe. The lower K_{bu} value could be due to the load has not been fully mobilised at the base.

Test Pile B

Figure 11 shows the load displacement curve for Test Piles B. The pile top settlement at pile working load was 12.1 mm, followed by pile top settlement of 38.6 mm in second cycle. During maximum test load in third cycle, excessive movement of 127.1 mm (more than 10% of pile diameter) was observed and the curve showing that

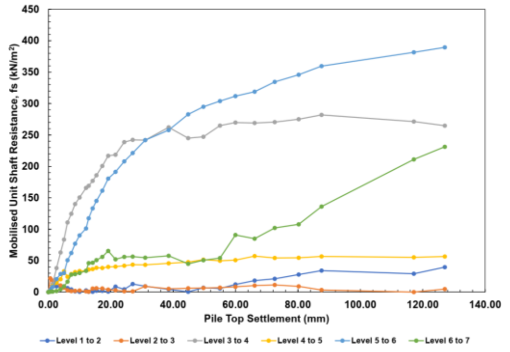

Figure 12. Mobilised unit shaft resistance for test pile B

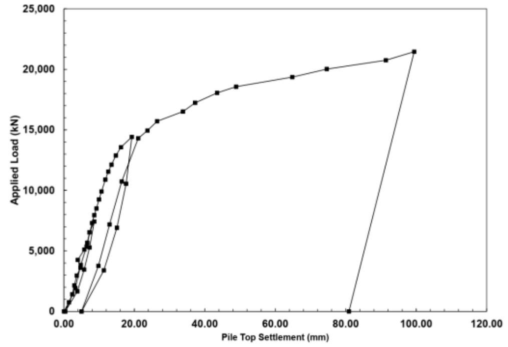

Figure 14. Load displacement curve for test pile C

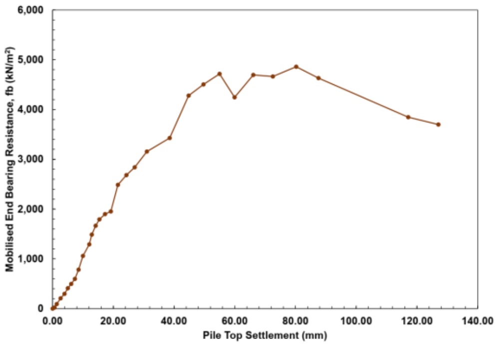

Figure 13. Mobilised end bearing resistance for test pile B

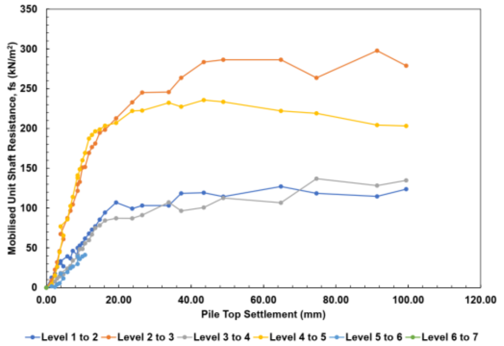

Figure 15. Mobilised unit shaft resistance for test pile C

the pile has reached its ultimate condition. The maximum applied test load was 29,400 kN with shaft contributed about 86% (24,736 kN) while the base contributed about 14% (4,184 kN).

Based on Figure 12, the mobilised unit shaft resistance was ranging from 21.8 kN/m^2 to 389.4 kN/m^2. Lower mobilised shaft resistance of 57.2 kN/m^2 is observed at strain gauge level 4 to 5 for average SPT-N value of 79. The maximum unit shaft resistance for weathered meta-sedimentary was encountered in the layer with average SPT-N value of 145. The maximum mobilised shaft resistance at each level of strain gauges is summarised in Table 3. It is observed that lower K_{su} of 1.8 is encountered for average SPT-N value of 12 at strain gauge level 2 to 3. Since the average SPT-N value for all layers are less than 150, the back calculated K_{su} values shows that it ranges from as low as 0.7 to as high as 11.3 with an overall average of 3.9.

The maximum mobilised end bearing resistance was 4,859 kN/m^2 and Figure 13 shows that the mobilisation curve of the end bearing has reached its peak of ultimate condition. The back calculated K_{bu} is 35.7 with SPT-N value of 136 at pile toe.

Test Pile C

Test Pile C is a 1,000 mm diameter pile and installed with six levels of strain gauges. The pile top settlement at pile working load as shown in load displacement curve in Figure 14 was 8.6 mm, followed by pile top settlement of 19.2mm in second cycle. During maximum applied test load of 21,454 kN in third cycle, movement of 99.4 mm was observed and the curve showing that the pile has almost reach the ultimate condition. The contributed shaft and base resistance were not able to be measured due to faulty strain gauges reading at level 6 (pile base).

Based on Figure 15, the mobilised unit shaft resistance at upper layer of material with average SPT-N value less than 50 was 41.0 kN/m^2. The maximum mobilised unit shaft resistance at bottom layer of weathered meta-sedimentary was 297.8 kN/m^2. The maximum mobilised shaft resistance at each level of strain gauges is summarised in Table 4. By limiting the SPT-N value to 150,

Table 4. Summary of Mobilised Unit Shaft Resistance (Test Pile C)

Strain Gauges Level	Maximum Mobilised Shaft Resistance, f_s (kN/m^2)	Soil Layer	Average SPT-N of Subsoil Layer	Back Calculated K_{su} Value	
				Average N	Limit N to 150
Level 1 to 2	127.2	Sandy SILT	14	9.2	9.2
Level 2 to 3	297.78	Sandy SILT	271	1.1	2.0
Level 3 to 4	136.95	Sandy SILT	72	1.9	1.9
Level 4 to 5	235.72	Sandy SILT	233	1.0	1.6
Level 5 to 6	41.00	Sandy SILT	110	0.4	0.4
			Average	2.7	3.0

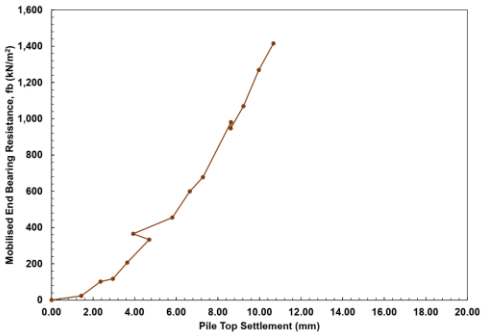

Figure 16. Mobilised end bearing resistance for test pile C

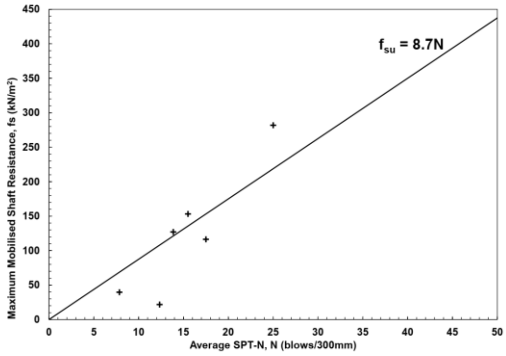

Figure 18. Relationship between unit shaft resistance and SPT-N (SPT-N <50)

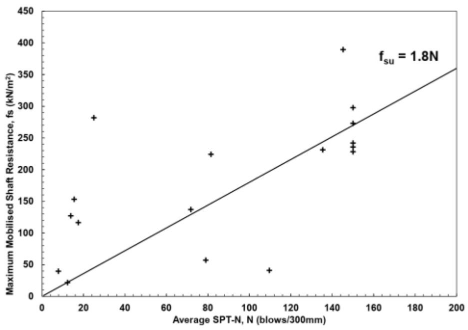

Figure 17. Relationship between unit shaft resistance and SPT-N (Overall)

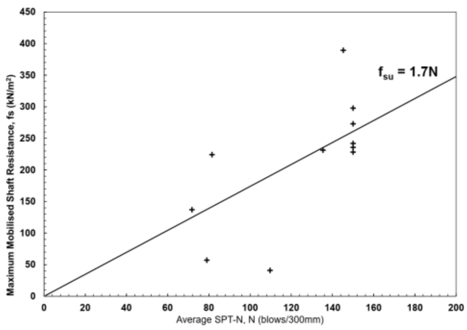

Figure 19. Relationship between unit shaft resistance and SPT-N (SPT-N >50)

the back calculated K_{su} values shows that it ranges from 0.4 to 9.2 with an overall average of 3.0. The lowest K_{su} value is observed at the last layer of strain gauge level 5 to 6 with average SPT-N value of 110.

As mentioned above, the maximum mobilised end bearing resistance at maximum applied test load was not able to be measured due to faulty strain gauge reading. However, the maximum mobilised end bearing resistance of 1,415 kN/m^2 was observed at the last strain gauge reading during applied load test to 140% in second cycle. Figure 16 shows that the mobilisation curve of the end bearing was far from reaching its ultimate condition could be due to most of the load had been transferred to the pile shaft. The back calculated K_{bu} is 21 with SPT-N value of 68 at pile toe.

Review of Mobilised Unit Shaft Resistance and End Bearing Resistance

All the maximum mobilised unit shaft frictions and average SPT-N values from Tables 2 to 4 were plotted to determine its relationship and validate the findings from previous literatures. The relationship between unit shaft resistance and overall SPT-N values (limited to SPT-N 150) is shown in Figure 17. It is observed the mobilised unit shaft resistance increases with increasing penetration resistance. The approximate value of K_{su} from the linear

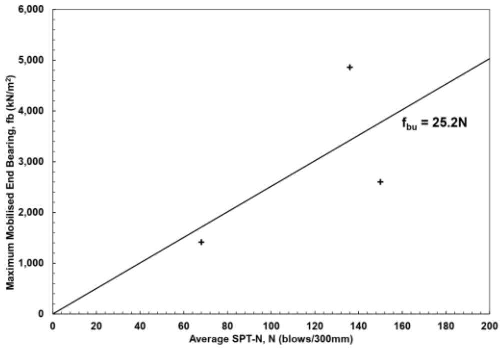

Figure 20. Relationship between end bearing resistance and SPT-N

trendline is 1.8 which is 90% to the value suggested by Chang and Broms (1991) and far from the reported value by Toh *et al*. (1989).

The relationship between unit shaft resistance and average SPT-N values was then divided into two categories (i.e., SPT-N less than 50 and SPT-N more than 50, limited to 150) as shown in Figures 18 and 19. It is observed that the value of K_{su} is approximately 8.7 for SPT-N value less than 50 and in a good agreement with the values suggested in previous literatures. However, a linear trendline from the considerably scatter of data in Figure 19 has resulted a K_{su} value of 1.7 for SPT-N value more than 50 and showing a trend of decreasing K_{su} value with increasing SPT-N values. This trend was also observed by Toh *et al*. (1989) but for SPT-N values more than 50.

Due to limited data of mobilised end bearing resistance, its relationship with SPT-N values is shown in Figure 20. The value of K_{bu} observed from the linear trendline is 25.2 compared with 30 to 45 as suggested by Chang and Broms (1991). However, this value is approximately 92% similar with findings from Toh *et al*. (1989) where the K_{bu} value is 27. As discussed above, the end bearing for Test Piles A and C were not fully mobilised.

Conclusions

Results from three instrumented test piles installed in the Kenny Hill formation have shown slightly different outcome from the previous literatures. General trend is observed where the unit shaft resistance increases with increasing SPT-N values but the value of K_{su} is found to be inversely proportional with increasing SPT-N values of more than 50. Test results suggest that f_{su} = 1.8N for bored pile socketed in weathered material with N≤150. For material is SPT-N values less than 50, the value of K_{su} is achievable with the value suggested by Chang and Broms (1991) and Toh *et al*. (1989), but not in a good agreement for SPT-N values more than 50. It is recommended that lower value of K_{su} should be considered for SPT-N more than 50.

The recommended value of K_{bu} from Toh *et al*. (1989) appears to be reasonable and comparable with the test results of the test piles. However, in view of limited data further validation is recommended in futures to determine the most optimum K_{bu} value.

Acknowledgement

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