Foundation challenges for high-rise at granite-limestone interface zone

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ABSTRACT

The foundation challenges for a high-rise development of up to 46-storeys high located at Granite-Limestone interface zone is described in this paper. The foundation system for the building comprises bored piles socketed into Granite bedrock. The geological conditions at the interface zone is very complicated where there are limestone floaters above the Granite bedrock. In addition, other types of rock such as breccia, pyrite and skarn are also encountered at the interface zone which further complicates the foundation design. Results of instrumented test piles carried out at site and construction challenges are discussed. The mobilised shaft friction ranging from 900-1350kPa and end bearing ranging from 1900-6750kPa for the granite rock matched well with the assumed design parameters after adjusting rock socket during construction based on site-obtained point load strength, $I_{s(50)}$ values.

Keywords: bored piles; granite-limestone interface; skin friction; end bearing; point load strength of rock

1 INTRODUCTION

A residential development of up to 46-storey high has been proposed at Kelana Jaya, Selangor, Malaysia which is situated at the boundary between Limestone and Granite Formation (Figure 1). The limestone formation is infamous for its erratic formation with floaters, irregular bedrock, etc. In addition to the complicated limestone formation, the difficulty to identify a clear boundary between Limestone and Granite formation further complicates the deep foundation design. This paper presents the foundation design challenges at the Limestone-Granite interface zone and the results of verification pile test.

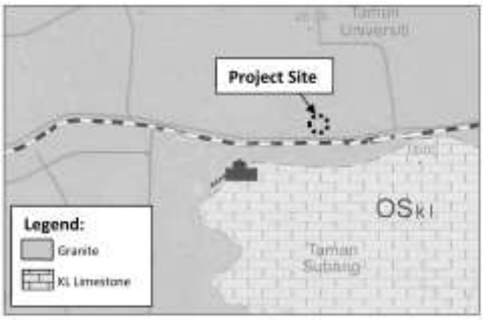

Fig. 1 Geological Map of Kuala Lumpur (New Series L7010 Sheet 94, Selangor Darul Ehsan, published in 2011).

2 SUBSURFACE INVESTIGATION (SI)

Detailed SI consisting of 161 numbers of boreholes were carried out for one of the high-rise towers. The extensive numbers of boreholes were proposed in collaboration with the developer’s in-house foundation contractor in order to mitigate uncertainties during construction and to obtain more reliable design and cost estimates prior to award of the foundation works. Boreholes were assigned to every column position within the tower footprint which was considered as critical zone and spread out within the podium area. Figure 2 shows summary of rock types encountered in each borehole while Figures 3 and 4 show some of the simplified borelogs for each zone. The overburden soil consists of loose silty sand to coarse sand with gravel. Granite bedrock was encountered at approximately 40-50m depth whilst some boreholes encountered intermittent limestone layer of up to 20m thick and also other types of rock such as breccia, pyrite and skarn. Intermediate soil layer with low SPT-N values is also detected between the Limestone floater and Granite bedrock.

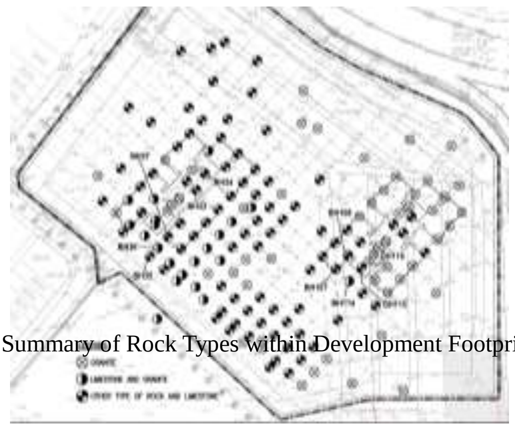

Fig. 2 Summary of Rock Types within Development Footprint

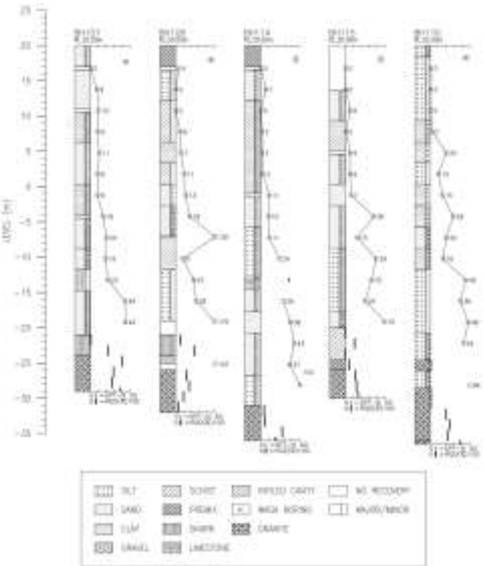

Fig. 3 Sample of Simplified Borelog for Granite Zone

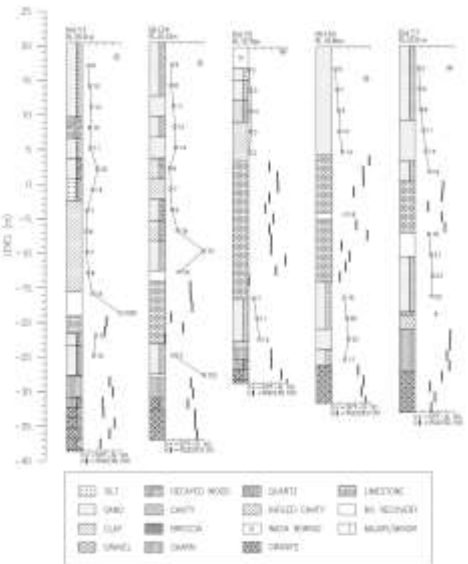

Fig. 4 Sample of Simplified Borelog at Limestone-Granite Zone

3 DESIGN OF FOUNDATION SYSTEM

3.1 Bored Pile Foundation Design

In view of the subsurface condition with large numbers of floaters which vary in thickness and nature, the piles were designed to terminate in the Granite bedrock as termination of pile toe in the limestone floater will result in excessive settlement. Therefore, bored pile foundation system was selected after considering the complex subsoil condition. The design parameters adopted are summarized in Table 1.

Table 1 Design Parameters Adopted for Bored Pile Design.

Descriptions	Design Parameters
Ultimate Shaft Friction in Soil (kN/m^2)	2× SPT-N Value
Average Ultimate Shaft Friction in Granite (kN/m^2)	1500
Ultimate End Bearing (kN/m^2)	4600 (15% of Pile Working Load)
Factor of Safety (FOS) for Geotechnical Pile Capacity	2.0

The bored pile diameters adopted for this development range from 750mm to 1800mm. All the bored piles were socketed into competent granite bedrock in accordance to the designed rock socket length based on average ultimate shaft friction of 1500kPa based on the Authors' local experience for bored piles in granite formation with minimum unconfined compressive strength of the rock of 40MPa. The competent bedrock was defined by the point load test (PLT) results (ASTM D 5731) on minimum three intact rock core samples retrieved during bored pile construction to achieve minimum index strength, $I_{s(50)}$ of 3MPa. In an event when the index strength, $I_{s(50)}$ of the rock samples is less than 3MPa, the equivalent rock socket length for every 1m of partial rock coring length is summarized in Table 2. The value of $I_{s(50)}$ of 3MPa is determined based on test results on rock core samples during SI where both compression tests and index strength tests were carried out from rock core samples retrieved from boreholes.

Table 2 Equivalent Rock Socket Length for Every 1m of Partial Rock Coring not Fulfilling $I_{s(50)}$ = 3MPa.

Average Values $I_{s(50)}$ per Layer (MPa)	Equivalent Rock Socket Length to $I_{s(50)}$ = 3MPa (m)
< 0.5	0
0.5 to < 1.0	0.20
1.0 to < 2.0	0.40
2.0 to < 3.0	0.60
≥ 3.0	1.00

4 INSTRUMENTED TEST PILE

4.1 Overview of Instrumented Test Piles

Two preliminary instrumented maintained load tests were carried out on 900mm (TP900) and 1800mm (TP1800) diameter bored piles prior to the construction of working piles in order to verify the adopted design parameters. The locations of the test piles were at the area without the limestone floater which is considered as worst-case scenario and consistent with the pile design concept. Kentledge blocks as reaction system was adopted for testing on 900mm diameter bored pile while bi-directional test was adopted for 1800mm diameter bored pile. The maximum test load adopted was three times of the pile working load. Borehole was carried out prior to the construction of test pile in order to obtain the subsoil stratification. Figures 5a – 5b show the locations of instruments for TP900 and TP1800 relative to the simplified borelog. The hydraulic jack for TP1800 was installed at the lower part of the bored pile. The estimated total shaft resistance above the hydraulic jack approximately 10% more than the shaft and base resistance below the hydraulic jack with the assumption that extra end-bearing can be mobilized during testing. The load movement behaviour of the pile undergoing three loading cycles are summarised in Table 3.

TP900 was terminated at 280% pile working load while TP1800 was terminated at 240% pile working load when excessive settlement was observed. Figure 6 shows the mobilized unit shaft friction along the pile body of TP900 while mobilized unit end bearing is shown in Figure 7. The mobilised unit shaft friction of TP1800 for segment above hydraulic jack and below hydraulic jack is shown in Figure 8a-8b while the mobilised unit end bearing of TP1800 is shown in Figure 9.

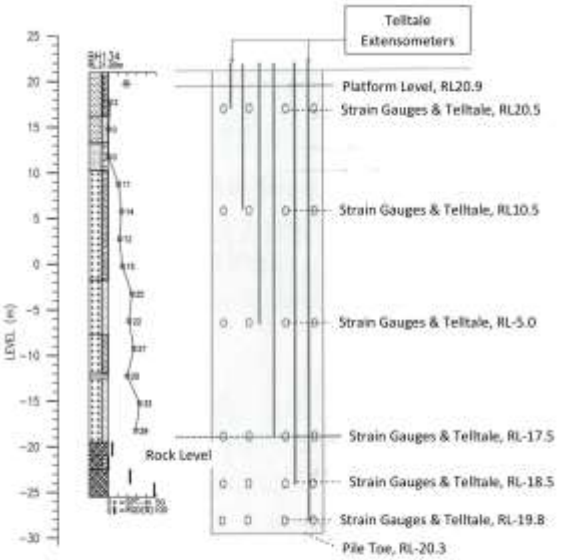

Fig. 5a Simplified borelog and Instrument Level for TP900.

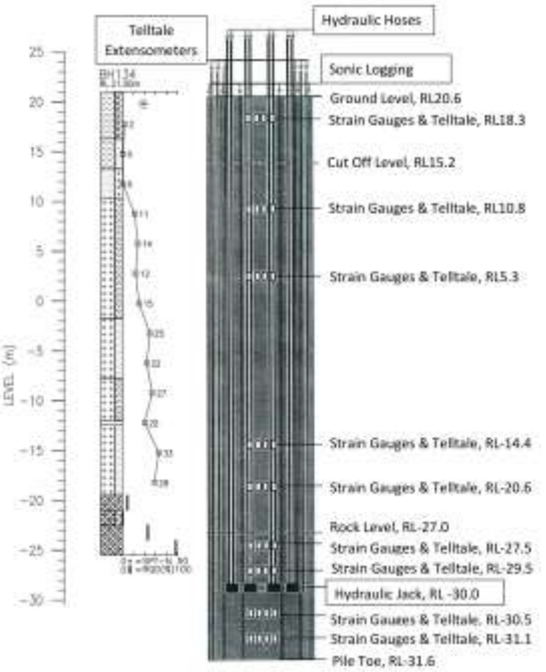

Fig. 5b Nearest Simplified Borelog and Instruments level for TP1800.

Table 3 Summary of Pile Displacement During Three (3) Loading Cycles for TP900 and TP1800.

TP900

Loading Cycle	Maximum Applied Test Load (tonnes)	Pile Top Displacement (mm)	Pile Residual Settlement (mm)
1	630	9.82	0.08
2	1,260	23.09	2.90
3	1,764	71.45	60.05

TP1800

Loading Cycle	Effective bi-directional load (tonnes)	Maximum Upward Movement of Jack (mm)	Maximum Downward Movement of Jack (mm)
1	2,540	5.2	4.5
2	5,080	12.5	11.4
3	6,066	16.5	82.8

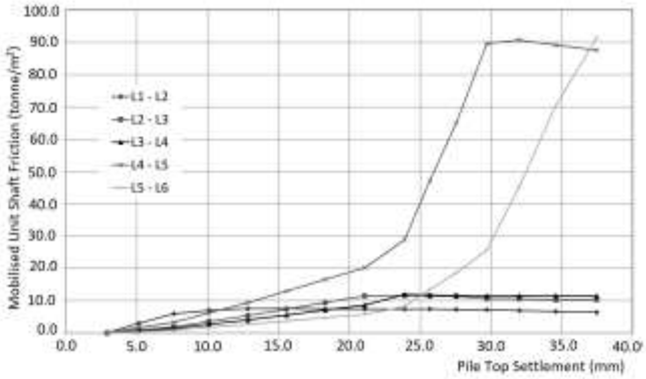

Fig. 6 Mobilised Unit Shaft Friction During 3rd Loading Cycle for TP900.

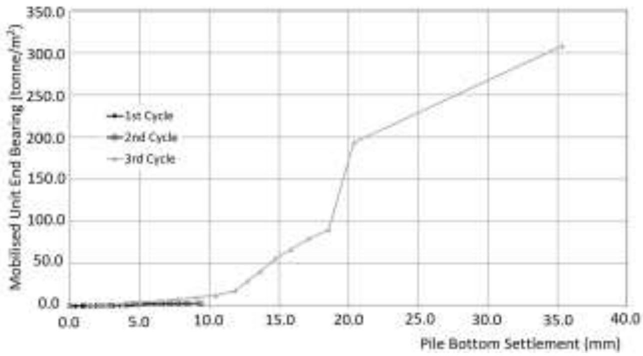

Figure 7 Mobilised Unit End Bearing During 1st, 2nd and 3rd Loading Cycle for TP900.

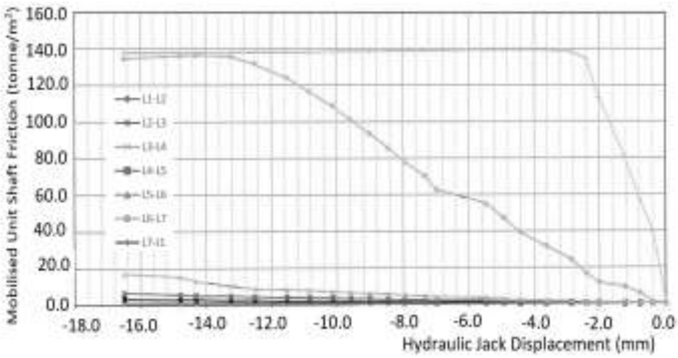

Figure 8a Mobilised Unit Shaft Friction Above Hydraulic Jack for TP1800.

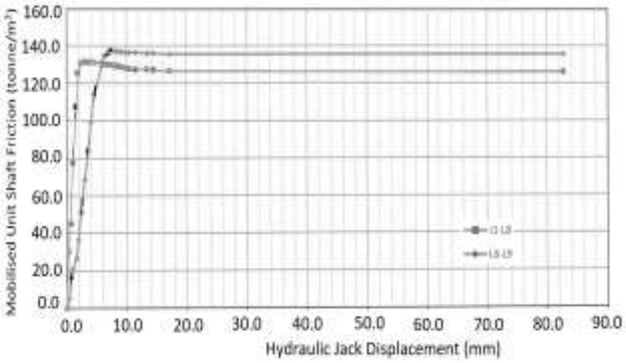

Figure 8b Mobilised Unit Shaft Friction for Segment Below Hydraulic Jack for TP1800.

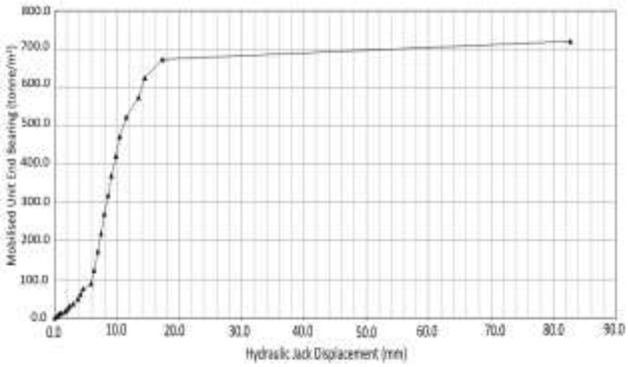

Figure 9 Mobilised Unit End Bearing for Segment Below Hydraulic Jack for TP1800.

4.2 Verification Test Results in Comparison to Design Assumptions

Table 4 shows the comparison between the design parameters with the instrumented pile test results. The ultimate shaft friction in soil for TP900 is higher than the design parameter by approximately 2.5 times where slight pile bulging is observed at the top portion of the test pile which may have contributed to the exceptionally high values of shaft friction. However, the tested average ultimate shaft friction and end bearing in Granite bedrock is lower than the designed parameters which is expected as the $I_{s(50)}$ values obtained at site is lower than 3MPa. Meanwhile, the test results of TP1800 indicated that the shaft friction in both soil and Granite bedrock were slightly lower than the designed shaft friction. However, the mobilised end bearing was higher than the designed end bearing. Overall, both the instrumented test piles results showed that the parameters assumed during design represented well the ground condition when adjusted based on site-obtained $I_{s(50)}$ values.

Table 4 Summary of Pile Test Results in Comparison to Design Parameters.

Descriptions	Design	TP900	TP1800
Ultimate Shaft Friction in Soil (kN/m^2)	2N	≈ 5N	≈ 1.5N
Average Ultimate Shaft Friction in Rock (kN/m^2)	1500	900	1350
Ultimate End Bearing (kN/m^2)	4600	1900	6750

The test results have proven the effectiveness of adjusting rock socket during construction based on $I_{s(50)}$ values as recommended by Tan & Chow, 2013. Comparison of tested rock socket length with proposed equivalent rock socket length based on site-obtained $I_{s(50)}$ values are summarised in Table 5. The average proposed equivalent rock socket length of TP900 is 0.7m which is similar to the actual tested equivalent rock socket length of 0.6m for TP900. The actual equivalent rock socket length of TP1800 is relatively close to the proposed equivalent rock socket length based on site-obtained $I_{s(50)}$ values. Therefore, the designed rock socket length is satisfactory based on the results of point load test on the recovered rock during bored piling works.

Table 5 Equivalent Rock Socket Length.

Test Pile Results					Calc. Equiv. Socket Length (m)
Test Pile	Layer	Is (50), MPa	$f_{s(ult)}$, MPa	Actual Equiv. Socket Length (m)	
TP900 (270% WL)	4-5	1.418	862	0.57	0.47
	5-6	3.517	932	0.62	1.0
TP1800 (230% WL)	6-7	6.430	1360	0.91	1.0
	7-8	8.474	1321	0.88	1.0
	8-9	4.415	1358	0.91	1.0

5 CONCLUSION

At the KL Limestone-Granite interface, limestone floaters with different length and nature were encountered while the Granite bedrock was encountered at 40m-50m depth. Bored pile foundation system was selected where the pile was designed to socket in Granite bedrock with the design ultimate shaft friction of 1500kPa and ultimate end bearing of 4600kPa (15% of Pile Working Load) for rocks recovered during bored piling works with $I_{s(50)} \geq$ 3MPa. The results of two instrumented pile tests matched well with the assumed design parameters and also proven the effectiveness of adjusting rock socket during construction based on site-obtained $I_{s(50)}$ values.

REFERENCES

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