Hand-Dug Caisson Piles in Granitic Formation, Penang

P.T.LEE[1], K.L.KWANG[2], KHAIRUL IZHAR BIN KASIM[3] and AMZAR BIN AZHAR[4]
[1,2,3 and 4]G&P Geotechnics Sdn. Bhd., Kuala Lumpur, Malaysia.
E-mail: ptlee@gnpgroup.com.my, klkwang@gnpgroup.com.my, khairulizhar@gnpgroup.com.my, amzargnpgeo@gmail.com

ABSTRACT: This paper presents the design and construction of caisson pile in a hill site development. Caissons are generally hand dug and adopted at hill site development due to platform accessibility for piling rigs, limited working area and space constraints. Basically, the base of caisson contributes significantly to the pile capacity compared to the conventional bored pile system as the caisson base can be cleaned and inspected properly to ensure firm contact between concrete and the toe of the pile. In view of space contraints and hilly condition, normal maintained load tests using kentledges or reaction piles are not suitable to be carried out. This paper discusses alternative tests such as shaft load test and plate bearing test that are carried out inside the caisson to verify the designed shaft and base resistances respectively. The test results of the shaft load test and plate bearing test are also presented in this paper.

KEYWORDS: hill site development, deep foundation, caisson pile, shaft load and plate bearing tests

1. INTRODUCTION

The proposed development is located at Teluk Kumbar, Penang in hilly condition with existing ground levels ranging from about RL20m to RL80m and this site is underlain by Batu Maung Granite.

As the site is located at hilly terrain, hand dug caisson is proposed as foundation system in view of difficulties in platform accessibility for piling rigs. Generally, base resistance contributes significantly to caisson pile capacity as compared to bored pile. This is due to the construction method of caisson which allows proper inspection of rock socket and base cleaning works to ensure firm contact between concrete and the toe of the pile prior to the concreting works. Rock quality for the rock socket length could be inspected and ensure the caissons are socketed in the intact bedrock.

Maintained load tests (MLT) using kentledge system or reaction piles system are commonly used in the construction industry to verify the pile performance. Generally, these systems need to utilise big area and stable platform in order to carry out the tests. However, in view of space contraints and hilly condition, these tests are not suitable to be carried in this project site. Alternatively, shaft load test and plate bearing test are carried out inside the caisson to verify the designed shaft and base resistance respectively. Setting up of the above mentioned tests and the testing results are also be discussed.

2. GENERAL GEOLOGY AND SUBSURFACE INFORMATION

The site is underlain by Batu Maung Granite. The aged of Batu Maung Granite is Early Permian to Late Carboniferous. It mainly consists of medium to coarse-grained biotite-muscovite granite. The overburden materials consist mainly of completely weathered residual soils. The geological map of the site is shown in Figure 1.

A total of 25 boreholes are carried out at the site to obtain the subsurface stratification, groundwater regime and necessary engineering parameters for geotechnical study and design. Generally, the subsoil mainly consists of silty SAND and sandy SILT. Boulders are encountered at most of the boreholes as a significant feature on Granitic formation.

3. CAISSONS – DESIGN

The geotechnical capacity of the caisson pile is derived from both shaft friction resistance and base resistance. As the construction method of caisson allows proper inspection of rock socket and base cleaning works to ensure firm contact between concrete and the toe of the pile prior to the concreting works, base resistance constribute significantly to the caisson pile capacity.

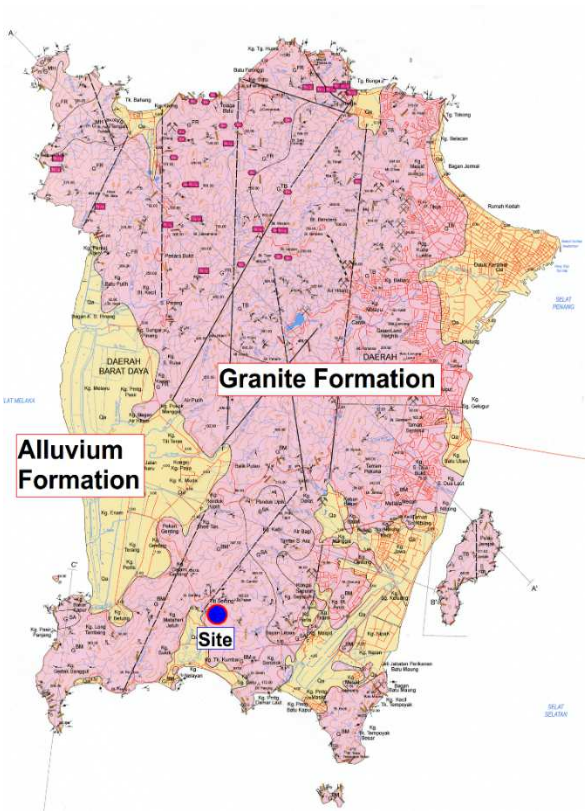

Figure 1 General Geological Map

3.1 Design Parameter for Shaft Friction Resistance

Based on design approach published by Rosenberg & Journeaux (1976), Horvath (1978) and Williams & Pells (1981), the ultimate rock shaft friction resistance with consideration of the respective strengths of intact rock and rock mass effect in association with the inherent discontinuities can be estimated with Equation (1).

$$fs_{(ult)} = \alpha.\beta.q_{uc} \quad (1)$$

where $fs_{(ult)}$ = Ultimate rock shaft resistance
α = Reduction factor with respect to q_{uc} (Figure 2)
β = Reduction factor with respect to the rock mass effect (Figure 3)
q_{uc} = Unconfined compressive strength of intact rock (Figure 4)

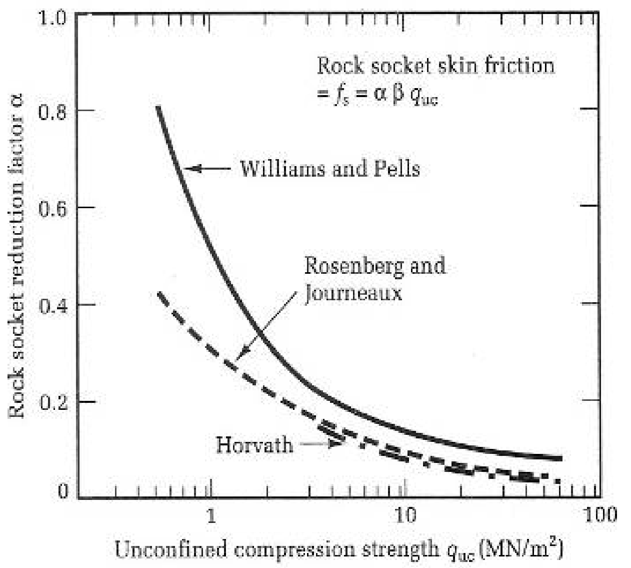

Figure 2 Reduction Factor with Respect to q_{uc}

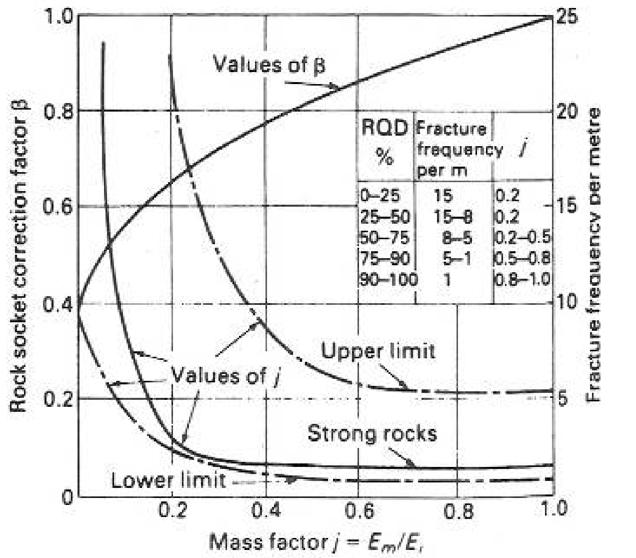

Figure 3 Reduction Factor with Respect to the Rock Mass

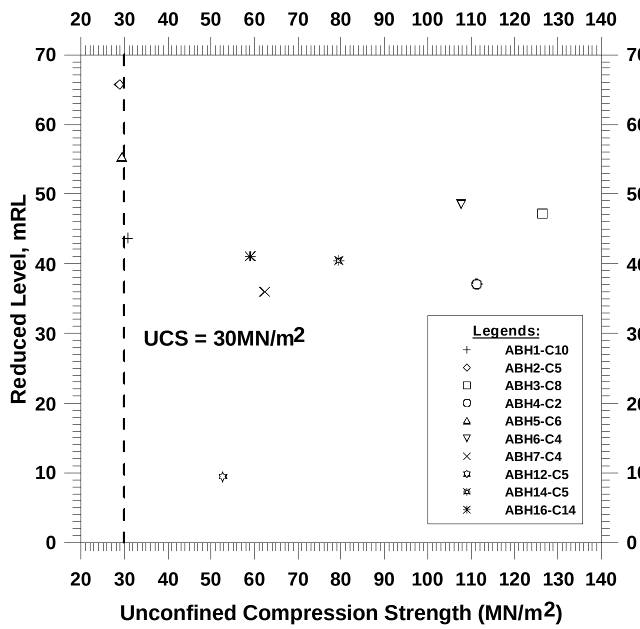

Figure 4 Unconfined Compression Strength of Rock

Based on the approach proposed by Willams & Pells (1981), with reference to the unconfined compressive strength of 30MPa (lower bound) and the Rock Quality Designation (RQD) of 25% to 50%, ultimate rock shaft resistance of 2,000kPa is adopted in the caisson pile design for this project site.

Generally, the shaft friction resistance in the soil is ignored in the pile capacity design in view of uncertainties in the quality of the concrete lining. However, as presented by Yee (2000), testings were succefully conducted and the tests proved that the caisson lining could transfer load onto the soil. It is reported that correlation of 2N can be adopted to determine the ultimate shaft friction resistance for soil contacted with the concrete lining.

3.2 Design Parameter for Base Resistance

Unlike conventional bored pile which usually ignoring the base resistance due to unsatisfactory base cleaning works, base resistance contributes significantly to the pile capacity in the caisson pile desing caisson. This is because under dry hole construction, the caisson pile can easily achieve satisfactory base cleaning to ensure the proper contact between concrete and the rock base.

The allowable base resistance for rock is estimated from the empirical correlation considering the spacing of discontinuities of bedrock as shown in Equation (2), as recommended by Canadian Foundation Engineering Manual (1992).

$$fb_{(all)} = K_{sp}.q_{uc} \quad (2)$$

where $fb_{(all)}$ = Allowable rock base resistance
K_{sp} = Coefficient based on spacing of discontinuities (Table 1)
q_{uc} = Unconfined compressive strength of intact rock

Table 1 Coefficients of Discontinuity Spacing (Canadian Foundation Engineering Manual, 1992)

Condition of Discontinuities	Spacing of Discontinuities (m)	K_{sp}
Moderately close	0.3 – 1	0.1
Wide	1 – 3	0.25
Very wide	> 3	0.4

Based on the above approach, the allowable rock base resistance could be estimated as 3,000kPa. However, lower allowable rock base resistance of 2,500kPa is adopted as normally larger base movement is required to mobilise the base resistance.

4. CAISSONS – PILE TESTING

In view of space constraint and hilly terrain, shaft load test and plate bearing test are adopted at this project site as alterantive options to the conventional maintained load test using kentledges or reaction piles system (which normally require large and stable working platform) to verify the shaft friction resistance and base resistance respectively.

4.1 Shaft Load Test

The schematic diagram for typical setup of shaft load test is shown in Figure 5.

A test concrete lining with length of 500mm is cast surrounding the rock socket perimeter. This concrete lining is cast for the testing purpose only. During construction of caisson, there is no concrete lining within the rock socket length. Prior to the test, a concrete slab is also cast at the base of caisson. Sufficient number of hydraulic jacks are placed between the test lining and the concrete slab. Then, the jacks will be jacked against the concrete slab / rock base in order to verify the shaft friction resistance between the test lining and rock. The test is carried out in two loading cycles. Photos for actual setup are shown in Figures 6 – 9.

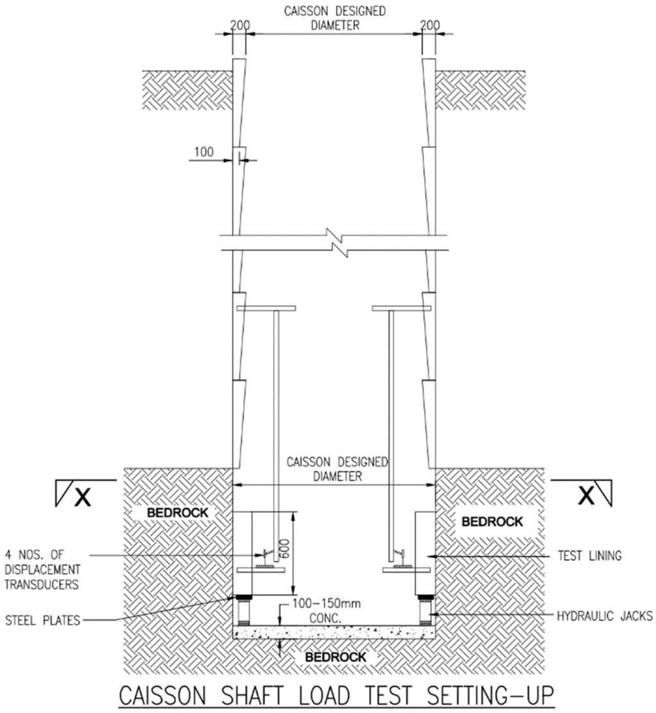

Figure 5 Schematic Diagram for Typical Setup of Shaft Load Test

Figure 6 Test Lining at Rock

Figure 7 Hydraulic Jacks

Figure 8 Allowable Gap on top of Test Lining and Independent reference beam

Figure 9 Displacement Measurement Device

The loading schedule is shown in Table 2.

Table 2 Loading Schedule for Shaft Load Test

Load Increment	Holding Time (minutes)
Staged Incremental of 12.5% until 100% DTL	10
100% DTL	60
Staged Decremental of 25% until 0% DTL	10
Completion of First Cycle	
Staged Incremental of 25% until 200% DTL	10
200% DTL	60
Staged Decremental of 50% until 0% DTL	10
Completion of Second Cycle	

The details of two shaft load tests on rock are as follows:

Table 3 Details of Shaft Load Test

Pile Diameter	Pile Ref.	1st Cycle	2nd Cycle
1,200mm	MC47	1,750kN	3,500kN
1,500mm	LMC74	2,250kN	4,500kN

4.1.1 Results of Shaft Load Test

Figure 10 shows the two shaft load test results. The tests are carried out against the rock surface to verify the designed rock shaft resistance. It can be observed that about 1mm to 3mm displacement are recorded for the first cycle and about 2mm to 7mm displacement measured for the second cycle. The residual settlement after the first cycle is relatively small, which is about 1mm only. The load-settlement curve for MC47 show relative stiffer behaviour compare to the load-settlement curve for LMC74. However, both tests show that the designed ultimate rock shaft resistance of 2,000kPa is achieveable.

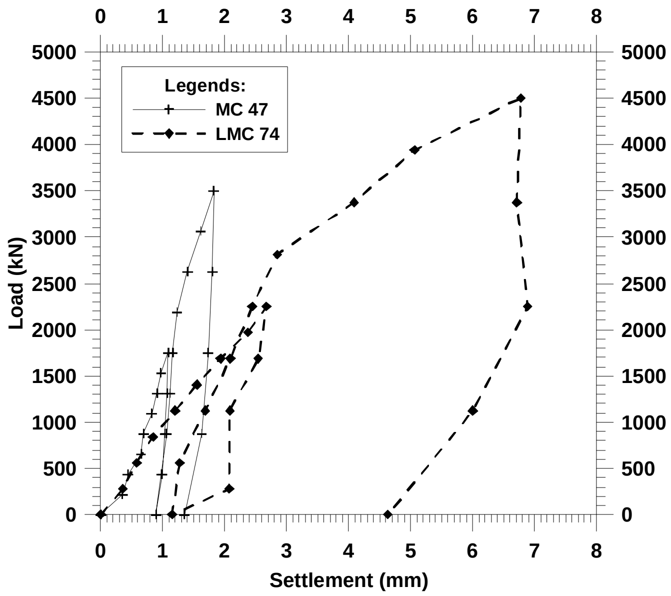

Figure 10 Shaft Load Test Results

4.2 Plate Bearing Test

The schematic diagram for typical setup of plate bearing test is shown in Figure 11.

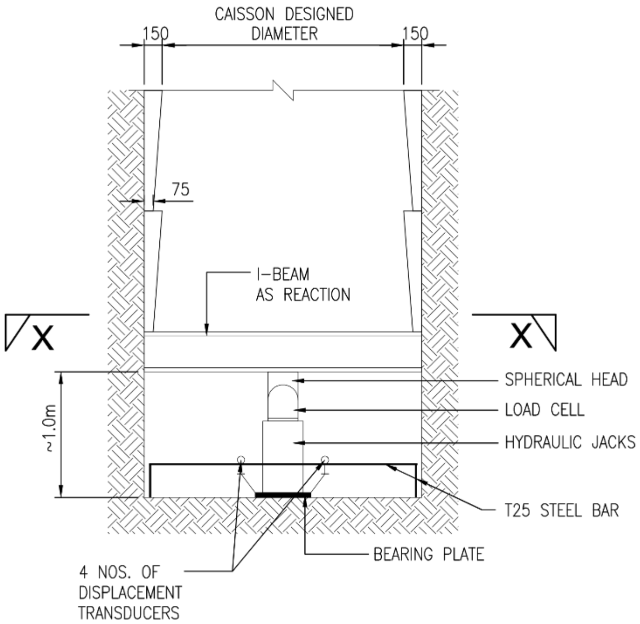

Figure 11 Schematic Diagram for Typical Setup of Plate Bearing Test

A certain size of bearing plate is used in this testing and the test load will be determined based on the size of the bearing plate. 300mm diameter bearing plate is used in this project site. Concrete lining at top will act as a reaction system during the testing. Besides that, a layer of lean concrete is applied at the rock surface at pile toe to ensure the surface is even before placing the bearing plate. The test is carried out in three loading cycles. Photos for actual setup are shown in Figures 12 – 13.

Figure 12 Setup of Plate Bearing Test

Figure 13 300mm Diameter Bearing Plate

The loading schedule is shown in Table 4.

Table 4 Loading Schedule for Plate Bearing Test

Load Increment	Holding Time (minutes)
Staged Incremental of 12.5% until 100% DTL	10
100% DTL	60
Staged Decremental of 25% until 0% DTL	10
Completion of First Cycle	
Staged Incremental of 25% until 200% DTL	10
200% DTL	60
Staged Decremental of 50% until 0% DTL	10
Completion of Second Cycle	
Staged Incremental of 25% until 300% DTL	10
300% DTL	60
Staged Decremental of 100% until 0% DTL	10
Completion of Third Cycle	

The details of plate bearing load tests on rock are as follows:

Table 5 Details of Plate Bearing Test

Pile Base	1st Cycle	2nd Cycle	3rd Cycle	Pile Ref.
Soil	95kN	190kN	285kN	MC2
Rock	180kN	360kN	540kN	MC25, MC47, LMC64, LMC74

For this project, plate bearing tests are conducted for soil and rock base. Most of the caissons are rock-socketted piles, except piles that are terminated at hard soil layer earlier due to unforeseen groundwater issue. Hence, plate bearing test is conducted at the soil base to further verify the designated soil base resistance of 4,000kPa.

4.2.1 Results of Plate Bearing Test

Based on Figures 14 to 16 which shows the plate bearing test results on soil and rock respectively, it can be observed that about 1mm settlement is measured for the first cycle and about 1mm to 2mm displacement is recorded for the third cycle.

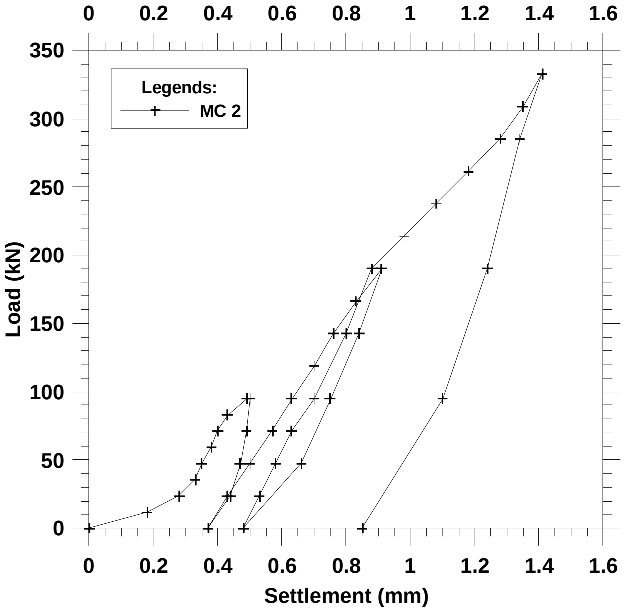

Figure 14 Plate Bearing Test Result on Soil Base

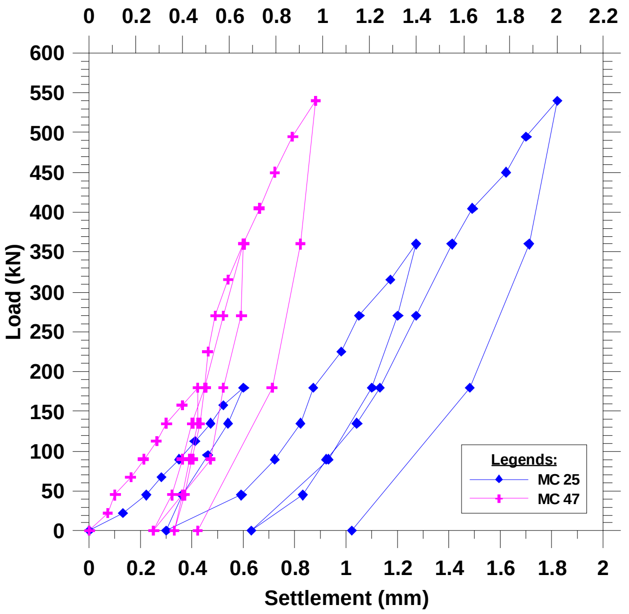

Figure 15 Plate Bearing Test Result on Rock Base

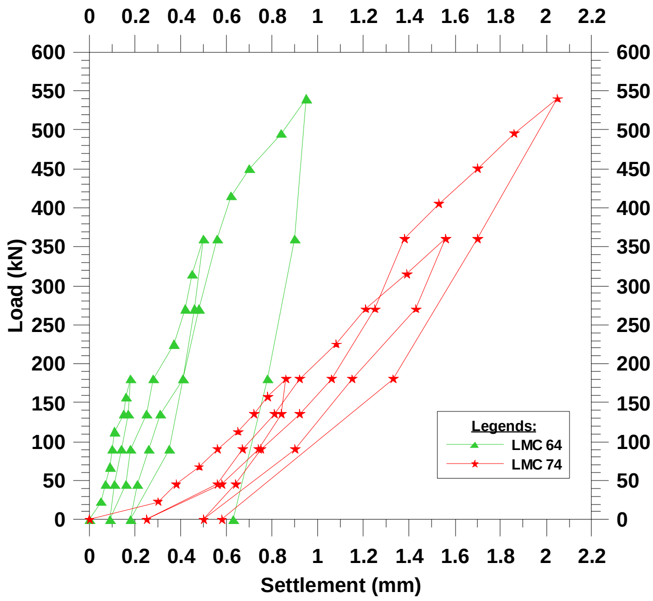

Figure 16 Plate Bearing Test Result on Rock Base

The estimated settlement for actual pile size is calculated from the empirical correlaction proposed by Terzaghi and Peck (1948), as shown in Equation (3).

$$\rho_f = \rho_B\,(2B_f / B_f + B_B)^2 \qquad (3)$$

where P_f = Settlement of foundation
ρ_B = Settlement of bearing plate
B_f = Foundation size
B_B = Plate bearing size

Table 6 Estimated Pile Base Settlement

Pile Ref.	Measured Settlement from Plate Bearing Test	Estimated Settlement of Pile Base
MC2	0.50mm	1.28mm
MC25	0.60mm	1.54mm
MC47	0.42mm	1.08mm
LMC64	0.18mm	0.46mm
LMC74	0.86mm	2.39mm

Figures 17 and 18 show the base condition of the plate bearing tests. Generally, the plate bearing test results yield that the adopted ultimate base resistances of 4,000kPa and 7,500kPa are achieveable for soil and rock respectively.

Figure 17 Soil Condition at Pile Base

Figure 18 Rock Condition at Pile Base

5. CAISSONS – CONSTRUCTION

5.1 Rock Quality Inspection in Caissons

Confirmation of rock socket into competent bedrock (Grade I and II) are carried out by experienced engineering geologist inside the caisson holes after the caissons reached the founding rock stratum. This is to identify the weathering grade of rock and existence of any prominent joints or fractures which will potentially affecting the pile performance and subsequently superstructure performance.

The inpection works are conducted by adopting visual inspection and geological hammer to further verify the rock quality and weathering grade and thus to confirm the rock socket. For visual inspection part, rock quality can be classified based on colour, joints, mass and fracture filling as per classification of rock outlined in Table 7. Whilst, for geological hammer, the rock characteristics could be defined by the ringing sound and breakability.

Table 7 Classification of Rock Material Decomposition Grades Extracted from Geoguide 3: Guide to Rock and Soil Description, Hong Kong, 1988

Grade		Rock Characteristics for Granitic Rock
VI	Residual soil	Original rock texture complety destroyed. Can be crumbled by hand and finger pressure into constituent grains. Reddish brown.
V	Completely decomposed	Original rock texture preserved. Can be crumbled by hand and finger pressure into constituent grains. Easily indented by point of geological pick. Slakes when immersed in water. Completely discoloured compared with fresh rock. Yellowish brown to reddish brown.
IV	Highly decomposed	Can be broken by hand into smaller pieces. Makes a dull sound when stuck by geological pick. Does not slake when immersed in water. Completely discoloured compared with fresh rock. Yellowish brown to yellowish orange/brown.
III	Moderately decomposed	Cannot usually be broken by hand; easily broken by geological hammer. Makes a dull or slight ringing sound when stuck by geological hammer. Completely stained throughout. Yellowish brown.
II	Slightly decomposed	Not broken easily by geological hammer. Makes a ringing sound when stuck by geological hammer. Fresh rock colours generally retained but stained near joint surfaces.
I	Fresh	Not broken easily by geological hammer. Makes a ringing sound when stuck by geological hammer. No visible signs of decomposition (i.e. no discolouration). Overall rock colour grey/white.

Next, 360° rock mapping logging is used during the construction works to closely document the rock socket condition. Sample of rock mapping logging and photos are shown in Figures 19 and 20.

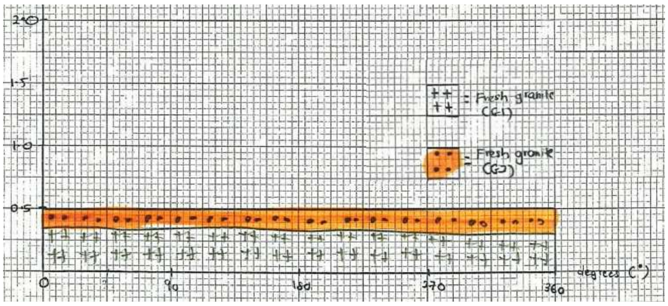

Figure 19 Rock Mapping Logging

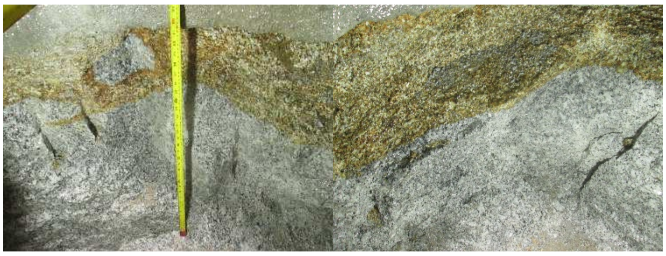

Figure 20 Rock Socket Photos for Rock Map Logging in Figure 19

6. CONCLUSION

Shaft load test and plate bearing test are alternatively utilised to replace conventional maintained load test for caissons. These pile tests could verify the design parameters of shaft resistance and base resistance independently. However, these tests are unable to verify the whole pile performance.

From the shaft load tests and plate bearing tests, ultimate rock shaft friction of 2,000kPa and ultimate base resistance of 4,000kPa and 7,500kPa for soil and rock respectively can be considered to be adopted in caisson pile design in granitic formation. The achieved design parameters are similar to the designed parameters reported by Liew and Lee (2011).

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