# Pile Performance in Weathered Meta-Sedimentary Formation and KL Limestone


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ABSTRACT: This paper presents the performance of three preliminary instrumented bored cast-in-situ test piles socketted in weathered meta-sedimentary formation and limestone formation in Kuala Lumpur, Malaysia. The test piles were instrumented with Global Strain Extensometer technology and were load tested under quick maintained load test with two piles under compressive axial loading and one under tensile loading. For the compression bored pile embedded in soil by wet hole construction, about 8.7% of the test loads was transferred down to the pile base, whereas the short rock socketted pile constructed with soft toe in wet hole condition has about 38% of the test loads transferred to rock socket. Unconfined compressive strength tests and point load tests on both the collected rock cores during boreholes exploration and the rock fragments recovered during pile construction were correlated with mobilised rock socket resistances. However, the correlations are scattered and do not have a clear trend.


## 1. INTRODUCTION

Three test piles presented in this paper are all bored cast-in-situ piles of 1500mm and 1800mm diameter for compression piles and 1200mm diameter for tension pile. For the two compression load tests, a total of 2494 nos. concrete blocks have been stacked-up for maximum test load of 4560 Tons with the height of approximately 20m above ground level. Two rows of 32 temporary steel pipe piles were driven to support the dead weight of the kentledge system and provide stable platform for pile testing (Fig. 1).

The required reaction load for tension test pile was provided by four reaction piles of 750mm diameter installed to 21m below testing platform. The reaction piles were installed at a distance of approximately 3.5 times test pile diameter away from the centre of the test pile. The maximum test load of 1120 Tons was applied to the tension pile using four 1000 Tons capacity hydraulic jacks.

This paper is aimed to present the test results of these three preliminary instrumented bored piles installed in the weathered meta-sedimentary formation underlain by Kuala Lumpur Limestone. The results from these test piles provide some understanding of the pile behaviour in terms of shaft friction mobilisation, end bearing mobilisation and load distribution for the similar geological materials with similar pile construction practice and use of bentonite as stabilising medium for the test piles construction.

Fig. 1 4560 Tons Maintained Static Pile Load Test & 1120 Tons Tension Pile Load Test

## 2. GEOLOGICAL FORMATION

The subsurface investigation revealed that the site is located at the geological contact of two formations, namely weathered meta-sedimentary formation, locally known as Kenny Hill formation, overlying Kuala Lumpur Limestone formation. The thickness of Kenny Hill formation at this site ranges from 15m to more than 60m. It consists of mostly highly decomposed Grade IV metamorphosed sandstone/shale and completely weathered Grade V residual soils. The overburden alluvial materials generally consist of silty or gravelly SAND.

## 3. PILE INSTALLATION AND INSTRUMENTATION

Figs. 2 to 4 show the nearest borelogs together with the details of three instrumented Test Piles A, B and C. Test Pile A has a diameter of 1500mm and was tested with excessive pile top settlement. Test Pile B is 1200mm diameter tension pile having a penetration depth of 24m from piling platform level. The test piles were embedded within the upper top layer of alluvium material of sandy soils and the lower weathered meta-sedimentary materials consisting of very stiff sandy silt with SPT-N more than 50. Test Pile C of 1800mm diameter has a penetration depth of 31m from piling platform level with 4m rock socket length into limestone of Rock Quality Designation (RQD) from 20% to 85%. A "polystyrene foam soft toe" was installed at the pile base for transferring load to rock socket with minimum load interference from pile base.

All test piles were formed by auger drilling through the overburden soils with bentonite slurry for hole stability and concrete casting using tremie method. The load tests were performed after the piles had achieved minimum 28 days of designed concrete strength. Temporary steel casing were driven to prevent borehole collapse for top 7m to 14m of loose alluvial soils. Drilling speeds in the installation of test piles are summarised in Table 1.

The test piles were instrumented with proprietary Global Strain Extensometer technology (Glostrext method) using the access to the pile shaft from the sonic logging tubes as presented by Hanifah & Lee (2006). This system uses advanced retrievable pneumatically-anchored extensometers coupled with high-precision spring-loaded vibrating-wire sensor with a simple analytical technique to monitor loads transferred down the shaft and the toe of test piles. It is a post-installation instrument which can accurately measure the relative deformations of anchored segments across the entire pile length.

Table 1 Summary of Pile Installation Records

| Test Pile Ref. | A | B | C |
|---|---|---|---|
| Pile Diameter | 1500mm | 1200mm | 1800mm |
| Drilling System | BG22/31 | CMV | BG45 |
| Stabilising Fluid | Bentonite | Bentonite | Bentonite |
| Temporary Casing Length | 7m | 7m | 14m |
| Drilling Speed (m/hr) in Respective to Geological Formations | | | |
| Kenny Hill Formation | 3.7m/hr | 6.7m/hr | 9.3m/hr |
| Limestone Formation | N.A. | N.A. | 0.5m/hr |
| Duration of Drilling | 1 day | 1 day | 1.5 days |
| Concrete Casting Method | Tremie 2 | Tremie 2 | Tremie 2 |

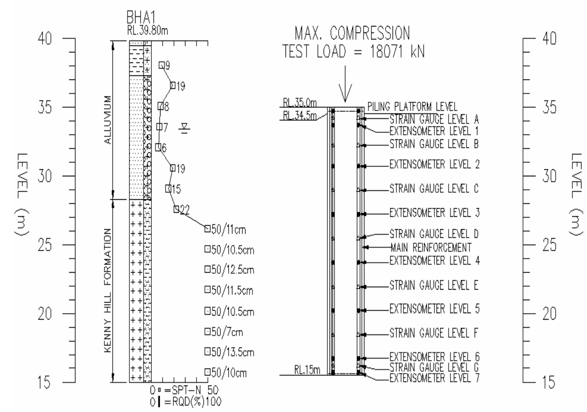

Fig. 2 Test Pile A (ϕ1500mm)

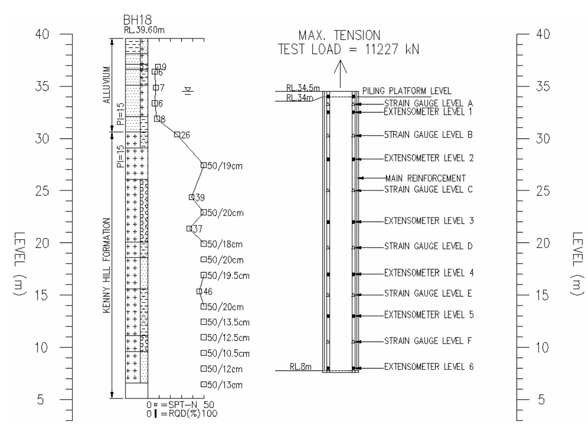

Fig. 3 Test Pile B (ϕ1200mm)

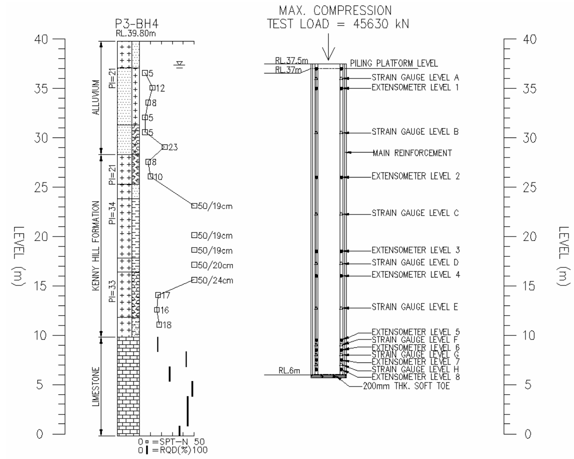

Fig. 4 Test Pile C (ϕ1800mm)

## 4. INTERPRETATION OF TEST PILE RESULTS

### *4.1. Test Pile A – Compression Pile*

Figs. 5 and 6 show the load displacement curve and mobilised unit shaft friction for Test Pile A. The maximum top pile settlement at the load of 10100kN at the first cycle was 12.04mm. In second cycle, excessive movement of 54.77mm was observed at the test load of 18071kN and evidenced that the pile has almost fully mobilised attaining the ultimate shaft friction.

The interpreted pile test results show strain softening at the upper alluvial soil layers between ground level and Level B but not at the lower alluvial soils. The large displacement of pile shaft of about 11mm has led to reduction of shaft friction at the upper portion of pile. Strain softening of pile-soil interface may affect the distribution of skin friction down to the pile toe and cause the reduction in shear transfer between the soil and pile interface, from peak value to residual value.

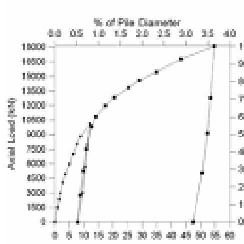

Fig. 5 Load Displacement for Test Pile A

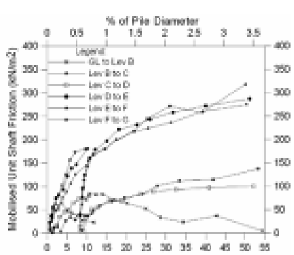

Fig. 6 Mobilised Unit Shaft Frictions for Test Pile A


Fig. 7 Double Zinc Sheets wrapped around Test Pile A

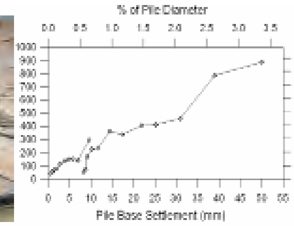

Fig. 8 Mobilised Unit End Bearing for Test Pile A

Fig. 7 shows the double-layer zinc sheets with lubricant grease in-between wrapped around the pile perimeter at the last 1.2m from the pile toe of Test Pile A for base resistance measurement by eliminating the influence of the pile shaft friction.

The high pile base movement as shown in Fig. 8 has led to gradual mobilisation of the end-bearing but without attaining failure. Pile base resistance attained 890kN/m$^2$ or 8.7% of the total test load with settlement of 50mm at the pile base implies possible existence of soft toe by deposition of bentonite cake or base softening as the test pile was constructed with only normal base cleaning procedures.

### *4.2. Test Pile B – Tension Pile*

Test Pile B is a tension pile reinforced with 26 nos. 40mm diameter high yield steel bars for taking the tensile test load. The load displacement curve and mobilised unit shaft friction are shown in Figs. 9 and 10. In first cycle, the pile top displacement at tension load of 4500kN was 4.02mm. For second and third cycles, the maximum displacements are 17.15mm and 28.98mm at tension load of 9253kN and 11227kN respectively.

Although Test Piles A and B were located at the same meta-sedimentary formation, the test results are varied significantly. Performance in term of shaft friction for Test Pile B was less satisfactory as compared to Test Pile A. Ultimate mobilised shaft friction for the pile shaft was also scattered. Maximum ultimate shaft friction interpreted for test pile was 180kN/m$^2$ with the displacement of 15mm. The average ultimate shaft friction was about 120kN/m$^2$. The reduction in shaft resistance could be attributed to the pile radial shrinkage from the Poisson ratio effect.

Similar to Test Pile A, strain softening at upper portion of alluvial soils was observed from the test results and there is relatively large displacement at the upper soil. Subsequent strain softening was observed at Level B to C and Level D to E. The result shows the tendency of progressive failure of the pile. Low pile axial stiffness was observed under tensile loading as most of the tensile test load was taken by the steel reinforcement at higher axial strain, probably cracking the concrete. Sonic logging results showed signal delay in first arrival time and decrease of signal energy during tension load test at top 15m of test pile as compared to the results before testing, which further confirm tensile cracking of concrete at the pile body.

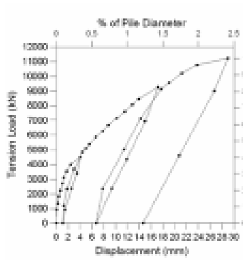


Fig. 9 Load Displacement for Test Pile B

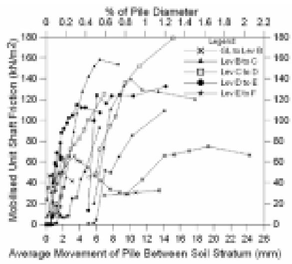


Fig. 10 Mobilised Unit Shaft Frictions for Test Pile B

### *4.3. Test Pile C – Compression Pile*

Fig. 11 shows the load displacement curve for Test Pile C which is a rock socketted pile with an artificial 'soft toe'. A typical soft slump zone layer was observed at 4m above the limestone bedrock interface, probably due to the natural leaching process and migration of fine soils from upper overburden soils into the lower relatively porous limestone formation.

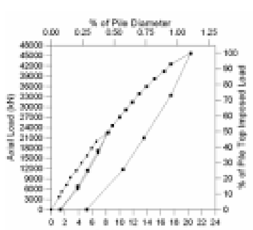


Fig. 11 Load Displacement for Test Pile C

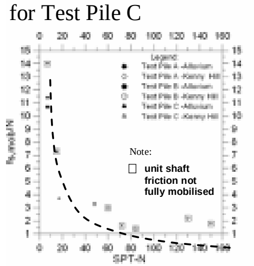


Fig. 13 $f_{s,mob}$/N ratio vs. SPT-N

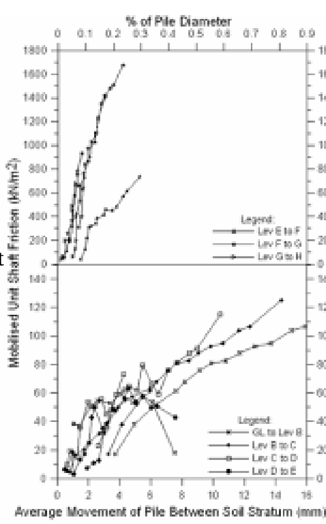


Fig. 12 Mobilised Unit Shaft Frictions for Test Pile C

The maximum pile top settlement at the first cycle test load of 22400kN was 8.17mm. In second cycle, the movement of 20.39mm was observed at the maximum test load of 45630kN. The 'soft toe' was seen to be performing well as noted from the low strain readings implying minimum load transferred to the pile toe, in which only about 5% of the total test load reached the pile base.

From the results of mobilised pile shaft friction, about 38% of the applied test load was transferred down in the form of rock socket friction. Fig. 12 shows that the maximum mobilised socket friction was 1670kN/$m^2$ between Level F and Level G where the RQD value of limestone is about 20%, whereas the maximum mobilised socket friction was 1400kN/$m^2$ for lower rock socket with RQD value of 85% from Level G to Level H. The relatively large displacement at the soil layer as compared to the rock socket length suggests that probably a slip along the pile shaft might have occurred during the load test. When the slip occurred, soil friction along pile shaft will most probably maintain unchanged if not reduced slightly due to strain softening effect and the subsequent imposed test load will be transferred to the rock socket friction, which tally well with the measured test results as demonstrated in this case.

Lower value of ultimate mobilised shaft friction of 63kPa was observed at the slump zone layer with average SPT-N of 17 blow counts (RL10m to RL16m) as shown in Fig. 12. This result reveals that more careful assessment is required during design and construction of bored pile foundation in limestone formation with evidenced existence of slump zone.

### *4.4. Summary*

A summary of mobilised pile shaft frictions and critical movements at pile-to-soil interface for every soil strata is tabulated in Table 2. From the test results, a similar trend of skin friction is observed for all the test piles. The ratio of mobilised unit skin friction, $f_{s,mob}$ to average SPT-N reduces exponentially with the increasing depth as well as with increasing SPT-N values as shown in Fig. 13. All the instrumentation results clearly show higher $f_{s,mob}$/N ratio for alluvium soil with lower average SPT-N values as compared to Kenny Hill formation with higher SPT-N values. This may probably due to the installation of temporary steel casing to prevent collapse of loose sandy material at upper alluvial soil layer, in which the actions of vibro-installation may provide sufficient compaction effort in densifying the upper loose alluvial soils. In weathered Kenny Hill formation of stiff to very hard soil consistency, the $f_{s,mob}$/N ratio for shaft friction under compressive load is fairly consistent, in which the ratio ranges between 1.5 and 2.2.

Table 2 Summary of Mobilised Unit Shaft Frictions and Critical Movements at Pile-to-Soil Interface

| Test Pile | Soil Stratum | $\bar{N}$ | $\frac{f_{s,mob}}{\bar{N}}$ | $\frac{f_{b,mob}}{\bar{N}}$ | Critical Movement, $Z_c$ (mm) (%)* | Remarks |
|---|---|---|---|---|---|---|
| A (c) | Alluvium | 7 | 11.4 | - | 4 (0.3%) | # |
| | | 15 | 7.3 | - | 6 (0.4%) | |
| | Kenny Hill | 130 | 2.2 | - | 12 (0.8%) | |
| | | 150 | 1.8 | 5.3 | 6.5 (0.4%) | |
| B (t) | Alluvium | 7 | 10.7 | - | 3.5 (0.3%) | # |
| | Kenny Hill | 48 | 3.3 | - | 6 (0.5%) | # |
| | | 60 | 3.0 | - | 9.5 (0.8%) | |
| | | 84 | 1.4 | - | 3.5 (0.3%) | |
| C (c) | Alluvium | 7 | 14 | - | 9 (0.5 %) | Pile-soil interface & socket not fully mobilised. |
| | Kenny Hill | 72 | 1.6 | - | >10 (0.6%) | |
| | | 17 | 3.7 | - | 5 (0.3%) | Slump zone # |

Notes: (c) represent compression pile; (t) represent tension pile; * represent percentage of critical movement as compared to pile diameter; # strain softening observed.

However, for tension pile, the $f_{s,mob}$/N ratio in Kenny Hill formation ranges from 1.4 to 3.3. The critical movement at pile-to-soil interface is recorded at displacement of 0.3% to 0.8% of pile diameter, which is relatively small.

For non-rock socketted bored pile, the critical movement at pile-to-soil interface in between 0.4% and 0.8% of the pile diameter is recorded in Kenny Hill formation. For rock socketted pile with brittle socket movement restricting the mobilisation of pile-soil interface resistance, hence there appears no sign of obvious yielding observed in the resistance mobilisation except for the slump zone, in which the yielding occurs at 4 to 5mm movement as shown in Fig. 12.

## 5. CORRELATION FOR ROCK SOCKET FRICTION

A total of 51 nos. rock cores were collected for Unconfined Compressive Strength (UCS) tests. According to ISRM (1985), the UCS value of rock can be correlated to $I_{s(50)}$ using conversion factor ranging between 20 and 25. However, it is also reported from the test results of many different rock types, the ratio can vary between 15 and 50 especially for anisotropic rocks. Hence, errors up to 100% are not uncommon if an arbitrary ratio value within the typical range is chosen to predict UCS value from point load tests.

As such, 13 out of 51 rock core samples were sent for point load tests (PLT) to establish the correlated ratio between the UCS and $I_{s(50)}$ for the limestone found at this site. The UCS tests and PLT results for the collected rock cores during the borehole exploration are shown in Table 3, in which the range of calibrated conversion factor is 13.3 ± 7.7 with considerable scatter of data. For design purpose, a linear relationship of 13.3 was adopted for the correlation of UCS with $I_{s(50)}$ for limestone bedrock as per Eq. (1).

$$UCS = 13.3\, I_{s(50)} \quad (1)$$

where UCS = unconfined compressive strength for rock; $I_{s(50)}$ = point load index for 50mm diameter core.

Table 3 Summary of Unconfined Compressive Strength Tests and Point Load Tests Results

| **BH** | **RQD (%)** | **$I_{s(50)}$ (N/mm$^2$)** | **UCS (N/mm$^2$)** | **Conversion Factor, K = UCS/$I_{s(50)}$** |
|---|---|---|---|---|
| P3/BH4 | 85 | 1.94 | 39.6 | 20.4 |
| P5/BH1 | 30 | 4.24 | 46.4 | 10.9 |
| P5/BH1 | 85 | 3.97 | 61.0 | 15.4 |
| P5/BH2 | 14 | 5.41 | 51.9 | 9.6 |
| P5/BH3 | 37 | 8.40 | 53.5 | 6.4 |
| P5/BH3 | 63 | 5.20 | 81.9 | 15.8 |
| P5/BH4 | 24 | 4.93 | 45.6 | 9.2 |
| P5/BH6 | 45 | 2.00 | 56.7 | 28.4 |
| P5/BH6 | 8 | 4.93 | 30.2 | 6.1 |
| P5/BH7 | 44 | 7.70 | 43.3 | 5.6 |
| P5/BH8 | 12 | 1.72 | 42.1 | 24.5 |
| P5/BH9 | 40 | 3.50 | 59.0 | 16.9 |
| P5/BH11 | 51 | 9.55 | 35.8 | 3.7 |
| **Sample Mean (MPa)** | | | $\bar{x}$ = | **13.3** |
| **Standard Deviation (MPa)** | | | = | **7.7** |
| **Coefficient of Variation (COV)** | | | = | **60%** |
| **Standard Deviation Range for Conversion Factor, K** | | | **Max =** | **21.0** |
| | | | **Min =** | **5.6** |

The estimated UCS values correlated to point load test results on the collected rock cores by Eq. (1) during coring of Test Pile C are shown in Table 4.

Neoh (1998) recommended the maximum allowable rock socket friction for Kuala Lumpur Limestone in Malaysia shall be limited to 5% of either the rock UCS value or characteristics concrete strength of pile, whichever is smaller as per Eq. (2) below.

$$f_{s,all} = 0.05 * \{\min(UCS, f_{cu})\} \quad (2)$$

where $f_{s,all}$ = allowable rock socket friction; $f_{cu}$ = concrete compressive strength at 28 days (35MPa).

Table 4 Summary of Point Load Tests and Mobilised Rock Socket Frictions for Test Pile C socketted in Limestone

| **Measured $f_{s,mob}$ (kN/m$^2$)** | ***UCS (N/mm$^2$)** | **Ratio of $f_{s,mob}$ / UCS** | **#$f_{s,all}$ (kN/m$^2$)** | **Ratio of $f_{s,mob}$ / $f_{s,all}$** |
|---|---|---|---|---|
| 1670 | 14.18 | 0.12 | 709 | 2.4 |
| 1400 | 33.60 | 0.04 | 1680 | 0.8 |

***** UCS is average UCS value estimated using Eq. (1) based on PLT results carried out on rock samples collected at every 0.5m spacing

\# Allowable rock shaft friction is calculated using Eq. (2)

The load transfer mechanism of earlier mobilisation of the upper portion of rock socket friction to much higher value has rendered relatively lower degree of socket mobilisation at the lower rock socket under the maximum test load. From the above results, it is clear that rock socket friction estimated by Neoh's approach appears to be conservative.

Another method of designing the rock socket friction commonly used in Malaysia is the approach developed by William & Pells (1981).

$$f_s = \alpha * \beta * UCS \qquad (3)$$

where $f_s$ = ultimate rock socket friction
$\alpha$ = reduction factor with respect to UCS
$\beta$ = reduction factor with respect to the rock mass effect

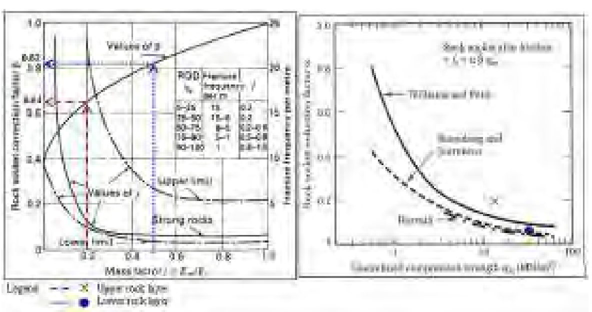

Fig. 14 Back-calculated $\alpha$ and $\beta$ factors of Test Pile C Data plotted on William & Pells Approach

The back calculated factors of and from the mobilised rock socket frictions for Test Pile C are plotted in Fig. 14. It is observed that the mobilised socket friction at the lower part of the rock socket is still lower than the ultimate socket resistance estimated by William and Pells' approach (1981). Such outcome is expected as the socket friction at this lower portion of rock socket has not been fully mobilised to its ultimate resistance at the maximum test load as yielding is not even reached in Fig. 12. However, the trend of test results tallies well with the approaches mentioned above and reveals that more optimistic performance of this limestone rock socket can be readily expected.

## 6. CONCLUSION

This paper presents three instrumented preliminary load test results on bored piles installed at alluvial deposits underlain by the meta-sedimentary Kenny Hill formation and followed by Kuala Lumpur Limestone formation. The following conclusions are made:

a. These test piles mainly utilise the frictional resistance to support the designed capacity of the pile with safety factor of at least 2.0. All the test piles were satisfied with the settlement requirements.
b. The contribution from mobilised base resistance is generally low within the limit of acceptable settlement, therefore base resistance should not be considered in the bored pile design unless proper base cleaning can be carried out.
c. The ratio of mobilised unit skin friction, $f_{s,mob}$ to average SPT-N reduces exponentially with increasing SPT-N values
d. The correlation of rock test results suggest a conversion factor of 13.3 is fairly acceptable for estimating unconfined compressive strength of limestone bedrock encountered in this site.
e. The application of point load test at field is useful to predict the UCS values of rock encountered during pile construction.
f. The test results evidence that the current Malaysia design practice of rock socket in limestone formation is somehow conservative. The inherent safety factor in allowable stress design is expected much higher than 2.0 as commonly perceived in pile design. However one shall always remember that failure of rock socket is normally brittle.

## 7. REFERENCES


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