Review of Load Test Performance of Bored Pile Foundation in Weathered Meta-Sedimentary Formation and Kuala Lumpur Limestone

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ABSTRACT: This paper presents the performance of three preliminary instrumented bored cast-in-situ test piles socketed in weathered meta-sedimentary formation and limestone formation in Kuala Lumpur, Malaysia. The test piles were instrumented with proprietary Global Strain Extensometer technology and were load tested under quick maintained load test procedures with two piles under compressive axial loading and one under tensile loading. The test results throw some light on the frictional load transfer behaviour of bored piles in meta-sedimentary formation. For the long compression bored pile embedded in soil by wet hole construction, only 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 the rock socket. Unconfined compressive strength tests and point load tests on both the collected rock cores during the boreholes exploration and the rock fragments recovered during pile construction were both correlated with the mobilised rock socket resistances. However, the correlations are rather 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 1125 Tons was applied to the tension pile using four 1000 Tons capacity hydraulic jacks.

Figure 1 4560 Tons Maintained Static Pile Load Test & 1120 Tons Tension Pile Load Test

This paper is aimed to present the test results of these three 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.

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 VI metamorphosed sandstone/shale and completely weathered Grade V residual soils. The overburden alluvial materials generally consist of silty or gravelly SAND.

3. 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 boreholes collapse for top 7m to 14m of loose alluvial soils. Drilling speeds in the installation of test piles are summarised in Table 1.

Table 1 Summary of Pile Installation Records

<table>
<thead>
<tr>
<th>Pile C ref</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Pile Ref.</td>
<td>1500mm</td>
<td>1200mm</td>
<td>1800mm</td>
</tr>
<tr>
<td>Drilling System</td>
<td>BG22/31</td>
<td>CMV</td>
<td>BG45</td>
</tr>
<tr>
<td>Stabilising Fluid</td>
<td>Bentonite</td>
<td>Bentonite</td>
<td>Bentonite</td>
</tr>
<tr>
<td>Temporary Casing Length</td>
<td>7m</td>
<td>7m</td>
<td>14m</td>
</tr>
<tr>
<td>Drilling Speed (m/hr)</td>
<td>3.7m/hr</td>
<td>6.7m/hr</td>
<td>9.3m/hr</td>
</tr>
<tr>
<td>Keng Hill Formation</td>
<td>N.A.</td>
<td>N.A.</td>
<td>0.5m/hr</td>
</tr>
<tr>
<td>Limestone Formation</td>
<td>1 day</td>
<td>1 day</td>
<td>1.5 days</td>
</tr>
<tr>
<td>Concrete Casting Method</td>
<td>Tremie 2</td>
<td>Tremie 2</td>
<td>Tremie 2</td>
</tr>
</tbody>
</table>

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 (Figs. 5a and 5b) 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.

4. INTERPRETATION OF TEST PILE RESULTS

4.1. Test Pile A – Compression Pile

Figs. 6 and 7 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. This displacement of pile shaft, 11mm has led to reduction of shaft friction at the upper portion of pile. Strain softening of pile 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.

Figure 8 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/m² 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. 10 and 11. 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/m² with the displacement of 15mm. The average ultimate shaft friction was...
about 120kN/m². 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

A summary of mobilisation of pile capacities and critical pile movement at every soil strata are tabulated in Table 2. From the test results, a similar trend of skin friction is observed for all the test piles. The rate of skin friction capacity is gradually reducing with the increasing depth as well as the SPT-N profiles. All the test results measured unit shaft friction for alluvium soil with lower average SPT-N values tends to provide a higher factor of $f_{s,mob}$/N as compared to Kenny Hill formation. This may due to the inherent surface roughness created by boring process between the alluvium soil, which mainly consists of coarse dense sand and the cast in-situ concrete. In Kenny Hill formation, the $f_{s,mob}$/N factor for shaft friction under compression load is quite consistent, in which it ranges from 1.5 to 2.2.

**Table 2 Summary of Mobilisation of Pile Capacities and Critical Pile Movements**

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Soil Stratum</th>
<th>$\overline{N}$</th>
<th>$f_{s,mob}$/N</th>
<th>$f_{b,mob}$/N</th>
<th>Critical movement, $Z_s$(%)*</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Alluvium</td>
<td>7</td>
<td>11.4</td>
<td>-</td>
<td>10.7 (0.7%)</td>
<td>#</td>
</tr>
<tr>
<td></td>
<td>Kenny Hill</td>
<td>15</td>
<td>7.3</td>
<td>-</td>
<td>34 (2.3%)</td>
<td>#</td>
</tr>
<tr>
<td></td>
<td>Hill</td>
<td>150</td>
<td>1.8</td>
<td>5.3</td>
<td>39 (2.6%)</td>
<td>#</td>
</tr>
<tr>
<td>B</td>
<td>Alluvium</td>
<td>7</td>
<td>10.7</td>
<td>-</td>
<td>19 (1.6%)</td>
<td>#</td>
</tr>
<tr>
<td></td>
<td>Kenny Hill</td>
<td>48</td>
<td>5.3</td>
<td>-</td>
<td>6 (0.5%)</td>
<td>#</td>
</tr>
<tr>
<td></td>
<td>Hill</td>
<td>60</td>
<td>3.0</td>
<td>-</td>
<td>8 (0.7%)</td>
<td>#</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td>14 (1.2%)</td>
<td>#</td>
</tr>
<tr>
<td>C</td>
<td>Alluvium</td>
<td>7</td>
<td>14</td>
<td>-</td>
<td>16 (0.9%)</td>
<td>Pile not</td>
</tr>
<tr>
<td></td>
<td>Kenny Hill</td>
<td>52</td>
<td>17</td>
<td>-</td>
<td>5 (0.3%)</td>
<td>Fully</td>
</tr>
<tr>
<td></td>
<td>Hill</td>
<td>17</td>
<td>3.7</td>
<td>-</td>
<td>5 (0.3%)</td>
<td>Mobilised</td>
</tr>
</tbody>
</table>

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 factor of shaft friction ranges from 1.4 to 3.3. The maximum skin friction is measured at displacements of 0.5% to 1.2% of pile diameter, which is relatively small.

For bored pile not socketting into rock, the maximum skin friction is observed at settlement of between 2.3% to 3.1% of the pile diameter. This implies the possibility of load transferred further away from the test pile as compared to tension pile. For rock socketted pile, the factors of mobilised skin friction at soil is lower with minimum settlement of 0.3% to 0.9% of pile diameter, this imply the likely of slip failure as mentioned earlier.

5. **CORRELATION WITH ROCK TEST RESULTS**

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 $L_{450}$ using conversion factor of 20 to 25. However, it is also reported that from the test results of many different rock types, the ratio can vary between 15 and 50 especially for anisotropic rocks. So errors of up to 100% are not uncommon if an arbitrary ratio value is chosen to predict UCS value from point load tests.

As such, 13 out of 51 rock core samples were also sent for point load tests to establish the correlated ratio between the UCS and $L_{450}$ for the limestone encountered at this site. The UCS tests and PLT results for the collected rock cores during the borehole exploration are shown in Table 3.

Table 3 shows that the range of conversion factor is 13.3 ± 7.7 with considerable scatter of data. For design purpose, a linear relationship of 13.3 for the correlation of UCS with $L_{450}$ for limestone bedrock was adopted.

$$UCS = 13.3 L_{450}$$

where $UCS = $ Unconfined Compressive Strength for rock

$L_{450} = $ Point Load Index for 50mm diameter core

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

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The results reveal that the test results tally well with the already exceeds the ratio of 2.0; in which, the mobilisation of rock shaft in maintaining the tested load. The higher the ratio indicates the higher the degree of friction and measured mobilised shaft friction can be used as a crude check on the degree of mobilisation of rock shaft friction at the lower portion of rock socket has not been fully mobilised to its ultimate resistance in the load testing.

6. CONCLUSION

The case study of the three aforementioned instrumented test piles throw some light on the frictional behaviour of bored pile installed in alluvial deposits underlain by the meta-sedimentary formation and Kuala Lumpur Limestone formation including the resistance mobilisation with pile movements. The test results indicate that these test piles mainly utilise the frictional resistance to support twice the designed capacity of the pile, while at the same time, satisfying the settlement requirements. It was also observed that contribution to pile capacity from mobilised base resistance is generally low within the limit of acceptable settlement.

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. The application of point load test at field is useful to predict the UCS values of rock encountered during pile construction. However, the correlation of UCS with mobilised shaft friction is rather scattered and does not have a clear trend.

7. REFERENCES