

INTERPRETATIONS OF INSTRUMENTED BORED PILES IN KENNY HILL FORMATION

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ABSTRACT

This paper presents the results of two instrumented bored cast-in-situ test piles at Kenny Hill Formation of Kuala Lumpur Area. Vibrating wire strain gauges and the extensometers have been specifically planned at the strategic locations to reveal the load transfer behaviour of both the pile shaft and the pile base. Irregular shaft surface has significant impact to the overall load transfer behaviour, in which the bulged pile shaft transferred significant amount of test load in the form of direct bearing to the subsoil at the lower portion of bulged shaft. For the long bored pile embedded in soil under the condition of wet hole construction, only about 1% to 3% of the total test load was transferred down to the pile base. Whereas the short rock socket pile constructed in dry hole condition has about 63% to 77% of the total test load transferred to the pile base. Unconfined compressive strength tests and point load tests on both the collected rock cores during the borehole exploration and the rock fragments during pile construction were carried out to correlate with the mobilised shaft resistances. However, the correlation is rather scattered and does not have a clear trend.

Keywords: Bored piles; Load test; Load transfer

1. INTRODUCTION

Two instrumented test piles, namely Instrumented Test Pile A (ITP-A) and Instrumented Test Pile B (ITP-B), were installed and load tested at the project site to verify the design of the bored pile foundation. Vibrating wire strain gauges and extensometers were installed in the test piles to reveal the load transfer behaviour along the pile. The two test piles were installed at different subsoil conditions, in which ITP-A is a rock socketted pile whereas ITP-B is a soil friction pile.

The site is located at Bukit Jalil, Kuala Lumpur. The site is predominantly underlain by weathered meta-sedimentary bedrocks of Kenny Hill Formation aged from Permian to Carboniferous, which comprises meta-sandstone interbedded with meta-siltstone and shale.

Figure 1 shows the boreholes at the close proximity to the two test pile locations. The site is underlain by weathered meta-sedimentary soils, which mainly consist of sandy clay to clayey silt. Beneath the subsoil layer is interbedded shale, sandstone and siltstone. Generally the bedrock is highly fractured with Rock Quality Designation (RQD) values of less than 20%. From the unconfined compressive test results on the rock cores recovered from the boreholes, the unconfined compressive strength (q_{uc}) ranges from 6.8 to 83MPa, which is classified as weak to moderately strong rock. Due to different weathering rate of layered materials, intermediate hard materials with extrapolated SPT-N value of more than 50 were also found at the site. The boreholes also show low groundwater table at the site, as the site is located over a hill with virtually insignificant catchment.

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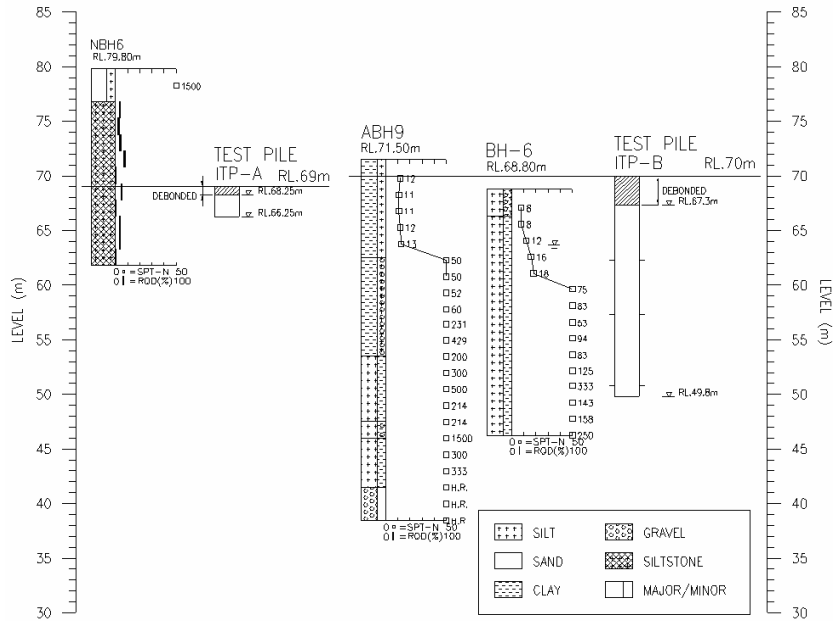


Figure 1: Borehole logs of piles ITP-A and ITP-B

2. INSTRUMENTED TEST PILE A

The instrumented test pile A (ITP-A) is a $\phi 900\text{mm}$ rock socket pile. This test pile is a preliminary test pile to be load tested to three times the pile structural capacity. According to BS8004: 1986, the structural capacity is based on the working stress of 0.25 times the 28-day concrete strength. For test pile ITP-A, the structural capacity is 4,700kN.

2.1 Installation of test pile

Figure 2 shows the instrumentation details of test pile ITP-A. Four levels of strain gauge and three extensometers were installed in the test pile. Each level consisted of four strain gauges. Strain gauges were welded to the main steel reinforcement. 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 at critical locations along the pile shaft.

During drilling of the pile, rock samples were recovered and collected at 0.25m intervals. By using a portable point load test equipment, the recovered rock samples were tested at the site. Unconfined compressive strength (q_{uc}) of the rock samples was correlated from the point load test results, in which the q_{uc} value was taken as 22 times the point load index (I_{s50}). The drilled hole was dry when the base of the pile was reached. The borehole wall surface was observed to be rough and irregular due to the inherent massive joint sets and foliation of the weathered bedrock.

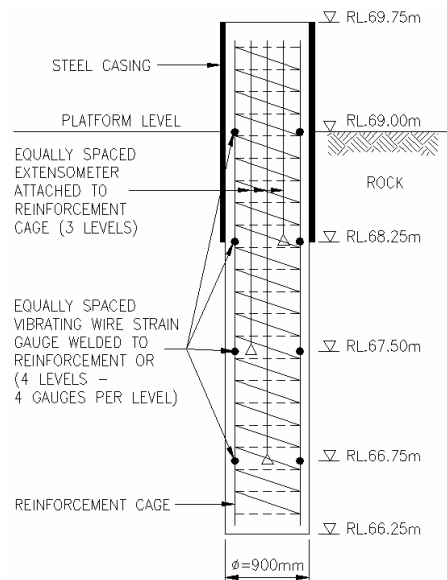


Figure 2: Details of test pile ITP-A

During concreting, steel casing of 1.5m long was installed to build up the pile head for testing. As shown in Figure 2, the embedment of the casing is about 0.75m below the ground surface. The first and second levels strain gauges were installed within the pile section with steel casing. Before the load test, the pile

shaft between the ground level to the first level strain gauges were debonded by removing the surrounding rock.

2.2 Pile load test

Maintained load test was carried out on pile ITP-A nine days after the pile installation. Static load was applied by using two hydraulic jacks acting against kentledge system. Four calibrated load cells were used as primary load measurement device and an additional pressure gauge connected to hydraulic jacks was also provided to check the applied load. Figure 3 shows the plot of the load measured using load cell against the load measured using pressure gauge. Differences in the readings between two devices indicate presence of ram friction in the hydraulic jack. During loading, the load recorded in the pressure gauge was about 9% higher than the load cell reading. The load variation during unloading was about 2%. These load variations justified the need to use a calibrated load cell for load measurement.

The pile head displacement was measured by using four displacement transducers with accuracy of up to 0.01mm, whereas displacement of the extensometers was measured by using displacement transducers.

The test pile was loaded and unloaded in three cycles with test load of up to 14,100kN, which is three times the pile structural capacity. The applied load was maintained for 15 minutes for each load increment and was maintained for 60 minutes at 100%, 200% and 300% of the pile structural capacity respectively.

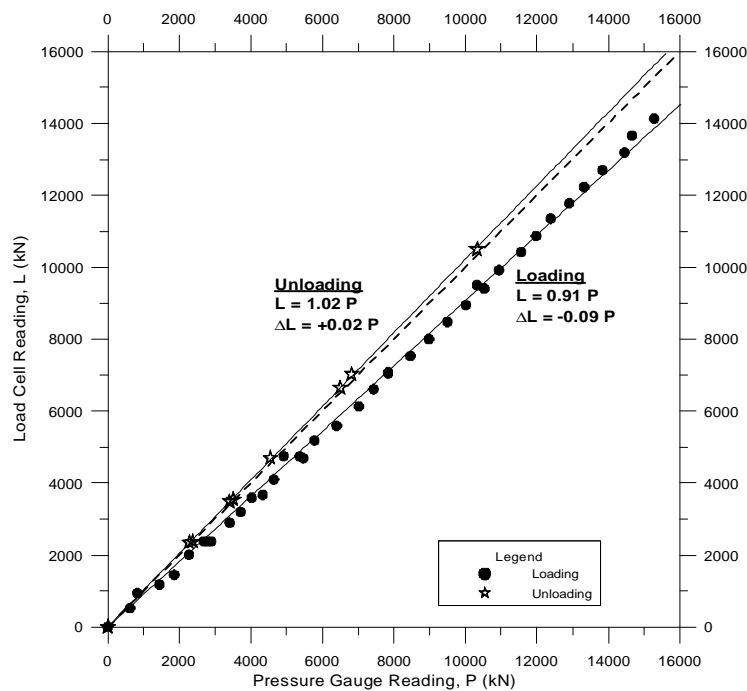


Figure 3: Comparison between readings of pressure gauge and load cell

2.3 Load test result

The attempt to fail the test pile ITP-A was not successful after loading up to three times the structural capacity. Pile top settlement was only about 9mm at maximum test load of 14,100kN.

When reviewing the strain gauge results, the strain at the second level was larger than the strain at the first level, even though both the first and second level strain gauges were located within the steel casing. It may imply that the pile section at the depth of second level strain gauges are more like a non-transformed section, as the second level strain gauges were located near the end of the steel casing. For analysis purpose in this paper, the second level strain gauges were assumed to be non-transformed section and were used as the calibrating level. Table 1 shows the interpreted load distribution along the pile shaft and the pile base. As shown in Table 1, about 63% to 77% of the applied loads were transferred to the base. It shows significant bearing resistance of the pile base in dry hole.

Table 1: Summary of load distribution of test pile ITP-A

Depth (m)	50% S.L.	100% S.L.	150% S.L.	200% S.L.	250% S.L.	300% S.L.
0	100	100	100	100	100	100
0.75	100	100	100	100	100	100
1.5	82	85	86	86	85	83
2.25	71	76	79	80	79	77
2.75	63	71	75	77	76	73

*S.L.: pile structural capacity = 4,700kN

The load transfer curves for the shaft resistance (f_s) and base resistance (f_b) are shown in Figure 4. The ultimate shaft and base resistances were not fully mobilised, as the load transfer along the shaft and the base still shows the trend of linearly increasing during loading to 14,100 kN (300% of structural capacity). The mobilised shaft resistance was greater at the upper portion of the test pile (ie. at depth from 0.75m to 1.5m), with maximum value of 1.1MPa under the applied top load of 14,100 kN. The maximum mobilised base resistance during the test was about 16MPa.

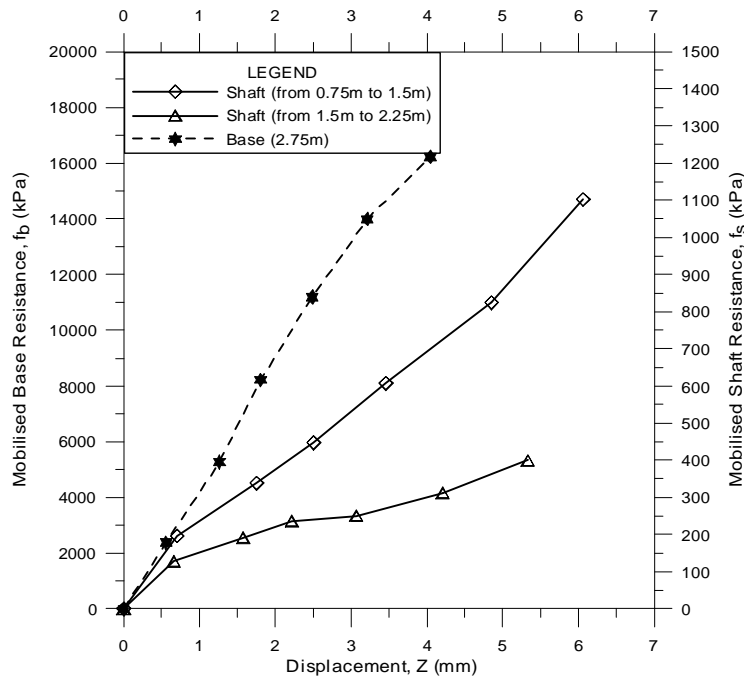


Figure 4: Load transfer curves for test pile ITP-A

The test results are also correlated with unconfined compressive strength (q_{uc}). For test pile ITP-A, the q_{uc} values are correlated from the point load index of the recovered rock samples by using the empirical equation $q_{uc} = 22 \times I_{s50}$, as shown in Figure 5. The q_{uc} values are quite scattered, probably due to the strength variation of the layered weathered meta-sedimentary formation. These average values are plotted against the maximum mobilised shaft resistance at the respective levels of the test pile ITP-A, as shown in Figure 6. However, it can be seen that the maximum mobilised shaft resistances are still much lower than the ultimate shaft resistances computed by using William and Pells' expression. Such results are expected as the shaft resistances were not fully mobilised to the ultimate resistance in the maintained load test. Therefore, it is anticipated that there is still plenty of room for the shaft resistances to approach the prediction by William and Pell's expression.

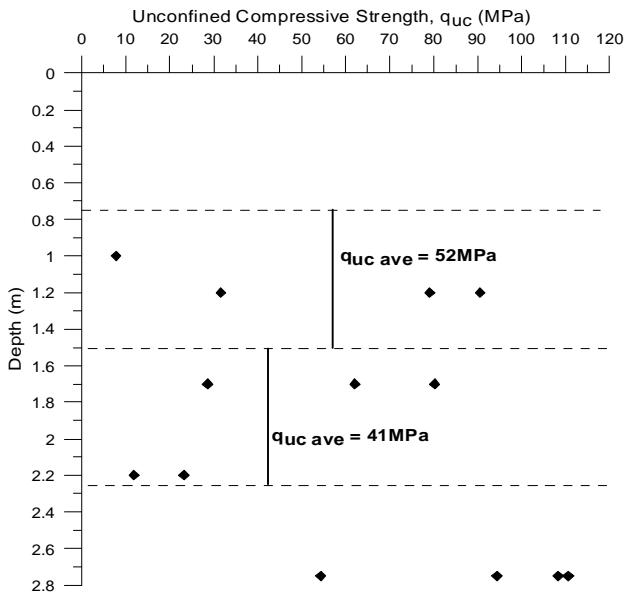


Figure 5: Depth vs. q_{uc} correlated from PLT

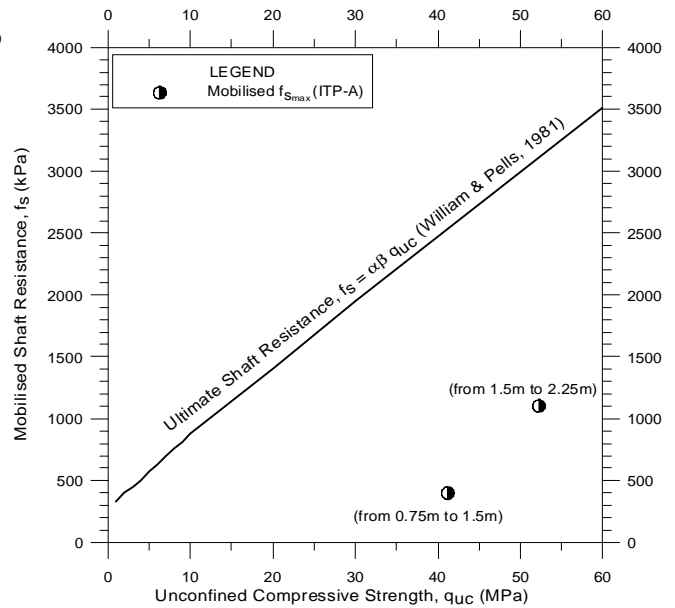


Figure 6: Correlation of mobilised $f_{s\max}$ and q_{uc}

3. INSTRUMENTED TEST PILE B

The instrumented test pile B (ITP-B) is a $\phi 1000\text{mm}$ working pile with pile working load of 3,600kN. The test pile is a soil friction pile. The pile length is about 20m from the ground level. This test pile was loaded up to 7,200kN, which is two times the pile working load.

3.1 Installation of test pile

Figure 7 shows the instrumentation details of test pile ITP-B. Five levels of strain gauges and four extensometers were installed in the test pile. Installation details of this test pile were modified to prevent recurrence of the strain problems encountered in the test pile ITP-A as mentioned in Section 2.3. The steel casing was removed up to the depth above the first level strain gauges, so that the pile cross sections at all instrument levels were same.

The drilled hole was wet when reaching the lower portion of the pile. In order to record any overbreak along the pile length, the concrete volume was monitored at certain levels during concreting. The actual concrete volume was then compared against the theoretical concrete volume. Based on the abovementioned information, the test pile had overbreak at depth from 1.5m to 7m below the ground level.

3.2 Pile load test

Maintained load test was performed on the test pile ITP-B eleven days after the pile installation. This test pile was only loaded and unloaded in two cycles with test load of up to 7,200kN. The applied load was maintained for 15 minutes for each load increment and was maintained for 6 hours at 100% and 200% of the working load.

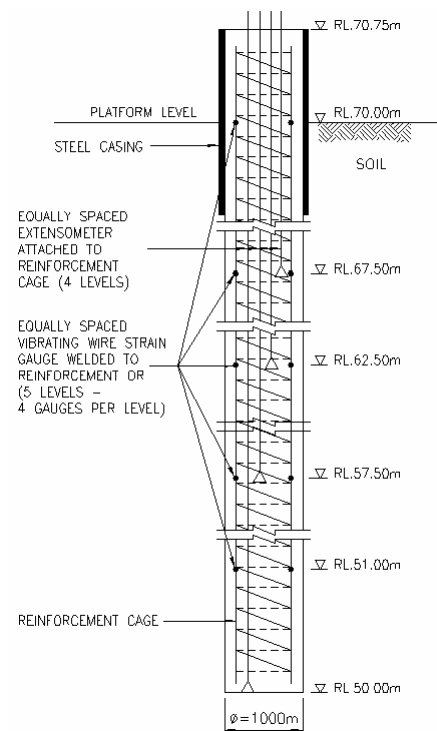


Figure 7: Details of test pile ITP-B

3.3 Pile load test results

The test pile ITP-B did not fail after loading up to two times the working load. The pile top settlement was about 10mm at the maximum test load of 7,200kN.

Table 2 tabulates the load distribution along the pile shaft and the pile base. Similar to the test pile ITP-A, the first 2.5m of the bored pile was debonded by pre-augering the soil surrounding the pile. Therefore, the load recorded at the second level strain gauges was same as the load at the first level strain gauges. As shown in Table 2, the pile capacity is mainly contributed by the shaft resistance, as only 1% to 3% of the applied load was carried by the base throughout the whole range of the applied top load.

Table 2: Summary of load distribution of test pile ITP-B

Pile top load	50% W.L.	100% W.L.	150% W.L.	200% W.L.
Depth (m)	Percentage of load transferred (%)			
0	100	100	100	100
2.5	100	100	100	100
7.5	50	58	62	66
12.5	28	37	42	48
19	4	5	7	9
20	0	1	1	3

*W.L.: pile working load = 3,600kN

Load transfer curves for the shaft resistance and base resistance are shown in Figure 8. The ultimate shaft and base resistances were also not mobilised, as the load carried along the shaft continued to increase with increasing pile settlement during the load test. Higher shaft resistance was mobilised at the upper portion of the pile (at 5m below ground level), with the value of up to 160kPa under applied top load of 7200 kN. This contradicts with the borehole information, which shows consistency of subsoil is increasing with depth. It is supposed that ultimate shaft resistance is increasing with higher SPT-N value (Meyerhorf, 1976). The portion with higher shaft resistance was about at the pile section with recorded overbreak. It is believed that some of the loads were transferred to the surrounding soil through direct bearing at the bulged pile section. Soft toe condition can be seen in Figure 8, in which the base resistance only started to be mobilised after about 1mm settlement at the pile base.

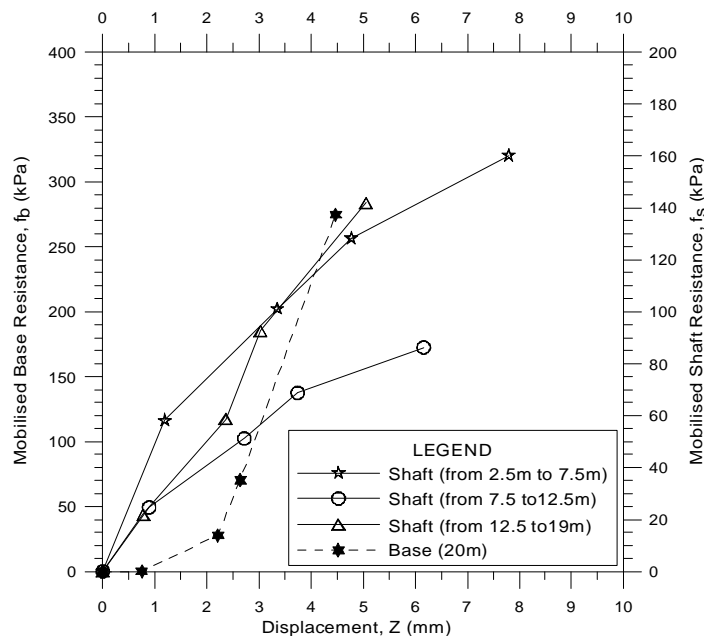


Figure 8: Load transfer curves for test pile ITP-B

The correlation of maximum mobilised shaft resistance ($f_{s \max}$) with SPT-N values is plotted in Figure 9. The suggested empirical correlation by Chang and Broms (1990) and Tan, et al. (1998) for the ultimate shaft resistance for residual soil, $f_s = 2 \times \text{SPT-N}$ is also plotted. Generally the mobilised shaft resistances did not reach the ultimate values as recommended by Tan, et al. At depth from 2.5m to 7.5m, the mobilised resistance was much higher than the predicted ultimate shaft resistance, implying that part of load was transferred through bearing on soil at the bulged pile section, which is considered not representative for this correlation.

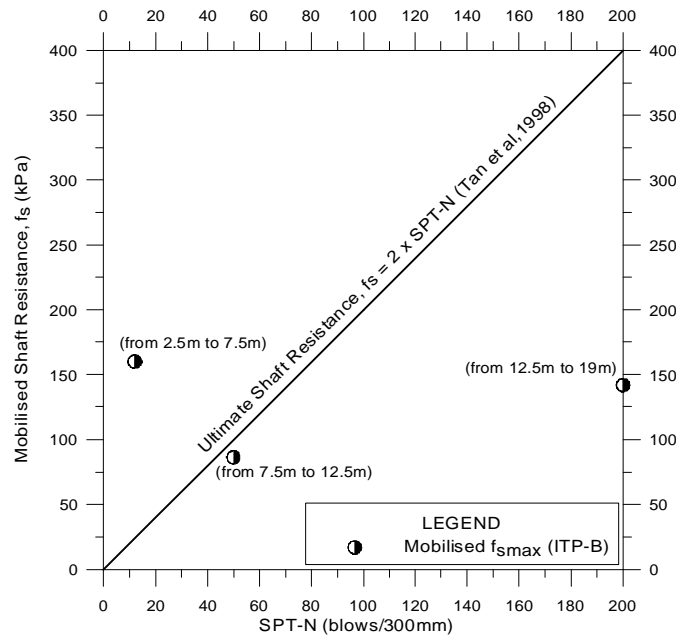


Figure 9: Correlation of mobilised shaft resistance with SPT-N

4. CONCLUSIONS

This paper presents two instrumented load test results on bored piles at Kenny Hill Formation. Based on the interpretation of the test results, correlations with soil/ rock parameters could not be established due to possible problems during the test pile construction, variation in ground condition and also the test piles were not load to failure. Nevertheless, the following conclusions are made:

- Irregular pile shaft will certainly affect the load distribution. Instead of shearing along the pile shaft, the load was transferred through direct bearing to the soil below the bulged shaft.
- Significant portion of top applied load was transferred to the base of the short bored pile constructed under dry hole condition with proper base cleaning. If the base resistance can be utilised, this will certainly allow optimisation of the pile design.
- For long bored pile constructed under wet hole condition, mobilised base resistance was insignificant. Soft toe condition was also observed. Therefore, base resistance should not be considered in the bored pile design unless base cleaning for proper base contact can be carried out.
- Construction records such as concreting record can be very useful to determine the variation of pile section and serve important information for interpretation of instrumentation results.

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