Effects of base grouting and deep cement mixing on deep foundation bored piles at Marina Bay Financial Centre and a study on the geotechnical design parameters for deep foundation bored piles in various soil formations in Singapore

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ABSTRACT

Deep foundation bored piles are normally adopted for the foundation system at Marina Bay area in Singapore. The geological formation in this area is very challenging for the underground construction. Currently, there are numbers of high-rise buildings in this area and would have many more in the future as shown in the government’s Master Plan. Therefore, an effective and innovative method to design the pile foundation safely and environmentally friendly has to be developed. In this paper, the author describes the effects of base grouting and deep cement mixing (DCM) on the pile performance at Marina Bay Financial Centre (MBFC). The results show that base grouted piles are well performed in the pile toe movement and mobilised skin friction at the bottom section of the pile. Soil improvement works are usually needed for the deep basement excavation for the earth retaining structure at Marina Bay area and DCM is normally adopted. The thickness varies from 5m to 7m and is usually founded in the reclaimed sand or soft marine clay layer. The effects of DCM on the mobilised skin friction of pile cannot be ignored. The effects of construction methods using Bentonite and Polymer are also discussed. Installation of bored piles using bentonite method would be much faster than polymer. However, the bentonite might cause the reduction of shaft skin friction. It has to be taken care to avoid it during construction. A study of the geotechnical design parameters for deep foundation bored piles in various types of soil formations in Singapore is presented. The data would be useful for the practitioners to provide a practical and economical bored piles design.

Keywords: deep foundation bored piles, base grouting, DCM, skin friction, end bearing

1 INTRODUCTION

MBFC is one of the most iconic projects in Singapore. The project commenced in 2006 and comprises of 3 high-rise tower blocks and 2 podiums. It is located at the Marina Bay area, which is one of the most difficult geological formations in Singapore. It is surrounded by the existing common services tunnel (CST) and mass rapid transit (MRT). Generally, the formation comprises reclaimed sand or hydraulic sand fill, upper marine clay, fluvial layer, lower marine clay and old alluvium. Total 6 numbers of preliminary bored piles and 1055 numbers of working bored piles from diameter 800mm to 1800mm were installed. Preliminary piles were instrumented with and without base grouting for the six ultimate load tests. The general geological formation for MBFC is shown in Fig. 1.

2 INSTRUMENTED TESTS RESULTS

The six instrumented tests piles are summarised in Table 1 and Table 2. The load transfer curves for the test piles are shown in Fig. 2.

Table 1. Information for the instrumented ultimate test piles

| Test ID     | Size   | Working Load | Max. Base Load | Within DCM | Drilling suspension fluid |
|-------------|--------|--------------|----------------|-------------|----------------------------|
| R1-UTP01    | 1500   | 13250        | 2.62           | Yes         | No                         |
| A2-UTP03    | 1500   | 13250        | 3.16           | No          | No                         |
| A2-UTP01    | 1200   | 8450         | 3.25           | Yes         | No                         |
| A3-UTP01    | 1200   | 8450         | 2.50           | No          | No                         |
| A4-UTP01    | 1200   | 8450         | 2.60           | No          | No                         |
| A2-UTP02    | 1000   | 5750         | 3.25           | Yes         | Bentzite                   |
| (Additional)|        |              |                |             | (9-16m below EGL)           |

The skin friction for the six load tests had been fully mobilised or substantially mobilised at a relative displacement between the pile and soil of about 1.5% to 2% pile diameter at maximum test loads, except for A2-UTP02. Generally, end bearing for the test piles was not substantially / fully mobilised due to the fact that the piles are rather long, from 49m to 60m. It was
noted that substantial pile toe settlement of about 0.5% to 4.5% pile diameter was required in order to slightly or substantially mobilise the end bearing of the piles from about 2829kN/m² to 8860kN/m². It is also to note that A4-UTP01 (Additional) was compensated and conducted at 6m away from the A4-UTP01. This was because the pile performance for A4-UTP01 was not satisfactory and did not comply with the contract specifications.

EGL ~ RL 103.500m

| Layer | Description | Thickness (m) | N ≤ 30 | N ≤ 4 | N ≤ 15 | N ≤ 30 | N ≤ 50 | N ≤ 100 |
|-------|-------------|---------------|--------|-------|--------|--------|--------|---------|
| Layer 1 | Fill material, Very loose to medium dense hydraulic sand fill | 10 to 25 | |
| Layer 2A | Very soft to soft marine clay (upper marine clay) | 10 to 15 | |
| Layer 3 | Loose to medium dense silty sand / clay (Fluvial material) | 5 | |
| Layer 2B | Soft to stiff marine clay (lower marine clay) | 5 | |
| Layer 4 | Medium dense clayey to silty sand and stiff to very stiff silty to sandy clay (Old Alluvium) | 0 to 10 | |
| Layer 5A | Dense, silty sand and hard, silty to sandy clay with some gravels (Old Alluvium) | 0 to 10 | 30 | |
| Layer 5B | Very dense, clayey to silty sand and hard, silty to sandy clay (Old Alluvium) | 5 to 15 | 50 | |
| Layer 6 | Very dense, silty sand and very hard, silty to sandy clay and sandy silt (Old Alluvium) | (>>40) | |

Fig. 1. General geotechnical formation at MBFC

Table 2. Summary of test results

| Test ID | Pile top settlement shortening (mm) | Elastic settlement (mm) | Pile toe settlement (mm) | Mobilised skin friction at Layer 6 (kPa) | Mobilised end bearing (kPa) |
|---------|-----------------------------------|------------------------|-------------------------|------------------------------------------|---------------------------|
| R1-UTP01 | 30.63 | 23.17 | 7.46 | 556 | 2829 |
| A2-UTP03 | 38.63 | 25.34 | 13.29 | 924 | 5684 |
| A4-UTP01 (Additional) | 66.63 | 16.21 | 50.42 | 229 | 8860 |
| A4-UTP01 | 73.75 | 20.17 | 53.58 | 286 | 3713 |
| A2-UTP02 | 11.25 | 11.25 | 0.00 | 214 | 335 |

Fig. 2. Load transfer curves for instrumented test piles at MBFC

3 FACTORS AFFECTING TEST RESULTS

The test results indicate a complex and erratic distribution of the relative pile soil settlements, mobilised skin friction and mobilised end bearing. Various factors would have contributed to the test results, such as base grouting, improved soil layer (DCM) and construction method (different stabilising fluids).

3.1 Effects of base grouting on end bearing and pile toe settlement

R1-UTP01, A2-UTP01 and A2-UTP02 had been base grouted at least 7 days before commencement of the load tests. Refer to Fig. 3, the pile toe settlement of R1-UTP01 was 7.46mm or 0.49% pile diameter, when the end bearing had been mobilised to 2829kN/m². For A2-UTP01, the pile toe settlement was 13.29mm or 1.1% pile diameter at 5684kN/m². The other three piles, which did not base grouted, required 2.44% to 4.50% pile diameter to mobilise 2829kN/m² to 8860kN/m² of end bearing. The above results show that effects of base grouting on pile toe settlements are substantial. Base grouted piles required little pile toe movement to mobilise end bearing compared with non-base grouted piles. For A2-UTP02, the end bearing was only slightly mobilised at 335kN/m² with zero pile toe movement. The reason for such minimal mobilisation was due to DCM.
3.2 Effects of base grouting on skin friction

Base grouting was not only help to reduce the pile toe movement. It was also help to enhance tremendously on the skin friction above the pile toe. For R1-UTP01, the mobilised skin friction increased rapidly from 54m to 60.77m (4.5D from the pile toe, where D is the pile diameter) and the skin friction was about 626kN/m² as shown in Fig. 1. The mobilised skin friction was 924kN/m² from 52m to 54.7m (2.25D from the pile toe) for A2-UTP01. However, the mobilised skin friction at the pile base location was only 229 kN/m² to 314 kN/m², for piles without base grouting. Fig. 4 and Fig. 5 show the relationship between maximum mobilised skin friction and mean SPT N-value. The difference between base grouted and non-base grouted piles are shown clearly in the figures and Table. For example, the factor of $f_s/N$ for layer 6 in R1-UTP01 is 5.6 and for layer 5B is only 0.69; For A2-UTP01, factors for layer 6 and layer 5B are 9.25 and 2.5, respectively. Based on the test results, it was concluded that skin friction for base grouted pile shaft at 2D to 4D from toe level would increase from (2.5−3) N to about (6−9) N or from (0.43−0.6) to (0.99−1.71) for the $\beta$-values when comparing with piles without base grouting. The relationship between beta value and effective stress is shown below.

$$f_s = \beta \times \sigma_v'$$  \hspace{1cm} \text{(1)}$$

$$\beta = K_s \times \tan \delta$$  \hspace{1cm} \text{(2)}$$

where,

$K_s$ = lateral earth pressure coefficient  
$\delta$ = interface friction angle in degrees  
$\sigma_v'$ = average effective vertical stress along the pile shaft

The beta value of the base grouted pile increases due to the fact that the interface angle of friction between the pile shaft and the surrounding soil increases. Hence, it improves the side resistance.

3.3 Effects of improved soil layer (DCM)

Preliminary pile A2-UTP02 was installed after the DCM had been completed. The improved soil layer was from about 9m to 16m from existing ground level. The geological profile of this pile is shown in Table 3. The majority of the DCM layer is located within the loose sand layer. The behavior of the pile was totally different from the others. The mobilised skin friction for the pile shaft at DCM layer was 375kN/m², which is as good as the skin friction for soil with SPT N-value greater 100 (refer to Fig. 1). The DCM layer had contributed about 7000kN skin resistance to the pile load, which is about 1.3 times of the working load of the pile. Therefore, the skin friction at the lower layers, such as Layer 5 and Layer 6, had not been substantially mobilised. The mobilised pile bearing capacity was also minimal, at 355kN/m².

Fig. 3. Pile toe settlement versus mobilised end bearing

Fig. 4. Relationship between maximum mobilised skin friction and mean SPT N-value

Fig. 5. Relationship between maximum mobilised skin friction and effective vertical stress
With the combined effects of base grouting and improved soil layer, the pile toe settlement is zero. As such, pile top settlement is same as the measured elastic shortening. It is also important to note that the rate of pile top settlement was increased rapidly from 0.3233x10\(^{-3}\) mm/kN to 0.3739x10\(^{-3}\) mm/kN when the pile load went beyond 7000kN due to the localised effects of high mobilised skin friction at DCM layer as shown in Fig. 6.

Table 3. Geological profile for A2-UTP02 test pile

| Soil Layer | From (m) | To (m) | Soil type          |
|------------|----------|--------|--------------------|
| Layer 1    | 0.0      | 15.2   | Loose sand fill    |
| Layer 2A   | 15.2     | 24.6   | Soft marine clay (upper) |
| Layer 3    | 24.6     | 27.6   | Medium stiff silty clay |
| Layer 2B   | 27.6     | 31.4   | Soft marine clay (lower) |
| Layer 5A   | 31.4     | 35.8   | Very stiff silty clay |
| Layer 5B   | 35.8     | 39.4   | Dense silty sand   |
| Layer 6    | 39.4     | 49.6   | Very dense silty sand |

Fig. 6 Rate of pile top settlement for A2-UTP02

3.4 Effects of construction method

All the piles in MBFC were constructed under water instead of dry pile method due to the geological formation of the site. The piles were deep, ranging from 50m to 80m and the construction time for one pile is from 18hours to 48hours depends on the type of stabilising fluid is used.

Long temporary casings, typically 20 to 30m depend on thickness of sand layer, were required to protect the loose sand layer when polymer is used as stabilising fluid. However, only short casing from 6 to 10m is required if bentonite is adopted to stabilise the loose to medium dense sand layer, very soft to soft marine clay layer and soft to medium stiff clay and sand layer. The reason that bentonite slurry can stabilise and prevent the loose to medium dense layer (from toe of the temporary casing up to the top of the very soft marine clay) from collapse during boring operation is because the loose to medium dense sand layer is highly permeable. Bentonite suspensions can seal the hole and provide the gel strength required to move the solids out of the hole. In other words, it would form a filter cake on those soil layers that are highly permeable to prevent loose to medium dense sand layer from collapsing. Therefore, adopting bentonite as stabilising fluid would have the advantages, such as short casing (6 to 10m) instead of long casing (20 to 30m) for polymer; no joining and welding of casings are required for each individual piles; smaller machine / crane is needed; no vibrator hammer or only smaller capacity of vibrator hammer (eg. 5tons hammer) is needed for installation and extraction of casings and minimal vibration. This is essential for this site as big portion of MBFC is located at MRT railway reserves zone and CST 6m protection zone.

These were the main reasons that driven the ultimate load tests for A2-UTP02, A4-UTP01 & A4-UTP01 (Additional) to be carried out using bentonite. In the initial construction stage, 232 numbers of working piles were constructed using polymer and it required lengthy construction time of four months to complete. The purpose of carrying out the three load tests using bentonite was to ascertain that the piles performance would not be compromised by adopting bentonite slurry. The successful of this approach would ultimately shorten the construction period. Total average duration required to install a 70meter deep foundation bored piles of diameter 1800mm with polymer and bentonite slurry is shown in Table 4. It shows that the duration was about 30 percent shorter for piles constructed with bentonite slurry comparing with polymer slurry.

Table 4. Average duration to construct a 70m deep and diameter 1800mm bored pile at MBFC with polymer and bentonite suspensions

| Stabilising fluid | Install operation | Lower Lower | Flushing (join and weld) | Casing pipe to toe, if not >200mm | Concreting | Extract Total casing |
|-------------------|-------------------|-------------|--------------------------|----------------------------------|------------|---------------------|
| 1st casing | 1st casing | 2nd casing | 2nd casing | 1st casing | 2nd casing | 1st casing | 2nd casing | 3rd casing |
| 1.0hr | 0.5hr | >200mm | 0.5hr |
| Polymer          | 1.0hr | 5.0hr | 12.0hr | 3.0hr | 1.5hr | 0.5hr | 6.0hr | 1.5hr | 30.5hr |
| Bentonite        | 1.0hr | 0 | 12.0hr | 3.0hr | 1.5hr | 0 | 6.0hr | 0.5hr | 24.0hr |

However, bentonite suspension do have disadvantage. The filter cake formed around the pile shaft would affect the skin friction. This was proven in the ultimate load test A4-UTP01. The fully mobilised skin frictions for this pile were only 30kN/m² for old alluvium (soil layers above layer 6) and 229kN/m² for layer 6. It was also need to note that the bore hole with bentonite suspensions was untouched for at least 24hours when the boring was only few meters to reach the toe level. This was due to boring rig faulty during
the installation. It was concluded that the thick filter cake was formed during the stoppage period and caused the lower mobilised skin friction. In order to verify this finding, another ultimate load test A4-UTP01 (Additional) was installed and tested at 6m away from A4-UTP01. The pile was constructed in continuous sequences without any disturbance. The mobilised skin friction for this pile is much higher than the previous one. The mobilised skin frictions were 233kN/m² and 286kN/m² for layer 5 and layer 6, respectively. The load transfer curves for both A4-UTP01 can be found in Fig. 1.

4 A STUDY ON THE DESIGN PARAMETERS FOR BORED PILE IN VARIOUS FORMATIONS

A study on the mobilised geotechnical parameters from the instrumented test piles for other sites in Singapore has been carried out. The results comprise of the majority of the geotechnical formations in Singapore. It includes Bukit Timah Formation (igneous rock), Jurong Formation (sedimentary rock) and Old Alluvium.

4.1 Bukit Timah Formation

There are eight instrumented tests for bored piles in Bukit Timah Formation have been analysed and the results are summarised in Table 5. All the piles had not fully mobilised in both skin friction and end bearing, except for the iULT 1 at Tagore Lane. The skin friction is fully mobilised at 350kN/m² for Grade IV granite. The maximum mobilised skin friction for Grade III granite is from 340 to 398kN/m² for non-base grouted piles. The mobilised skin friction can be increased to as high as 1455kN/m² for base grouted piles at the section of 1.7D from the pile toe. This finding is consistent with the study discussed in Section 3.2. As for Grade V granite, the mobilised skin friction can be 249kN/m².

The maximum mobilised end bearing is at 9948kN/m² among all the piles for Grade III granite at Woodlands T203. However, it is to note that this is not the ultimate value.

4.2 Jurong Formation

The results for the total eleven instrumented tests for bored piles in Jurong Formation are presented in Table 6. The mobilised skin friction for S VI, residual soil with SPT N = 100 is from 196 to 537kN/m². The mobilised skin friction for S V, completely weathered siltstone, is from 29 to 450kN/m². The mobilised skin friction for S IV, highly weathered siltstone, is from 252 to 415kN/m². Lastly, the mobilised skin friction for S III, moderately weathered siltstone, is from 441 to 665kN/m². The maximum mobilised end bearing for S V, S IV and S III is 5800, 8756 and 9774kN/m², respectively.

The results show that all the piles had not fully mobilised in the skin friction and end bearing. It shows that the mobilised skin friction for the upper section is higher than the lower section of the pile for L7 – 300, L4 – 300, L3 – 300 and ULT – 1100. This is because the design parameters adopted for the preliminary piles design is far too conservative. The test load is majority supported by the upper section of the pile. This is also explained why the mobilised end bearing for L6 – 300, L7 – 300, L5 – 300, L4 – 300, L3 – 300 and ULT – 600 are recorded from 107 to 1504kN/m² only.

| Test ID – Pile Size (mm) / Location / Rock Type | Base grouting | Maximum Mobilised skin friction (kN/m²) / Rock Grade | Maximum Mobilised end bearing (kN/m²) / Rock Grade |
|-----------------------------------------------|---------------|------------------------------------------------------|---------------------------------------------------|
| PTP 1 – 1500 / Woodlands T203 / Granite       | Yes           | 886 – G III (within 1.7D from pile toe)              | 9948 – G III                                      |
| PTP 2 – 1200 / Woodlands T203 / Granite       | Yes           | 1038 – G III (within 1.7D from pile toe)             | 1980 – G III                                      |
| PTP 3 – 1200 / Woodlands T203 / Granite       | Yes           | 1455 – G III (within 1.7D from pile toe)             | 513 – G III                                       |
| ULT 1 - 900 / Jalan Dusun / Granite           | No            | 374 – G III                                         | 4926 – G III                                     |
| ULT 2 – 900 / Jalan Dusun / Granite           | No            | 398 – G III                                         | 5950 – G III                                     |
| iULT 1 - 800 # / Tagore Lane / Granite        | No            | 350 – G IV                                          | 8413 – G IV                                      |
| iULT 2 - 800 / Tagore Lane / Granite          | No            | 340 – G III                                         | 650 – G III                                      |
| ULT (O-Cell) - 1000 / Thomson Road / Granite | No            | 259 – G V or SPT N 60                                | 8216 – G V or SPT N 60                            |

#: Skin friction had been fully mobilised

4.3 Old Alluvium Formation

There are seven instrumented tests for bored piles in Old Alluvium Formation have been analysed and the results are summarised in Table 7. All the piles had not fully mobilised in both skin friction and end bearing. The mobilised skin friction for SPT N < 30 Silty Sand (OA) is at 6.4N to 14.9N. The mobilised skin friction for SPT (30 < N < 50) OA is at 0.5N to 2.5N. The mobilised skin friction for SPT (50 < N < 100) OA is at 2.6N to 6.1N. The mobilised skin friction for SPT N = 100 OA is at 1.7N to 3.9N. The maximum mobilised end bearing for SPT N = 65 OA is at 6165kN/m² and SPT N = 100 OA is at 5020kN/m².

The results again show that the design parameters adopted in the preliminary piles design are too conservative. It leads to the lower mobilised skin friction for the lower section of the pile as compared with the upper section. It also causes the small mobilisation of end bearing at pile toe.
Table 6. Maximum mobilised parameters for the instrumented ultimate test piles in Jurong Formation

| Test ID – Pile Size (mm) / Location / Rock Type | Base grouting | Maximum Mobilised skin friction (kN/m²) - Rock Grade | Maximum Mobilised end bearing (kN/m²) - Rock Grade |
|-----------------------------------------------|---------------|------------------------------------------------------|--------------------------------------------------|
| ULT – 800 / Bukit Batok East / Siltstone      | No            | 326 – S IV                                           | 9774 – S III                                    |
| L1 - 300 Boon Lay Way / Siltstone             | No            | 237 – S V                                           | 5800 – S V                                      |
| L2 - 300 Boon Lay Way / Siltstone             | No            | 340 – S IV                                           | 8473 – S IV                                      |
| L6 - 300 Boon Lay Way / Siltstone             | No            | 313 – S IV                                           | 595 – S IV                                       |
| L7 - 300 Boon Lay Way / Siltstone             | No            | 300 – S VI (SPT – N 100)                             | 404 – S V                                        |
| L5 - 300 Boon Lay Way / Siltstone             | No            | 415 – S IV                                           | 1012 – S IV                                     |
| L8 - 300 Boon Lay Way / Siltstone             | No            | 252 – S IV                                           | 8756 – S IV                                     |
| L4 - 300 Boon Lay Way / Siltstone             | No            | 537 – S VI (SPT – N 100)                             | 169 – S V                                        |
| L3 - 300 Boon Lay Way / Siltstone             | No            | 196 – S VI (SPT – N 100)                             | 107 – S V                                        |
| ULT – 900 Jalan Bukit Merah / Claystone       | No            | 450 – S V                                           | Not available                                    |
| ULT – 1100 Pandan Avenue / Siltstone          | No            | 555 – S III                                          | 4644 – S III                                    |
| ULT – 600 Pandan Road / Siltstone             | No            | 441 – S III                                          | 1504 – S III                                    |

5 CONCLUSIONS

The effects of base grouting and improved soil layer for the performance of the deep foundation bored piles are significant and it cannot be ignored. Pile design could be more effective and innovative in considering the contribution of the base grouting. It could help in reduce the unnecessary pile penetration length into the ground. Also, the contribution of the improved soil layer on the pile must be considered in the design.

The usage of stabilising fluid in deep foundation construction is unavoidable. The adoption of the type of stabilising fluid must be carefully evaluated and the construction history of the pile must be recorded and analysed.

The current preliminary piles design needs to be changed to a more realistic approach with appropriate design geotechnical parameters, i.e. ultimate skin friction and ultimate end bearing. It is shown in Section 4 that the current design is too conservative and the skin friction and end bearing are not substantially mobilised in the ultimate load tests for the three major geological formations in Singapore.

Table 7. Maximum mobilised parameters for the instrumented ultimate test piles in Old Alluvium

| Test ID – Pile Size (mm) / Location / Soil Type | Base grouting | Maximum Mobilised skin friction (kN/m²) - SPT N | Maximum Mobilised end bearing (kN/m²) - SPT N |
|-----------------------------------------------|---------------|--------------------------------------------------|--------------------------------------------------|
| ULT 1 – 800 / Pasir Ris Dr 3 / Silty Sand      | No            | 298 – N = 20                                      | 82 – N = 52                                      |
| (f_s = 1.7N)                                  | (f_s = 2.5N)  | 161 – N = 25                                      | 3063 – N = 100                                   |
| (f_s = 3.8N)                                  | (f_s = 6.4N)  | 25 – N = 48                                       | (f_s = 0.5N)                                    |
| ULT 2 – 800 / Pasir Ris Dr 3 / Silty Sand      | No            | 257 – N = 34                                      | 634 – N = 50                                     |
| (f_s = 7.9N)                                  | (f_s = 0.9N)  | 45 – N = 50                                       | (f_s = 0.9N)                                    |
| ULT 3 – 800 / Pasir Ris Dr 3 / Silty Sand      | No            | 355 – N = 70                                      | 1575 – N = 88                                    |
| (f_s = 5.1N)                                  | (f_s = 2.6N)  | 232 – N = 88                                       | (f_s = 3.9N)                                    |
| ULT - 800 Lor 6 Toa Payoh / Silty Sand         | No            | 392 – N = 100                                     | 6165 – N = 65                                    |
| (O-Cell)                                      | (f_s = 6.1N)  | 234 – N = 86                                       | (f_s = 6.1N)                                    |

REFERENCES

1) Singapore Standard, CP 4:2003 Code of Practice for Foundations.