Local Construction Practice and Geotechnical Performance of Rock Socketed Bored Pile in Sedimentary Crocker Formation in Sabah

C.J. Wong[1] and S.S. Liew[2]
[1]*CJ Perunding, Kota Kinabalu, Malaysia*
[2]*G&P Geotechnics Sdn Bhd, Kuala Lumpur, Malaysia*
E-mail: jack_wcj@yahoo.com

ABSTRACT: Bored cast-in-situ concrete pile with rock socket has become a popular foundation option to support highly loaded structure. However, unlike drive-in or jack-in piles, the evaluation of pile performance on both capacity and deformation of this pile type can be very subjective when it comes to deciding the required pile length or rock socket length during construction. Dispute on the technical requirements and, more often, on contractual issues arise during construction against the ideally designed cases. This paper presents the authors' experience in the bored pile construction practice in the local founding formation, namely Crocker Formation, along the west coast of Sabah. Review of the geotechnical performance of bored piles socketed into this sedimentary rock formation with different weathering condition were performed on the fully instrumented test pile using Global Strain Extensometer technology.

KEYWORDS: Bored Pile, Local Construction Practice, Geotechnical Performance, Sedimentary Crocker Formation

1. INTRODUCTION

Bored cast-in-situ pile with rock socket into competent bearing stratum has become a popular foundation option to support the highly loaded structures. Bored piling is a type of replacement pile, in which the soils and rocks are bored out and infill with reinforced concrete into the bored hole. The allowable compression structural capacity of bored pile is normally the permissible stress derived from the concrete strength (negligible contribution from steel bars), in which the permissible compressive stress is limited to not greater than 25% of the concrete characteristic strength in accordance with the British Standard, BS8004. However, the structural capacity of the bored pile is seldom a concern when there is proper quality control and assurance of the concrete supply and concreting work. However, the main challenge of bored pile construction is the estimation and verification of the geotechnical pile capacity from the bonded pile shaft and the pile base bearing contact. Unlike drive-in or jack in piles, it is very subjective when it comes to deciding the required pile length and socket length. To achieve the necessary geotechnical capacity, bored pile is normally required to be socketed into the competent stiff residual soils or bedrock for high shaft friction between the concrete and rock mass and high end bearing when desired. However, end bearing is often ignored due to soft toe effect unless pile base cleaning or pile base stiffening treatment with the verification test is provided. The commencement level of the load deriving rock socket length has often been the argument to fulfil the technical and the contractual requirements, especially the underlain bedrock are inter-layered sedimentary rocks with different weathered grades as in Sabah.

2. GENERAL GEOLOGY AND BEDROCK PROFILE

Crocker formation is generally sedimentary rock formed during the Oligocene Epoch, which consists of flysch-type sandstone, shale, siltstone with tuff, limestone breccia and agglomerate. Due to intense weathering process of tropical climate and regional tectonic actions, these sedimentary rocks have already been subjected to physical and chemical weakening processes in the engineering characteristics, particularly at the upper stratum.

At west coast of Sabah, it is noticeable that most exposed rock slopes show evidences with folding of sub-vertical bedding of sedimentary rocks, which tallies with the geological history of West Crocker formation after experiencing the NW Borneo fold-trust belt (see Figure 1). Therefore, it is unwise to interpolate the bedrock profile based on simple interpretation of material layering from the exploratory boreholes. As the geotechnical performance of the bored pile varies in the different types of sedimentary rocks (i.e. Sandstones and Shale), the design should have taken consideration of the oriented bedding sequence.

Figure 1 Exposed Rock Slope with Sub-vertical Bedding

3. LOCAL PRACTICE AND PROBLEMS

Bored pile construction is not new to the local geotechnical industry, as there are plenty of slope remedial works adopting contiguous bored pile as retaining structure to reconstruct the road platform or as slope stabilisation for the past 30 years. It is not common as a building foundation system until the recent decade of the local property boom, in which the requirement of the foundation bored piles is then more emphasized on the high axial capacity. Therefore, the local geotechnical data for foundation bored pile design is limited. The termination criteria are normally based on the minimum socket length into the hard shallow bedrock, especially in Kota Kinabalu, the Capital City of Sabah, to obtain high geotechnical capacity. This paper will present and share some issues from the several bored pile construction jobs that the authors have involved directly or indirectly.

3.1 Definition of Rock

One of the issues is the over-simplified definition of rock for the socket length, in which, most of the time, only "competent rock" or "hard rock" is specified. As a result, these commonly used term is often subject to dispute when it comes to the interpretation during actual construction. Issues can arise when the encountered rock does not tally with the specified rock and even the method to verify the specified rock quality can be disputable.

In addition to the poorly defined rock, site supervising personnel for a building construction contract with little experience in substructure works will also add on the difficulty in work execution technically and contractually. As such, the decision on the termination of each boring work is often delayed as engineer's representative is unable to provide timely decision. On the other hand, site supervising personnel usually relies heavily on the judgment by the boring machine operator to determine the commencement level of rock socketing, i.e. the site supervising personnel was notified by the operator on commencement of rock socketing rather than making judgment or decision himself.

3.2 Neglected Highly Weathered Rock or Soft Rock

It is not uncommon to encounter thick highly to completely weathered rock mass or soft rock at upper level prior to reaching the anticipated competent "hard rock" at lower depth. This relatively weaker layer is fairly easy to be bored through especially when the powerful boring machine is used. As there is practical difficulty in verifying or justifying the encountered weak material as "rock", it is often simply neglected for its contribution to the geotechnical pile capacity by the pile designer who has little experience in bored pile construction. For the area where there is thick weathered rock mass overlying the competent rock, it would be overly conservative to ignore its contribution to the pile capacity. Though this is not a normal design practice in other parts of Malaysia, there is still a design preference in Sabah to have the pile resistance primarily derived from the unweathered rock socket in Sabah.

Conversely, lack of understanding of the actual load transfer mechanism for the bored pile can lead to overestimating the pile capacity. When the conventional uninstrumented static load test results with a constructed socket length as designed indicate satisfactory overall pile performance, the design assumptions are deemed to have been verified. With that, the verified design rock socket friction is normally back calculated by distributing the achieved pile capacity averagely over the recorded rock socket length only and totally neglecting the possible high resistance contribution from the hard layer (say SPT-N greater than 50) and highly weathered rock of significant thickness above the relatively unweathered rock socket. When it comes to situation with shorter overall pile length due to shallow rock formation or thinner soft rock, the overall capacity of the bored pile could be overestimated, thus resulting in lower safety margin if the required rock socket length is not adjusted accordingly.

4. COMMENCEMENT OF ROCK SOCKET LENGTH

Termination criterion of a bored pile is the key design information to be conveyed to the construction site team, in which the required bored pile length or the rock socket length shall be clearly specified. As discussed earlier, in most of the cases, minimum required socket length is specified and therefore the criterion of the rock socket quality to be considered as the commencement of rock socket is crucial. It is important to make the requirements clear in the tender to avoid contractual dispute and to demand the site personnel ensuring that it is constructed according to the designer's needs. Followings are some of the typical methods by the local piling industry that the first author has come across in deciding the commencement of the socket length:

- Effort of the Machineries and Tools
- Standard Penetration Test (SPT) in Bored Hole
- Strength test on the Core Samples
- Explicit Subsurface Investigation and Full Time Supervision by Engineer or Geologist

4.1 Effort of the Machineries and Tools

There are some piling contracts requiring the contractor to justify the classification of rock encountered for rock socketing based on the type and model of their boring machine. As the rock coring effort of one machine has indicative correlation to the rock quality and strength, thus the pile geotechnical performance, this allows the consistency of rock socket quality using the same calibrated coring tools. Besides that, the changing of rock coring tools when soil boring tools cannot proceed further in depth will also indicate the encountering of rock that requires increased effort in coring into harder materials.

However, in a relatively small local piling market, specifying the required type and model of machineries will reduce the intended competition during the tender. Besides that, for project site with large quantity of bored piles with different pile sizes, it is unlikely for the contractor to provide the boring rigs of single capacity to cope with the necessary pile production. When it comes to different sizes of bored pile, the drilling effort for a smaller bored pile is less than the bored pile with larger diameter. Therefore, relying on the effort of the boring machine can be very subjective, but nonetheless still a practical approach in some cases.

Most importantly, the decision of the commencement of rock socket will rely on the machine operator who might have different interest than the project client and designer. Therefore, this is not the preferred option by the authors but rather a useful supplementary reference during construction.

4.2 Standard Penetration Test (SPT) in Bored Hole

Some contracts have allowed SPT to be carried out for each pile point when thick hard stratum or weathered rock mass is expected for confirming the commencement of socketing in the competent load bearing hard layer or weathered rock mass. Depending on the design requirements, if the bored pile is expected to be socketed into thick hard stratum, the commencement of socket length is typically considered when SPT N of 50 or greater. However, it can also be used to determine if rock is encountered as the split spoon rebounds during the SPT.

Verification using SPT is useful and straight forward. However, there is limitation when it comes to distinguishing the rock quality without physical sample recovery. Therefore, a more conservative value has to be adopted for assessing the ultimate shaft friction. Besides that, other than the cost of the test, the time cost should also be considered. If SPT-N profile of the founding materials is available, this method can be used to assess the ultimate shaft friction reasonable using modified Meyerhof method. However, when extrapolated SPT-N values beyond 50, the assessed ultimate shaft friction will have to be limited to a threshold value as excessive extrapolation will reduce reliability of the evaluated ultimate shaft friction. As a result, this can lead to conservative estimate sometimes.

4.3 Strength Test on the Core Samples

Strength test on the recovered core samples is a more common method in deciding of the rock socket strength. It is generally believed that the strength of the rock has direct correlation with the shaft friction between the bored pile and the rock. However, it is important to establish the correlation between the rock strength and the rock socket capacity. There are several tests the first author has encountered in the local bored pile construction.

4.3.1 Unconfined Compression Test

Unconfined compression test is a very common test to determine the strength of the particular rock core sample. However, the recovery of cores from boreholes or preparation of the rock core sample from the recovered sizeable rock fragments during boring operation are time consuming and the test equipment is not normally available at construction site and therefore requiring testing in the laboratory. This test is probably not a viable option during pile construction, in which the site will require immediate decision or else idling cost pending for decision may kick in. Furthermore there is always perception that the test results may represent the overly optimistic intact rock strength rather than the actual jointed rock mass strength encountered at site, unless correlation between pile test performance results and the intact rock strength is established.

Figure 2 Unconfined Compression Strength Test

4.3.2 Point Load Test

Point load test is a simpler test as compared to the unconfined compression strength test. The test is often adopted at site to estimate the intact rock strength due to the cost and size of test equipment. The only slight disadvantage of the test is the need for statistical correlation with the UCS.

$$UCS = k\, I_{s(50)} \tag{1}$$

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

In view of the limited study and data compilation on the correlation locally, the typical conversion factor of 24 from Bieniawski (1975) is often adopted locally. Another published conversion factor for meta-sedimentary formation in Peninsular Malaysia by Liew, et al (2011) shows a mean value of 13.3 with a wide range of value from 5.6 to 21.0. This implies that different formations or rock type will likely have different correlation value or factor. Besides that, it is also noted the high variation of the correlation factor, probably due to the inherent heterogeneity in cementation strength of different directions, particularly the sedimentary formation or its further derivatives after partially metamorphism, where the sedimentation relict structures are not completed diminished.

Figure 3 Point Load Test

4.3.3 Rebound Hammer Test

Rebound hammer test is a rare method but similar concept as the point load test. Rebound hammer, also known as the Schmidt hammer, is best known of its purpose to estimate the in-situ concrete strength. The test can be performed on the collected rock samples from the coring works immediately. Similarly, it is crucial to carry out correlation on the obtained indirect intact rock strength with the pile geotechnical capacity.

Figure 4 Rebound Hammer Test

Despite of the effectiveness in deciding on the rock quality, there is also weakness in this method if there is lack of supervision. As mentioned, Crocker Formation is layered sedimentary rocks which generally consists of flysch-type sandstone, shale and siltstone. These layered sedimentary rocks could have different rock strength when tested at different directions. During coring work, it is common that only sizable rocks selected for the strength tests. The laminated sedimentary rock mass with joints with practically RQD value of close of 0% in core recovery, where representative rock samples cannot be obtained from the cored rock fragments, thus sometimes leading to no testing. Unless massive cemented rocks are encountered, the test results also represent potentially optimistic intact strength without considering the joints and lamination weakness in sedimentary rocks. Therefore, when there is lack of attention from site supervisory team in obtaining representative strength value, the tests results can be potentially misleading. In all the methods of obtaining intact rock strength, correlation with test pile performance and also consideration of the discontinuity features in rock mass shall be dully established and accounted for respectively.

4.4 Explicit Subsurface Investigation with Full Time Supervision by Engineer or Geologist

In the recent projects handled by the authors, extensive subsurface investigation has been implemented to have better understanding of the rock profile for the site under a full time supervision by a trained site engineer. The preliminary test pile was then constructed at the designated location where the subsurface investigation (SI) borehole was carried out. The coring effort, recovered rock fragments are compared with the core samples from SI information. With the foundation layout adequately covered with the SI boreholes, the termination of the bored pile can then be referred to the nearby SI reference boreholes.

However, with this method, it is important to have a full time supervising engineer or engineering geologist to witness important features consisting of bedding orientation, thickness and joint fractures of the sedimentary sequences, etc, and verifying the consistency of materials encountered in the boring of the test pile. It is important to determine the material types of the load bearing stratum and, with the correlated strength of the materials encountered, assess its corresponding ultimate resistance interpreted from the instrumented pile load test. The same approach is adopted for working pile production.

5. GEOTECHNICAL PERFORMANCE

Several instrumented pile load test results using the Global Strain Extensometer technology to reveal the load transfer behaviour have been executed and compiled for the local Crocker formation. However, this paper will elaborate the test results of one (1) instrumented 1200mm diameter grade G40 test pile, with the designed working load of 10500kN, planned and supervised by the authors.

Prior to preliminary design, detailed SI was carried out. A sacrificial test pile with full instrumentation was then selected at the location among the SI boreholes for the best representation for the project site. Four nos. of 68mm steel pipes were installed at the test pile reinforcement steel and the Global Strain Extensometers were inserted and anchored at designated levels within these four pipes several days after the concreting prior to the load test setup. Unlike the conventional local strain gauges, the post concrete installation has prevented the potential installation damage of the strain gauges. Besides that, the global strain readings will also provide more representative average strain over the interest pile segments as compared to measuring the highly varying localised strain over a gauge length normally not more than 100mm.

The test pile was socketed into Sandstone occasionally with interbedded Shale. The pile was loaded to 2.5 times of its working capacity, yet it was not sufficient to fail the pile. However, the load test to 2.5 times the working load has provided sufficient informative results on the mobilized pile shaft and rock socket frictions for the subsequent design work. As bored piling work relies heavily on the site judgment, the load bearing rock strata have been classified into two (2) layers to ease the supervision personnel. The first layer of rock, namely “soft rock”, is generally highly weathered rock while the 2nd layer of rock, “hard rock”, is rock with less weathered condition. The classification of these two material types can be visually judged and assisted by the on-site strength tests and also boring/coring effort as mentioned above.

5.1 Subsurface Information from SI Borehole

The SI boreholes shows the overburden soil with SPT N value of about 10 down to a depth of 15.0m below ground level (mbgl). From 15.0mbgl to 22.5mbgl, highly weathered sandstone was encountered with SPT N value generally greater than 50 and the samples recovered from triple tube core barrel between the SPT shows RQD profile of 0%. From 22.5mbgl downward, fresh massive sandstone with RQD ranging 30% to 86% was encountered. A correlation between the point load strength index and the UCS has been established with the mean value of 20.9 from the range of 2.5 to 82.2, indicating the inherent large variation of the conversion factor. Based on the SI information, a preliminary assumption on the average allowable shaft friction of 300kPa was adopted to decide on the test pile length, after considering that the chances of fully mobilising the high RQD rock beyond 22.5mbgl is low.

5.2 Construction of Test Pile

The test pile was constructed next to a SI reference borehole using hydraulic rotary boring rig. During the boring work, yellowish Silty SAND or Sandy SILT were observed until about 16.5mbgl. From 16.5mbgl to 19.25mbgl, soft greyish fragmented Sandstone with thin Shale bedding was encountered. The tested point load strength indexes were 0.08MPa and 0.3MPa and the recorded boring time of 38mins for the 3m coring. From 19.25mbgl until 24.7mbgl (end of bored pile), hard to very hard greyish Sandstone with Shale fragment was observed with the recorded point load strength index ranging from 1.9MPa to 3.77MPa. The boring time from 19.25mbgl to 21.2mbgl was about 30 mins before changing from soil boring tools (soil auger and boring bucket) to rock core barrel. Subsequently, it took about 115 mins to complete the remaining 3.5m. During the boring/coring works, a 12.5m long temporary steel casing was used to stabilise the overburden soil in bored hole without using other stabilizing fluid. There was insignificant seepage in the rock mass, thus the coring of the rock socket was mostly in dry hole condition.

5.3 Interpretation of Test Result

The load test was carried in 3 loading cycles, which were to 100%, 200% and 250% of the designed working load. Apart from assessing its ultimate capacity, it was also intended to verify the performance of the test pile at service load. As the arrangement of the global strain gauges were based on the boring/coring record and also the SI information, the upper soil stratum was separated into two layers, in which overburden soils showed mobilised shaft friction of 45.6kPa and 57.9kPa at 2.5 times working load. For the soft rock of about 3m thick, the interpreted mobilised shaft friction was 294kPa. Over the first part of the hard rock, the interpreted mobilised shaft friction was 930kPa while the second part was in the range of 710kPa to 1315kPa. The construction record and the test result was summarised in Table 1 for easy reference.

The interpreted pile test results show that the soil layers from ground level to Level B and Level B to Level C have shown sign of mobilising the ultimate shaft resistance at the displacement of 12mm and 11mm thereafter respectively. While for the soft rock layer between Level C to Level D, the shaft friction has achieved its maximum resistance at 8mm and yet no yielding is shown. However, from Level D downwards, where the pile socket embedded into the defined hard rock, maximum rock socket friction was mobilised with socket movement of 7.7mm to 8.7mm even until reaching the intended maximum test load. From the linearity of the load transfer curve, it is believed that there are still room for mobilising higher rock socket resistance if higher test load can be applied. Figure 5 shows the mobilised pile shaft movement of each respective pile segments in the 3 loading cycles.

Table 1 Summary of Construction Record Vs. Performance

Depth/ Level (mbgl)	Boring / Coring Time (mins)	Point Load Strength Index (MPa)	Mobilised Shaft Friction (kPa)
GL – 9.0m (GL – Lev B)	-	-	45.6
9.0m – 16.5m (Lev B – Lev C)	-	-	57.9
16.5 – 19.25 (Lev C – Lev D)	38	0.08 – 0.30	294
19.25 – 21.0 (Lev D – Lev E)	30	2.31	930
21.0 – 22.5 (Lev E – Lev F)	25	2.64	1315
22.5 – 24.0 (Lev F – Lev G)	60	1.90	710
24.0 – 24.7 (Lev G – Toe)	30	3.77	-

(Note: Change of soil boring tools to rock coring tool at 21.2mbgl)

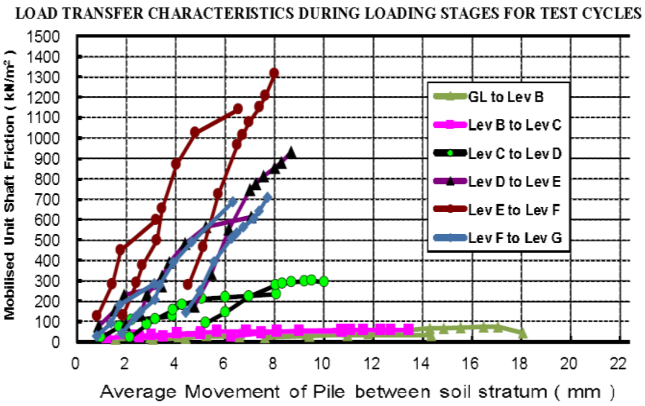

Figure 5 Mobilised Unit Shaft Frictions

6. RECOMMENDATIONS

Based on the Authors’ experience with the local construction practice in rock socketed bored pile as the building foundation, the following are some recommendations for the design and construction of bored pile:

- Understand the geological setting of the site
- Carry out sufficient exploratory boreholes for good reference of representative ground model during the design stage and later use for pile construction validation
- Carry out instrumented static load test (preferably on a sacrificial pile) for detailed verification of the design parameters/assumptions to establish local bored pile design database, which is fairly lacking at the present stage.
- Full time supervision by site engineer/engineering geologist who has relevant experience. Communication between the site personnel and the designer is crucial to deliver the desired requirement/criteria to the construction site.
- More load tests on working piles for further verification of the adopted design parameters and also improving site judgement.
- Clear specification of termination criteria, rock socket definition with respect to the rock quality by quantitative measurement that is relevant to the actual site condition at tender stage to avoid dispute during construction.

7. CONCLUSION

This paper presents the local construction practice of the rock socketed bored pile in Sedimentary Crocker Formation in Sabah which highlights the limited geotechnical data of the bored pile performance. Although there are limitations for all methods used to determine the rock quality for the commencement of rock socket and contractual length of rock socket, an experienced site personnel will be able to make full use of the information/reference for timely decision making in pile construction. This paper also presents the performance of a fully instrumented bored pile with comparison of the construction records. From cross reference of the instrumented test results and the construction records, it shows that highly weathered rock or "soft rock" can practically contribute to the overall pile geotechnical capacity.

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8. REFERENCES

British Standard Institution, BS8004: *Code of Practice for Foundations*

Bieniawski, Z. T., (1975), The point load test in geotechnical practice, Eng. Geol., 9, 1-11.

Hanifah, A. A. and Lee, S. K., (2006), Application of Global Strain Extensometer (Glostrext) Method for Instrumented Bored Piles in Malaysia, *10th International Conference on Piling and Deep Foundations*, Amsterdam, June 2006

Liew, S.S., Khoo, C.M., Tan, S.T. & Loh, Y.E., (2011), Performance Review of Bored Pile Foundation & Basement Excavation in Meta-Sedimentary Formation in Kuala Lumpur, *14th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Hong Kong.

Tan, Y.C. & Chow, C.M., (2003) Design & Construction of Bored Pile Foundation, *Geotechnical Course for Pile Foundation Design & Construction*, Ipoh 29 – 30 September 2003