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ABSTRACT: This paper presents a case study of caisson foundation system for a high-rise residential building in Kuala Lumpur granitic formation. Due to site congestion and sloping terrain of the project site, hand-dug caisson was adopted as the deep foundation system rather than conventional bored pile or driven pile systems. This paper discusses the design aspects of caisson pile especially on the assessment of rock socket which involves estimation of both shaft and base resistances for rock and their load transfer behaviour. Checking of concrete lining and some interesting findings on the belled-out base caisson is briefly discussed here.

The performance of the caisson pile was verified with two numbers of static maintained load tests with test load up to 1,700 tonnes and 3,000 tonnes (instrumented test pile) respectively, and one plate bearing test at the excavated caisson base. Generally, the caisson pile performance was better than prediction due to the inherent rough and irregular rock shaft surface formed by mechanical hacking and the advantage of proper rock socket cleaning prior to concreting compared to conventional bored pile construction.

This paper also presents typical construction sequence together with some good practices on 360° rock condition mapping and, safety and health of the workers during construction.

1. INTRODUCTION

The site is located at Kuala Lumpur with development area of 2.34 acres. The development comprises 3 blocks of high-rise residential building with about 20 storeys of service apartments, ground floor retailer shops and elevated car parks respectively.

The site is underlain by granitic formation with depth of bedrock ranging from 1m to 20m below ground. For area with shallow bedrock, footing was adopted as building foundation system for practical and economy reasons (refer to Fig. 1). Hand-dug caisson was adopted as the deep foundation system due to the existing site condition of partially sloping terrain as the boring rigs or jack-in machine of the conventional foundation piling system require relative flat and large platform for the pile installation. The limited space of the site also restrained the use of the large piling machine.

Caisson pile construction involves hand digging of earth materials and rock hacking using portable tools. The staged excavation is temporarily retained by cast in-situ circular concrete linings before subsequent lift excavation until the founding level. Concrete lining is unnecessary in bedrock and stable stiff soil with reasonably dry condition when water table is below excavation. Proper cleaning and checking on the completed digging hole would be carried out before lowering of steel cage and concreting.

2. DESIGN OF CAISSON PILE

2.1 Rock Socket Design

During the soil excavation in caisson pile, concrete lining would normally be cast in-situ to temporarily support the excavated soil surface. However, the quality of cast concrete is difficult to be controlled due to the local caisson construction norm of only using hand tapping for compaction after concreting to avoid dislodging of the surrounding soil. Hence, the contribution of pile shaft resistance in the soil would be uncertain and is usually ignored in the pile capacity design unless proper site verification test is carried out.

The geotechnical capacity of the caisson pile is primarily derived from both the shaft and base resistances of the founding materials. Unlike conventional bored pile usually ignoring the base resistance due to unsatisfactory base cleaning, caisson pile under dry hole construction can easily achieve satisfactory base cleaning and allow inspection of rock socketting condition before concreting.

Eq. (1) with design approach established by Rosenberg & Journeaux (1978), Horvath (1978) and Williams & Pells (1981) was adopted to estimate the rock socket resistance with consideration of the respective strengths of intact rock and rock mass effect in association with the inherent discontinuities.

\[
f_{s(all)} = \alpha \beta q_{uc}
\]

\[
f_{s(ult)} = \text{Ultimate shaft resistance of rock}
\]

\[
\alpha = \text{Reduction factor with respect to } q_{uc} \text{ (Fig. 2)}
\]

\[
\beta = \text{Reduction factor with respect to the rock mass effect (Fig. 3)}
\]

\[
q_{uc} = \text{Unconfined compressive strength of intact rock}
\]

Note: Factor of safety (FOS) of 2.5 implying virtually zero risk of failure is often applied on Eq. (1) after Williams & Pells (1981)

Based on the approach by Williams & Pells (1981), the allowable rock shaft resistance \(f_{s(all)}\) with FOS of 2.5 shall be in the range of 3% to 5% of unconfined compressive strength of intact rock \(q_{uc}\) for \(q_{uc}\) more than 10MPa and Rock Quality Designation (RQD) exceeding 75%.
The allowable rock base resistance was estimated from the empirical correlation considering the spacing of discontinuities of founding rock as shown in Eq. (2) as recommended by Canadian Foundation Engineering Manual (1992).

\[ f_{b\ (al)} = K_{sp} \cdot q_{uc} \]  

(2)

\[ f_{b\ (al)} = \text{Allowable base resistance of rock} \]

\[ q_{uc} = \text{Unconfined compressive strength of intact rock} \]

\[ K_{sp} = \text{Coefficient based on spacing of discontinuities (Table 1)} \]

<table>
<thead>
<tr>
<th>Condition of Discontinuities</th>
<th>Spacing of Discontinuities (m)</th>
<th>( K_{sp} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moderately close</td>
<td>0.3 - 1</td>
<td>0.1</td>
</tr>
<tr>
<td>Wide</td>
<td>1 - 3</td>
<td>0.25</td>
</tr>
<tr>
<td>Very wide</td>
<td>&gt; 3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

For moderately close spacing of discontinuities on the rock, the allowable rock base resistance \( f_{b\ (al)} \) is approximately 10% of the unconfined compressive strength of intact rock. The \( f_{b\ (al)} \) of the pile design shall not in any way exceed the specified allowable concrete structural capacity of 0.25\( f_{cu} \) as required in BS8004 (1986).

The rock socket design for this project was optimised with consideration of load distribution behaviour in the rock socket. The load transfer in the rock socket was apportioned between shaft due to bond friction and base resistance in accordance to Pells & Turner (1979) and the limiting ratio of the belled-out base diameter \( (D_b) \) to the straight shaft pile diameter \( (D_s) \) can be assessed using Eq. (4) to maximise the contribution of belled-out base resistance. The allowable base resistance of rock \( (f_{b\ (all)}) \) shall not exceed the allowable concrete structural capacity of 0.25\( f_{cu} \) for the narrowest pile shaft.

\[ D_b \leq \frac{0.25 f_{cu}}{f_{b\ (all)}} \cdot 0.5 \]  

(4)

Initially, the belled-out base (to be socketed at minimum 0.5m below the surface of rock) was thought to have significant savings in rock hacking for the equivalent straight shaft in the condition of flat bedrock surface (Fig. 5). It was finally found that the savings in shortening the rock socket length with belled-out base was insignificant as majority of the bedrock surface for this project site found to be slanted. Hence the belled-out base for inclined bedrock surface exceeding gradient of 1V:2H was found to be unfeasible.

Generally, the contribution of shaft friction from the cone-down rock contact surface shall be downgraded or even ignored as the downward movement of the pile may separate its contact with the rock. It is also prudent to downgrade the bearing capacity of the belled-out base which is sitting directly on the founding rock with inclined rock surface. Caisson with belled-out base was eventually not adopted for this project due to neither significant cost/time savings (majority of slanted bedrock surface for this project) nor technically more superior compared to vertical shaft rock socket which was much easier to construct.

<table>
<thead>
<tr>
<th>Figure 4 Load Distribution in rock socket (after Pells &amp; Turner, 1979)</th>
</tr>
</thead>
</table>

| Figure 5 Rock socket in flat & slanted bedrock surface |

3. **PILE LOAD TEST**

The performance of the caisson piles was verified by two numbers of static maintained load test with test load up to 1,700 tonnes (pile diameter, \( \phi=1.2m \), rock socket, \( L=2m \)) and 3,000 tonnes (\( \phi=1.5m \), \( L=2.5m \)) respectively. Caisson test pile with diameter of 1.5m was instrumented with four levels of arch weldable strain gauges and tell-tale extensometers respectively. A plate bearing test was also carried out to check the performance of the rock base resistance.

The design shaft and base resistances for the granitic rock were conservatively adopted as 1000kPa and 2000kPa respectively due to the low unconfined compressive strength (averagely 10MPa to 35MPa) on the rock samples collected during subsurface investigation (SI).
The results of the pile load tests are summarised in Table 2. Generally, both of the static maintained load tests show satisfactory results with maximum pile top settlement and residual settlement of not more than 9mm and 2mm respectively. The plate bearing test recorded a load of 6000kPa on the rock base only with a residual settlement of 0.5mm only. The pile load distribution recorded in the instrumented static load test (Fig. 6 & Fig. 7) shows that the mobilised rock shaft resistance was 1050kPa and 1750kPa for top 1m and subsequent 1.5m socket length respectively. The mobilised rock base resistance was about 3700kPa (contribution of about 20% of the total test load) with base settlement of about 2mm only. As in most cases of rock socket pile, the settlement performance of the test pile under the maximum imposed test load is mostly governed by the elastic shortening of the pile body, in this case about 5mm. The shaft resistance of the concrete linings in soil, which was intentionally ignored, was able to mobilise up to 270kPa.

Generally, the performance of the two caisson test piles was better than prediction. As the rock socket for the caisson pile was formed by manual hacking involving removal of localised rock wedges, the resulting rock surface was very much irregular compared to those formed by conventional rock coring bits. The high roughness on the rock surface for caisson pile will undoubtedly increase the normal contact stress at the socket interface due to dilation of socket diameter during shearing. Nonetheless, it is also important to consider the failure of the rock asperities when the local contact stresses exceed the rock strength during the process of concrete sliding over the irregular rock surface.

Unlike conventional bored pile construction which usually has problems of soft toe and smear of bentonite cake on borehole surface, proper cleaning and inspection of the rock socket of caisson pile can be carried out prior to concreting to ensure no debris on the surface of shaft and base. Hence, the above beneficial factors, which also influence greatly on the performance of the caisson pile, shall always be considered besides the strength and stiffness properties of rock mass.

Table 2: Summarised Results of Pile Load Test

<table>
<thead>
<tr>
<th>Pile Tests</th>
<th>Pile Diameter</th>
<th>Testing Load / Pressure</th>
<th>Max. Pile Top / Plate Settlement</th>
<th>Residual Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintained Static Load Test</td>
<td>φ1500mm (instrumented)</td>
<td>30,000 kN (2x Design Load)</td>
<td>7mm</td>
<td>1.5mm</td>
</tr>
<tr>
<td></td>
<td>φ1200mm</td>
<td>17,500 kN (2x Design Load)</td>
<td>9mm</td>
<td>2mm</td>
</tr>
<tr>
<td>Plate Bearing Test</td>
<td>φ1500mm (plate size= 6750mm)</td>
<td>6000 kPa (3x Allowable Base Resistance)</td>
<td>2mm</td>
<td>0.5mm</td>
</tr>
</tbody>
</table>

The typical sectional view of the caisson pile is shown in Fig. 13. Generally, caisson pile construction can be simultaneously carried out by several independent teams of workers. At least a worker shall be stationed on the ground surface to oversee the excavation of two adjacent caisson holes and help in transferring the excavated material to the ground surface by using the electrically operated hoisting bucket and ensure the safety of the workers inside the holes.

For soil excavation without rock boulder or construction debris, the production rate was typically 1.5m per day after waiting for minimum of 24 hours curing of the concrete linings. Hence for caisson pile with 20m overburden soil, it would require minimum of two weeks to complete the soil shaft excavation. Rock hacking usually starts at one edge of the rock encountered and the average rate of hacking for a pile diameter of 1.5m is about 0.15m per day for the granitic rock with weathering grade of Grade III or better (hacking took about 20 days for 2.5m rock socket length).

Existence of rock boulders in the overburden soil, which require to be breakdown into smaller pieces for removal out of the hole, was always time consuming in caisson pile construction. Excavation time would be further increased if the rate of seepage water through the base or less watertight lining into the digging hole is high.

4. CONSTRUCTION AND INSPECTION

4.1 Construction Sequence and Rate

Construction of hand-dug caisson involves manual soil and rock excavations using portable digging and chiselling tools. Hence, the smallest workable pile diameter is about 1m for a man to work in a confined hole. The typical construction sequences for caisson pile are:

a) Platform preparation
b) Excavate soil vertically to form a circular shaft to a depth not greater than 1.5m.
c) Placing of steel mould (Fig. 8) and steel mesh before concreting of circular lining. Apron is to be cast with normally 250mm above surrounding ground to prevent ingress of surface run-off.
d) Set-up of hoisting frame (Fig. 9) and ventilation system.
e) Dismantle the steel mould 24 hours after lining casting and excavate for the next lift of soil (Fig. 10). Repeat the earth excavation and lining casting until the designated pile base level (Fig. 11). Generally, no concrete linings are required in rock socket.
f) Cleaning of rock shaft and base using pressurised water jet and inspect the rock socket. If the rock mass is highly fractured, the rock socket hole shall be lined for stability.
g) Lowering down the steel cage. Final inspect the rock socket and steel cage (Fig. 12) before concrete casting using tremie method.

Figure 6 Test pile load distribution
the weathering grade of rock and existence of any prominent joints or fractures with adverse settings potentially having unfavourable excessive movement/settlement of the foundation. The 360° rock mapping logging (Fig. 13) was used during the construction to closely document the rock socket condition. The logging is important, especially for slanted rock surface or encountering different level of competent rock surface. For sloping rock surface, the lowest point of rock face with weathering grade better than Grade III intersected by pile was prudently taken as the commencement level of the rock socket. The uncertainty of rock strength interpreted from visual inspection was also verified with point load test at site.

4.3 Safety and Health Issues

Besides the raised apron above the surrounding ground to prevent ingress of surface run-off, the caisson hole shall also be barricaded to prevent any object or even human from accidentally falling into the hole. As mentioned earlier, at least one worker should be stationed on top of the caisson while excavation is carried out in the hole to monitor the condition in the caisson.

For rock hacking in the deep hole, the worker shall not work continuously exceeding one hour in the hole as the dust or noxious gases produced during rock hacking can likely be trapped at the bottom of the hole due to poor or insufficient ventilation. It is important to ensure the sufficient capacity of chain and bucket to transfer the excavated material and the weight of workers to be hoisted down to the hole by the bucket.

5. CONCLUSION

In summary, caisson pile is feasible and practical to be adopted for site with condition of sloping ground and limited space restricting the construction of using conventional bored piling or installation of prefabricated piles.

Unlike bored piles, base resistance of the caisson pile can be considered in the design of pile capacity as it has the advantage of cleaning and inspecting all the load bearing surface and the founding stratum (usually rock) to ensure no soft toe problem before concreting. The rock socket design can be optimised with consideration of load distribution between shaft and base resistance based on functions of pile geometry and the relative stiffness between concrete and rock mass with assumption that displacement at the rock socket is small and bond rupture has not occurred. The normal safety factor adopted shall be sufficient to attain the condition of the assumptions.

The performance of the test piles in this case study was better than prediction due to the much higher roughness on the rock surface for caisson pile which will enhance greatly the shaft resistance compared to the rock socket formed by the conventional rock coring bits. In addition, the problem of soft toe and smear of bentonite cake in the rock socket for caisson pile can be eliminated after proper cleaning and inspection prior to concreting.

Construction of caisson pile has the advantage of working simultaneously by several independent teams. The recorded average rate of hacking for pile diameter of 1.5m with granitic rock of weathering grade III was about 0.15m per day. The 360° rock mapping log was used to closely monitor the rock conditions of the socket during the construction.

It is risky working in the deep shaft caisson. Safety and health of the workers working in the caisson hole shall not be compromised during construction.

1.7 REFERENCES