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Piling in ancient coral formations, Simberi Island, Papua New Guinea

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ABSTRACT

Allied Gold Pty Ltd commissioned Golder Associates Pty Ltd to investigate, design and supervise the foundation construction for the new Semi-Autogenous Grinding (SAG) mill at the gold mine site on Simberi Island, Papua New Guinea. This paper describes the geological setting, investigation method, assessment and design process, construction aspects and results of measured set during driving. One of the most interesting factors about the project was that the ground conditions comprised ancient coral beds. There is limited published past experience with deep foundations in such materials worldwide and especially for work in PNG. The gold mine site is located on an idyllic island setting on the northern most Tabar Group of islands in the New Ireland Province of PNG. Simberi Island is an eroded Pliocene (2.6 million years old) alkaline volcano. It is part of chain which comprises a series of Pliocene to Holocene (Recent) volcanoes that occupy the New Ireland Basin. Geologically, the site is quite complex. Raised Quaternary coral reef platforms surround steep sided volcanic plateaus, which rise up to 300 m above sea level. The plateaus are fringed by Miocene (23 million years old) to Pliocene raised coral and limestone reefs (cemented material) and deposits of lower quality weathered reef deposit or limestone (coronous) and alluvial and colluvial mixtures of sand, silt and clay. These sequences make for variable ground conditions, adding to the challenge of designing piles in such materials.

Keywords: coronous, ancient coral formations, limestone, driven pile

1 INTRODUCTION

The SAG mill grinds rock boulders for ore processing at the mine site. There are static and dynamic loads associated with this process so careful consideration of the foundation conditions was required. The formation soils are a mixture of weathered coral beds, interspersed with colluvial and alluvial clays and some limestone boulders. Shallow foundations were considered but because of the material variability, loads, limitations on differential settlement criteria, and concern over the crushing potential of the corals especially under dynamic forces, deep foundations were selected.

2 GROUND INVESTIGATION

The investigation comprised drilling of boreholes with rock coring where possible, and Standard Penetration Testing (SPT). The formations encountered on site were variable in terms of strength, void ratio and permeability due to the degree of coral lithification, zonation and weathering. The boreholes showed that the uppermost layers of the site comprised alluvium overlying layers of weathered cemented coral / limestone. Coral limestone / coronous formations were typically encountered below 12 m depth. Some high strength coral cobbles were encountered. Groundwater level was observed in the boreholes generally around 4 m below ground level (BGL) which corresponded roughly with sea level at the site. Core samples were able to be retrieved from some of the boreholes, but very few samples were suitable for testing. Those which were tested revealed unconfined compressive strength (UCS) between 1 MPa and 8 MPa, suggesting low strength rock.

3 ANALYSIS

3.1 Foundations

The strength profiles within the boreholes indicated a possible pile founding level between 12 m and 23 m BGL, within cemented limestone / coral. We assessed the options of deep foundations using driven and bored concrete piles. The final decision was influenced by site logistics. Our assessment was that bored piles would be preferable in such ground conditions where there was a general lack of reliable founding layer suitable for driven piles. However, within the construction timeframes available and the difficulty of providing access for a large bored rig to Simberi island, the option to use driven circular hollow section (CHS) piles was adopted.

Calcareous / carbonate sands and coral formation present difficulties in foundation design because they do not typically conform to traditional silica sand and quartz formation based analysis techniques. Driving a pile into calcareous material has a tendency to crush the particles at the base and around the pile shaft creating irregular contact. We analysed the driven pile by assuming the founding layer would effectively be a gravel rather than intact rock. To limit overall ground disturbance, we kept the pile diameter relatively small at 324 mm.

Due to the presence of high strength cobbles within the ground profile, we recommended that a driving shoe 25 mm thick be welded to the pile toe. The drive shoe was made 0.5 m long, and flush with the outer diameter of the pile tubes, thus helping to protect the steel tube piles from damage if the piles encountered high strength cobbles during driving.

The capacity of driven piles depends on, amongst other things, the ground profile, pile diameter and depth of penetration. In this case, steel 324 mm diameter CHS piles, with 12.7 mm wall thickness were used to assess axial capacity as a function of depth of penetration.

We recommended that the ultimate skin friction (f_s) and ultimate end bearing (f_b) be limited to 20 kPa and 4000 kPa, respectively.

We anticipated variability in material strength and penetration depths would be encountered to achieve the required pile set (pile penetration per hammer blow) and level of resistance to limit settlement to acceptable values. We recommended a geotechnical strength reduction factor (ϕ_g) of 0.52 be used in accordance with the Australian Standard for piling (AS 2159-2009).

We insisted that all pile operations be supervised by a geotechnical engineer, and early pile load tests be carried out to verify pile capacity because of the uncertainties associated with the performance of piled foundations at this site.

3.2 Pile Capacity

As the open ended steel CHS piles were to be driven into cemented coral / limestone, we expected a plug would form inside the pile, making the pile act like a closed end pile increasing toe resistance as the pile was driven into the coral.

The extent of plugging of the pile was determined on the basis of the measured difference between the ground levels inside and outside the pile following driving. The ultimate axial capacity of the pile in end bearing was assessed assuming a pile plug develops, where the ultimate end bearing capacity is equal to the lesser of:

- the bearing pressure of the full plugged area; or
- the sum of the internal skin friction of the pile plus the net bearing pressure of the steel pile (Tomlinson and Woodward 2008).

The design ultimate axial pile, ($R_{d,ug}$), capacity of a single vertical pile and settlement was calculated using the program AllPile (CivilTech 2009) in accordance with Australian Standard AS 2159-2009.

Figure 1 illustrates the design ultimate skin friction, tip resistance and geotechnical strength as a function of depth below existing ground surface.

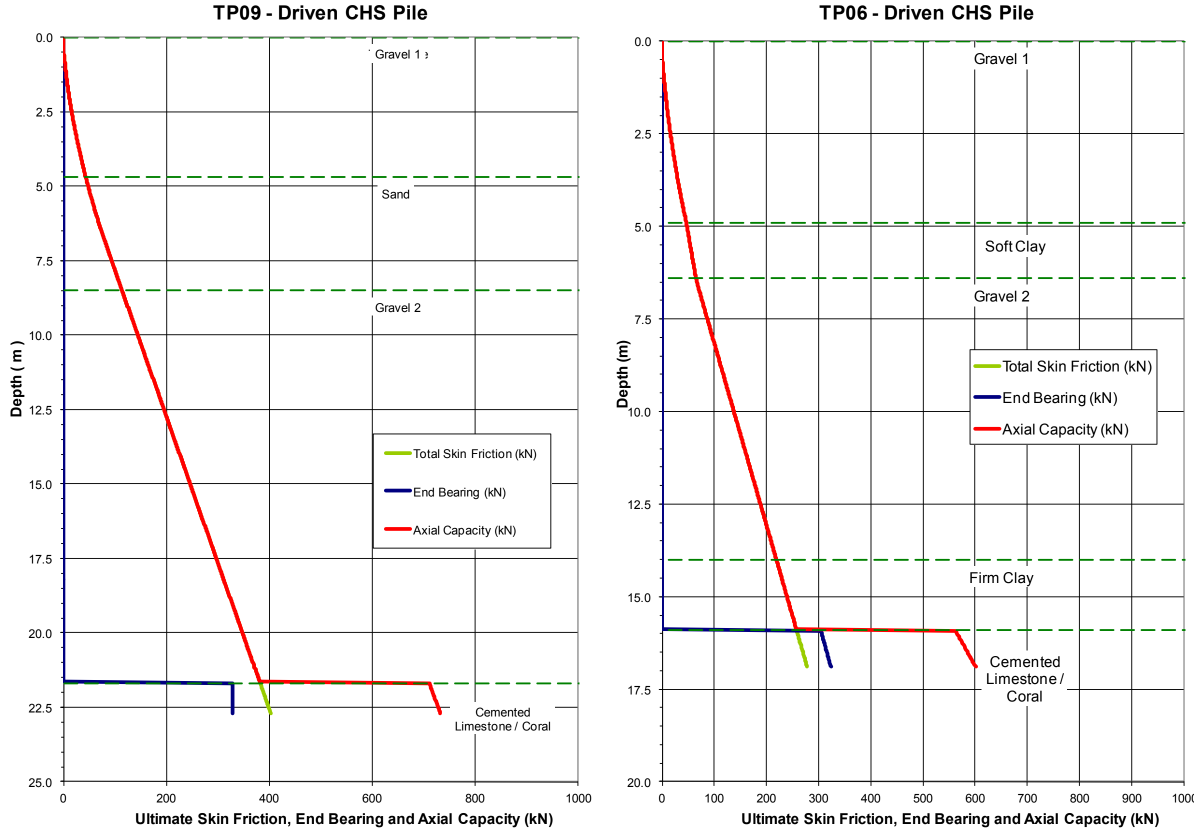

Figure 1: Ultimate pile capacity estimate for a driven pile

Skin friction for the pile within 1.5 times the diameter of the ground surface was neglected in the model to represent possible strength reduction during installation or deterioration of the in-situ soil over time (AS 2159-2009).

The results showed that a 324 mm pile driven 1 m into the cemented coral / limestone layer between 17 m and 23 m (below ground level) would achieve a design geotechnical strength, ($R_{d,g}$), of 310 kN.

3.3 Pile Driveability Analysis

In terms of hammer selection, we assessed that there was a fine line between selecting a hammer which could achieve the required penetration depth and design geotechnical capacity, and one which would have too much energy, potentially crushing the coral / limestone, possible damaging the pile toe and overdriving the piles making it difficult to achieve the desired set required to verify the anticipated design loads.

Driveability analysis was carried out using the program GRL WEAP (2005). The program models the pile and the pile driving hammer using the wave equation. Output from the program provides a plot of calculated pile penetration per blow versus a range of assumed pile ultimate driving capacities, as well as hammer transferred energy and maximum pile compression and tension stresses for a selected hammer and pile combination.

Two end plug conditions were considered in the assessment; a fully developed end plug during pile driving (total end bearing capacity) and no end plug during driving (end bearing of the pile tip cross sectional area only). We considered both cases so that if a plug did not develop, the hammer would still have sufficient energy to install the piles to the required founding level for the allowable working loads.

Our results indicated that an impact hammer with transfer energy of approximately 12 kNm would be sufficient to drive a steel CHS into the cemented limestone / coral. We recommended that during construction, the piles be driven to a penetration (set) of 10 mm/blow, calculated to provide just over 600 kN in ultimate geotechnical strength.

The driveability assessment was indicative and based on assumed soil conditions, soil damping factors, pile lengths and estimated distributions of pile skin friction and end bearing (from the AllPile analysis). The hammer efficiency value in the analysis was based on the manufacturer's recommendations included in the GRL WEAP program database. The onsite hammer efficiency was to be assessed following on site pile testing.

4 CONSTRUCTION OBSERVATIONS

The Contractor brought to site a 2T Delmag D22 diesel hammer, which had a 1 m drop height and a transfer energy of about 12 kNm. In addition an ICE 416 vibrating hammer was mobilised. Steel H sections and cross beam pile guides were first erected at the site using the vibrating hammer, so that the CHSs could be driven straight within the pile guides.

The CHS pile sections were provided in 12 m lengths. Welding of the steel piles was carried out onsite by local contractors. All welding was carried out in accordance with AS/NZS 1544.1-2004.

The driving of the steel piles was supervised by a geotechnical engineer, who measured the set of each pile during driving. Pile Driving Analyser (PDA) testing equipment was mobilized to site, but unfortunately the telemetry connection between the top of the pile and the computer was not working and no PDA tests were able to be carried out. The capacity of the driven piles was verified on site by the measured set of each driven pile, adopting the Hiley Formula (Bowles 1996) below with a hammer efficiency of 80%.

where, is the ultimate pile capacity
is the hammer efficiency
is the mass of the ram
is the mass of the pile (and helmet)
is the height of fall
is the coefficient of restitution
is the temporary compression of the pile head and cap
is the temporary compression of the overall pile
is the temporary compression of the ground
is the set (mm / blow)

A total of 20 piles were driven to depths between approximately 15 m and 23 m. The Hiley Formula indicated that for each pile, an ultimate pile geotechnical capacity of approximately 600 kN was achieved. The pile plug lengths were also measured following driving, and indicated that a soil plug had developed in all driven piles.

5 DISCUSSION

The design pile capacity was verified using AllPile, adopting the ground conditions from a nearby borehole, the measured pile length and a soil plug. Figure 2 shows the ultimate skin friction, ultimate tip resistance and ultimate geotechnical strength as a function of depth for one of the driven piles, driven to 18.27 m below existing ground surface.

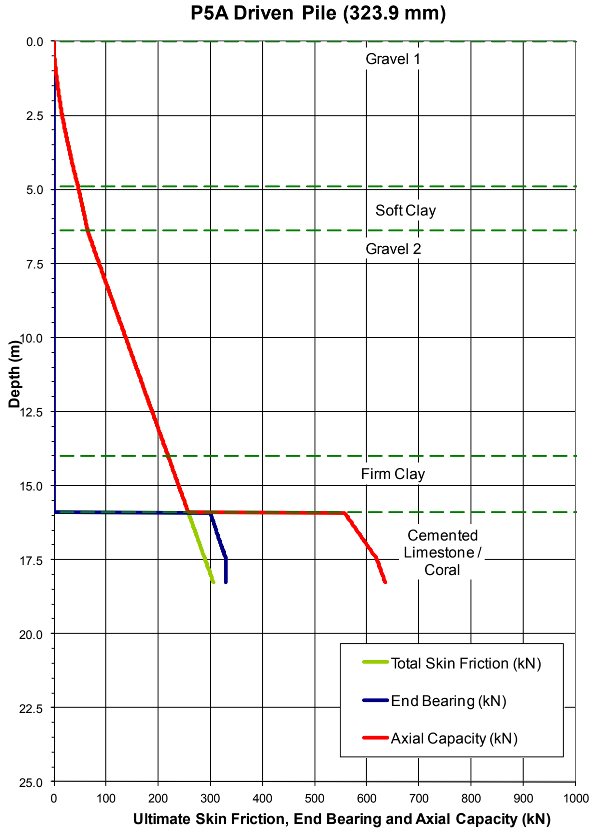

Figure 2: Ultimate pile capacity estimate for driven pile as constructed.

The result shows that the pile driven into the cemented coral / limestone layer (about 18 m below ground level) achieved an ultimate geotechnical strength of about 630 kN and a design geotechnical strength, ($R_{d,g}$), of about 330 kN based on a geotechnical strength reduction factor of 0.52. The safe working load of the same pile estimated using the Hiley Formula and a factor of safety of 2.5, was approximately 330 kN.

We carried out a driveability assessment for the pile using the GRL Weap program. The results indicated that the pile could be driven to a penetration (set) of approximately 15 mm/blow which would provide 635 kN in ultimate geotechnical strength. The actual set measured on site was approximately 9 mm / blow.

In general, the results following construction agreed with the design estimates of pile capacity and driveability. An ultimate skin friction (f_s) and ultimate end bearing (f_b) value of 20 kPa and 4000 kPa respectively for coral limestone / coronous formations appear to be appropriate for pile driving at this site. The pile installation parameters are summarised in Table 1.

Table 1: Summary of Pile Assessment Details

<table>
<tr><th>Stratigraphy</th><th>Parameters</th><th>Estimated Founding Depth (m)</th><th>Actual Founding Depth (m)</th><th>Pile Parameters</th><th>Design Geotechnical Strength, Rd,g (kN)</th><th>Hammer Details</th></tr>
<tr><td>Clayey Sandy Gravel, medium dense</td><td>fs = 5 to 15 kPa</td><td>-</td><td>-</td><td rowspan="5">323.9 mm CHS, 12 mm wall thickness, 25 mm thick driving shoe</td><td rowspan="5">310 to 380</td><td rowspan="5">2t diesel, Delmag D22, 12 kNm transfer energy</td></tr>
<tr><td>Clay, very to soft</td><td>fs = 12 kPa</td><td>-</td><td>-</td></tr>
<tr><td>Clayey Sandy Gravel, medium dense</td><td>fs = 18 to 20 kPa</td><td>-</td><td>-</td></tr>
<tr><td>Clay, firm</td><td>fs = 20 kPa</td><td>-</td><td>-</td></tr>
<tr><td>Cemented Coral / Limestone</td><td>fb = 4000 kPa
fs = 20 kPa</td><td>17 to 23</td><td>15 to 23</td></tr>
</table>

6 CONCLUSIONS

In summary the piles were installed successfully into the cemented coral / limestone formation, using the Delmag hammer with a transfer energy of about 12 kNm. The driveability analysis results and the estimated cemented coral / limestone parameters were confirmed on site by measuring the pile set during installation, and pile plug length following construction.

7 ACKNOWLEDGEMENTS

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