Driving Tubular Steel Piles into Weak Rock – Western Australian Experience

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ABSTRACT

This paper presents a discussion of recent over-water pile driving experience for marine structures in the Pilbara Region of Western Australia. The recent resources boom has created a need for developing and upgrading export facilities along the coast of Western Australia. In the Pilbara Region on the North West coast of Western Australia, many of the port facilities are located in areas of shallow, Quaternary, weak sedimentary rocks with varying calcium carbonate content. The wharf structures are typically founded on tubular steel piles driven into the rock. With the long lead time required for ordering tubular steel piles and high cost of field splicing, it is important to be able to accurately predict pile driveability and penetration depths at design time. Pile penetrations, soil resistance to driving and driveability parameters are discussed, using, as an example, results of driving 1200mm diameter piles during part of an upgrade of Hamersley Iron's Dampier Port.

1 INTRODUCTION

The Pilbara coastline of Western Australia is home to several major existing and proposed export facilities. Major existing and proposed facilities are located along the Pilbara coastline of Western Australia at Onslow, Cape Preston, Dampier, Cape Lambert and Port Hedland. They are used to export iron ore, oil and gas products including LNG, petrochemicals including ammonium, salt, feldspar, manganese, scrap metal and livestock. Given the region is the most active cyclonic region in Australia, these port facilities need to be designed for large Category 5 cyclones. The over water facilities typically include:

- An access jetty with a conveyor or pipe work to transport the export product to the wharf head.
- A wharf head with adjacent berthing pockets. This usually supports a ship loader.
- Independent mooring and berthing dolphins.

The wharves and dolphins are typically founded on driven tubular steel piles, subject to compression and uplift loading. Some piles include anchors or drilled extensions to provide additional uplift capacity. With advancement in pile driving hammer size and efficiency, increasingly larger and higher capacity piles are being utilised for the foundations.

Most of the sites along the Pilbara coast are underlain by variably cemented, Quaternary, sedimentary rocks and so high capacity driven piles are possible, however heavy driving is typically required to achieve adequate pile embedment and uplift capacity. This paper discusses the typical challenges faced by the pile designer and includes an example driveability assessment for some typical piles recently installed at the Hamersley Iron, Parker Point, upgrade in Dampier.

2 NEAR SHORE GEOLOGY

The near shore geology along the Pilbara coast line is typically comprised of recent carbonate and/or mangrove sediments, overlying variably cemented sedimentary rocks, overlying igneous bedrock. The recent carbonate or mangrove sediments are typically shallow (<5m thick) and have little influence on pile design. The variably cemented sedimentary rocks were typically clay and sand mixtures derived from erosion of the rocks further inland and deposited over successive ice ages, when sea levels were lower. More recently they have been cemented by carbonate and limonite cementing agents in the marine environment. These rocks typically have calcium carbonate ($CaCO_3$) contents ranging from 20% to 80%. More familiar calcarenites, comprised of cemented shells, ooids and marine organisms, with over 90% $CaCO_3$ are also common along the Pilbara coast and North West Shelf, however they are not found at most of the export facilities along the Pilbara coastline.

The weak rocks encountered along the Pilbara coastline are characterised by their high degree of variability in strength, both laterally and vertically. The rock strength typically ranges from extremely low strength (virtually uncemented) to very high strength and varies over short distances.

Underlying the weakly cemented rocks there is typically igneous bedrock. At some sites this is deep enough to have no influence, and at others, especially around Dampier it is shallow and has a major impact on pile design. Driven piles typically cannot penetrate igneous bedrock and so, where encountered at a shallow depth, piles usually require anchoring or drilled extensions.

3 PILE DESIGN CONCEPTS AND CHALLENGES

In recent years, piles for major wharf projects in the Pilbara are typically procured in South East Asia or China. Given the high labour costs in the Pilbara, it is cheaper to order the piles, fully fabricated and painted and have them delivered to site by barge, ready to drive, rather than order standard length pile sections and fabricate the piles on site. Splicing costs during driving are extremely high, compared to the cost of ordering longer piles to start with.

Piles for wharf projects in the Pilbara are typically 600mm to 1200mm in diameter, spirally welded, with wall thicknesses ranging from 12mm to 25mm and a total length in the range of 20m to 50m. Note that the distance from seabed to the pile cap is typically 10m to 30m and the pile embedment is typically 5m to 20m. Pile ultimate limit state design action effect (S*) compression loads are typically in the range of 2,000kN to 12,000kN, and pile uplift loads can range typically up to 4,500kN, but on some projects as high as 7,000kN. Typically, hydraulic pile driving hammers are used for driving the piles. These generally have ram weights that range from 9t to 20t, with drop heights of 1.2m up to 1.9m.

The piles usually have 0.7 to 1.5m long, heavy driving shoes, typically 32mm to 40mm thick (outside diameter matching the pile diameter), with a chamfer on the inside face. The driving shoes are used to prevent buckling and to reduce the internal friction in order to assist in obtaining maximum penetration, to maximise uplift capacity. Where anchors are likely to be required, the piles can be ordered with spiral weld beads on the inside of the pile, to improve grout-to-steel load transfer. Typically, the piles are painted from the top, down to a nominated level below the low tide level to provide corrosion protection.

A typical major project would have in excess of 150 piles. Given the long lead time on ordering piles, by the time the first pile is driven on site it is usually too late to order additional pile lengths, if required. Alternatively, if all the piles stop much shallower than expected there could be large amounts of off cuts and the bottom of the pile painted section may not end up in the correct position. The biggest challenge facing the pile designer is therefore in estimating the pile toe levels and providing the right balance of conservatism so that the number of field splices required during driving are kept to a minimum, whilst making sure the amount of offcuts are also kept to a reasonable amount. It is interesting to note that the cost of making a field splice on a large diameter pile, which typically ties up the piling barge for at least a whole shift, could be more than the cost of ordering at least an extra 50m or more of steel.

As previously stated, the weak rocks in the Pilbara have a high degree of strength variability. This variability becomes evident during pile driving where commonly adjacent piles in the same pile group, with the same diameter and driving hammer can refuse typically at depths varying by 4m and sometimes, for example in Port Hedland, refusal depths can vary by as much as 6m or 8m. This variability adds to the difficulty in predicting toe levels and making pile orders. The need for avoiding splices dictates that there will have to be significant lengths of off cuts on most piles.

The authors' recent experience in the Pilbara is that piles are more likely to refuse shallower than expected, rather than deeper than expected, and so a major challenge is in predicting how many piles might refuse so shallow that they have insufficient penetration for uplift capacity and therefore require anchors. Both prestressed anchors and passive drilled extensions have been utilised in the Pilbara. Each has their own advantages and disadvantages.

Prestressed anchors have the advantage that it is easier to procure the reinforcing elements and can usually be installed by a different crew, at some time after the piling barge has moved away from the piles and so are not on the critical path for the piling barge. Their main disadvantage is that, for piles subject to similar tension and compression loads, the compression capacity (structural and geotechnical) needs to be equal to the sum of the compression and anchor prestress loads. The designer therefore needs to know in advance that an anchor will be required, so that the correct pile section can be ordered and the pile can be driven to the required compression capacity.

Passive drilled extensions have the advantage that they don't require the pile to have additional compression capacity and so can be installed in piles that weren't originally designed to need them. However, passive drilled extensions usually need to be installed at an earlier stage in construction, before the pile cap construction is commenced and typically use the same piling barge. They are therefore on the critical path for the piling barge and so the drilling equipment and anchor reinforcements need to be on site, ready to install if required.

4 PILE DESIGN METHODS

There is little information available in the literature for design of steel piles driven into weak rock. Most of the design methods available for rock relate to bored piles, where socket roughness is a major contributor to pile capacity.

The authors have found a reasonable design can be conducted, using the methods for driven piles in clay, given in Clause 6.4 of the API RP2A-WSD-2000 code. In this method, the shaft adhesion is based on an "αs_u" approach, where s_u is the undrained shear strength, taken as half of the intact rock Uniaxial Compressive Strength (UCS) and α is a function of the ratio of s_u and effective overburden stress, σ'_{vo}. End bearing for clay is typically estimated as 9 x s_u, and so for weak rock the authors have estimated it as 4.5 x UCS.

It has been observed that with driving shoes, the piles do not tend to plug during driving, however it is possible that pile driving refusal occurs at the moment when the pile becomes plugged and so even though the pile is plugged, there may not be a measurable difference in soil height inside the pile compared to outside the pile. The authors have based the pile compression capacity design on an end bearing capacity of 4.5 x UCS applied over the gross area of the pile, multiplied by a factor of 0.77 to account for the compressibility of the plug, as suggested by Bruno and Randolph (1999). Stevens *et al.* (1982) used a unit end bearing value of 3 x UCS for evaluating driveability in rock. In applying the API code recommendations for clay to the design of piles in rock, the authors have limited unit shaft friction to 500kPa and unit end bearing (over the gross area) to 15MPa.

5 DAMPIER PORT DRIVEABILITY EXAMPLE

Hamersley Iron's Phase A Dampier Port Upgrade is located at Parker Point, Dampier and involved extension of the existing berth to increase the existing load-out capacity. Junttan HHU 16A and Junttan 20S hydraulic hammers were used to drive 1200mm diameter piles and a Junttan HHK 9A hydraulic hammer was used to drive 700mm diameter piles. For some of the larger capacity dolphin piles, a Menck MHUT 270 hammer (16t ram with 1.89m drop height) was used. CAPWAP® (2000) results on piles driven on this project indicated that piles typically achieved ultimate compression capacities of 12,000kN to 16,000kN for the HHU 16A, 14,000kN to 18,000kN for the 20S and MHUT 270, and 5,000kN to 7,000kN for the HHK 9A hammers. This example focuses on estimating the driveability characteristics of a selection of piles driven with the HHU 16A hammer.

5.1 Design Geotechnical Parameters

The general geology at the Parker Point site comprises of very soft siliceous carbonate silt, overlying very low to low strength, variably cemented siliceous calcarenite, overlying high to very high strength igneous rocks such as granite, granophyre, gabbro or dolerite. The generalized soil profile obtained from borehole BH6 at the site is provided in Table 1.

Table 1: Generalized Soil Profile – Borehole 6

Elevation mCD	Layer	Description
-7.81	Siliceous Carbonate SILT	Very soft, low plasticity, trace of shells.
-10.7	CLAY	Very stiff, medium plasticity, trace of sand.
-11.2	Clayey Siliceous Carbonate SAND	Medium dense to dense, weakly cemented.
-14.8	Clayey Siliceous CALCARENITE	Very low to low strength, medium to coarse grained.
-22.3	CALCARENITE	Medium to high strength, fine grained, with high strength limestone bands.
-28.7	Conglomeratic CALCIRUDITE	Medium to high strength, granophyre cobbles cemented by limestone.
-29.4	BRECCIA	Medium to high strength, angular.
-30.7	Brecciated DOLERITE	High to very high strength, distinctly weathered.

A profile of UCS is required for pile design, and this was obtained from a combination of laboratory testing and field estimation. Reliable UCS testing of the weak rocks found on the Pilbara coastline is difficult as often pre-existing defects, poor end preparation or anisotropic cementing results in premature failure of the UCS specimens and gives results that are lower than the true intact rock strength. The authors have found that field strength estimation based on the field description in Australian Standard (AS1726), supplemented by Point Load Strength Index (Is_{50}) testing, can be a more reliable indicator of rock strength for pile design, since the Is_{50} tests require shorter samples that are not as affected by end preparation.

The inferred UCS values are obtained from Point Load Strength Index testing by linear correlation of tests on adjacent samples, filtering out the test results that appear unreliable. At the Dampier site, a correlation of UCS = 7 x Is_{50} was adopted for design. The rock strength estimated from the field description (AS1726) for BH06 is provided in Figure 1. The design UCS line for BH06 based on the field description, measured UCS values and inferred UCS from Is_{50} is shown in Figure 2. For this example of pile driveability assessment, the upper value of the rock strength from the field description and test results was used as shown on Figure 2.

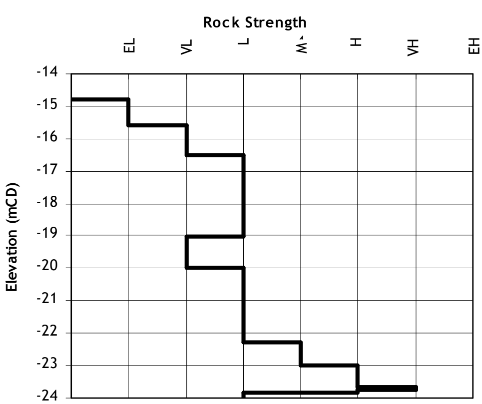

Figure 1 Strength Based on Field Description

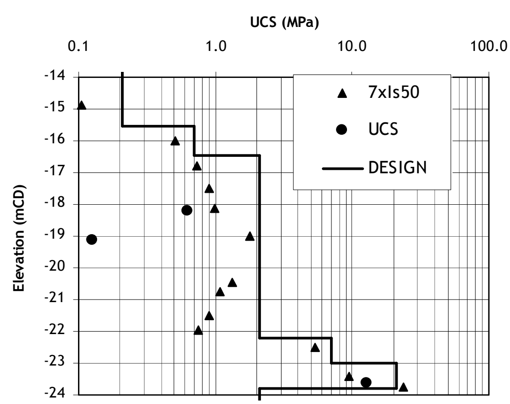

Figure 2 Design UCS for BH06

5.2 Driveability Analysis Parameters

Soil quake and damping parameters were determined from a back analysis and review of the CAPWAP results for 1200mm diameter tubular steel piles driven during previous piling operations at Dampier port. The following parameters were determined for the weak rock and used in this study:

- Skin Quake: 2.0mm
- Skin Damping: 0.5sec/m
- Toe Quake: 1.0mm
- Toe Damping: 0.35sec/m

5.3 Soil Resistance to Driving

For the driveability assessment, soil resistance to driving was estimated based on the ultimate geotechnical capacity (R_{ug}) estimated using the static design method described in Section 4 and the profile of UCS described in Section 5.1, with no allowance for capacity reduction during driving. The estimated unit shaft friction and unit end bearing is given Figure 3. The estimated soil resistance to driving for a 1200mm diameter tubular steel pile for BH06 is presented in Figure 4. Lower and upper bound alternatives of fully coring, neglecting all internal friction and fully plugged using end bearing over the gross area of the base were trialled. The lower bound coring capacity was estimated by adding the outside shaft friction with end bearing resistance on the pile shoe area only.

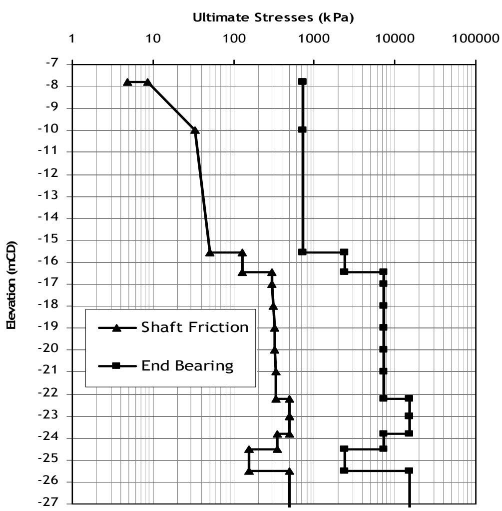

Figure 3 Ultimate Stresses

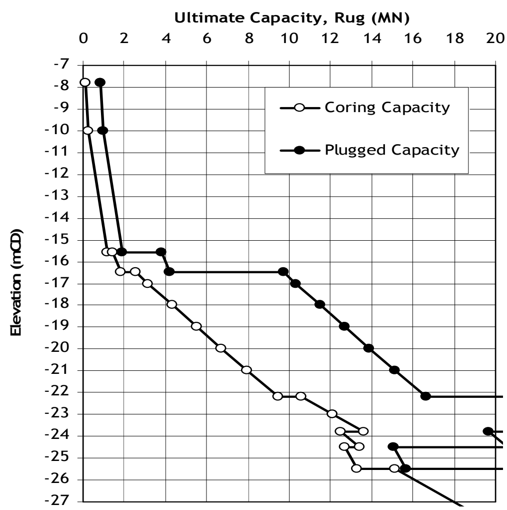

Figure 4 Ultimate Pile Compression Capacity

5.4 Driveability Analysis Results

Using the computer package GRLWEAP (Version 2003), the soil resistance to driving was assumed equivalent to the pile static capacity estimated using the API RP2A method. For this example, pile set-up factors were ignored. The default GRLWEAP "manufacturer's recommended" parameters for the driving system (hammer cushion, helmet, etc), a pile cap weight of 8kN and a hammer efficiency of 90% were used in the analysis. The analysis confirmed that a soil resistance of up to 16,000kN can be overcome with a Junttan HHU 16A hydraulic hammer, which is compatible with CAPWAP results.

Pile driveability analysis was performed for a 30m long, 1200mm diameter tubular steel pile with a 1.5m long driving shoe. The pile has a wall thickness of 20mm and the driving shoe has a wall thickness of 40mm. The estimated blowcounts for a ram drop height of 1.5m and a hammer efficiency of 90% are given in Figure 5. The cumulative driving energy, calculated as the blows x drop height x hammer weight for plugged and coring cases, are given in Figure 6. The driveability analysis shows that the pile could possibly refuse at the medium to high strength calcarenite (-22.3 mCD), if driven plugged. Field blowcounts and cumulative driving energy of 9 piles driven nearby to

BH6 are also provided in Figures 5 and 6. Pile 68-P3 is the closest pile to the borehole, and had a blowcount of 570 blows/m (approximately 1.8mm/blow, which may be considered as pile refusal) at about -21.9 mCD. Pile refusal at this depth implies that the pile may have plugged at this stage.

From the results it can be seen that the coring assumption well predicts blow counts during the early part of driving, but under predicts blow counts during the latter part of driving. The fully plugged case over predicted the blow counts during the early part of driving and gave a reasonable assessment of the deepest level at which refusal of the production piles occurred. It is interesting to note that, even though the analysis was based on an upper bound assessment of rock strength and the piles being fully plugged, with no reduction in pile capacity during driving, some of the piles still refused up to 4m shallower than the driveability assessment predicted.

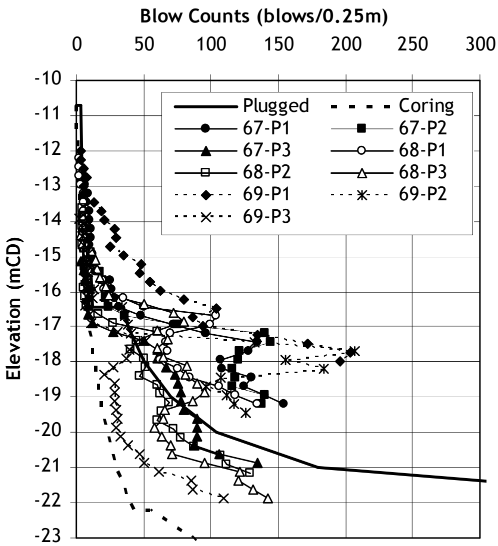

Figure 5 Estimated and Field Blowcounts

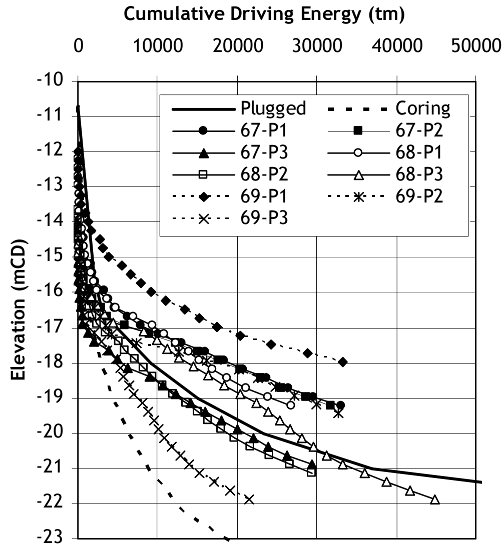

Figure 6 Estimated and Field Cumulative Energy

6 CONCLUSIONS

The following conclusions are drawn from the driveability analysis of steel tubular piles in rock:

- It is probable that the laboratory measured UCS results may be unrealistically low, therefore careful consideration should be given to field strength descriptions to select design UCS values.
- The soil resistance to driving in rock may be estimated using the API RP2A method for determination of pile static capacity. Using this method, the assumption of fully plugged with end bearing equal to 4.5 x 0.77 x UCS appears to give reasonable results for driveability.
- Steel tubular piles can be driven through low strength rock (Is_{50} < 0.3MPa) using sufficiently sized hammers, but it is highly likely that piles could refuse in a medium strength rock.

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