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Design, driveability, driving and dynamic testing of large diameter close ended steel tube piles for a pedestrian bridge

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ABSTRACT: The Matagarup bridge connects East Perth to the Burswood Peninsula and the new Perth Stadium. The bridge is approximately 65 m high, 370 m long with a 165 m long cable stayed central span. The bridge is supported on two river piers and two piers on land. A total of forty-eight 1.05 m and 1.2 m diameter close ended steel tube piles were driven to support the piers.

This paper presents the results of driveability assessments carried out to assess the feasibility of driving the steel tube piles close ended. Closed ends were used because the specification required open steel tubes to be mucked out and filled with reinforced concrete for long term durability. The results of the driveability assessments are compared with the actual pile driving results and the results obtained from end of drive and restrike dynamic pile testing.

1 INTRODUCTION

The State Government of Western Australia has planned a new 60,000 to 70,000 seat major sports stadium and sports precinct at Burswood, Western Australia. The proposed transport infrastructure associated with this sports precinct includes construction of a 370 m long Matagarup Bridge that will cross the Swan River from the East Perth foreshore (western river bank) to the western side of the Burswood Peninsula (eastern river bank). The proposed bridge comprises two river piers and two piers on land. The site and pier locations are shown on Fig. 1.

Figure 1. Site layout

2 GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

The site lies on the sedimentary deposits and rocks of the Swan Coastal Plain and Perth Basin, respectively. During the early Tertiary, sediments of the Kings Park Formation (KPF) were being deposited in a shallow marine to estuarine environment within a major river valley and its tributaries. The KPF comprises sandstone, siltstone or shale and forms the bedrock currently underlying the site. The KPF includes the Mullaloo Sandstone member. This is also present beneath the site and comprises uncemented variably dense sand.

Ancient river channels are termed palaeochannels and review of investigation data and current literature suggest that at least two or perhaps three identifiable palaeochannels exist in the area of the Burswood Peninsula.

Channel infill deposits and associated floodplain deposition when channels overtopped have been divided into two main units called the Swan River Alluvium (SRA) and Sandy Channel Deposits (SCD). The SRA represents the deposition associated with the current river channel but includes deposition from approximately 30,000 years ago to present day and as such, includes deposition that infills the youngest of the palaeochannels. The SCD represents infill deposits associated with older palaeochannels.

Various stages of site investigation were carried out at this site. The tests locations are shown on Figure 1. Figure 2 provides an illustration of the inferred subsurface profile along the proposed bridge centreline.

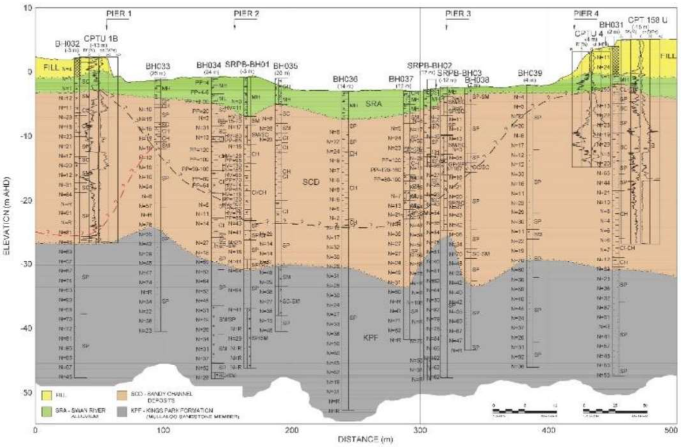


Figure 2. Inferred subsurface profile along the proposed bridge centreline

The general design soil parameters for the site are presented in Table 1 below.

Table 1. Summary of design soil parameters

Geological Formation	Soil Description	Unit Weight, γ (kN/m^3)	Friction Angle, ϕ (°)	Undrained Shear Strength, su (kPa)
SRA	Very soft to firm organic saturated silty clay/clayey silt	15	-	10
SCD – Sandy Zones	Medium dense to dense, fine to coarse grained grey sands and firm to very stiff sandy silts or clays	19	33 – 37	
SCD - Clayey Zones		18	-	30 – 200
KPF	Very dense sands, sandstones and siltstones of the Kings Park Formation	20	40	-

3 DESIGN REQUIREMENTS AND CONSIDERATIONS

The design requirements for this project state that hollow steel tubes are not permitted and must be entirely filled with reinforced concrete. Due to long term durability concerns in the marine environment, the steel tubes must be considered as sacrificial formwork for their full length.

The following pile foundation options were considered for the bridge during design:

- 450 mm square precast concrete piles
- 1200 mm, 1500 mm and 1800 mm diameter bored piles
- 1050 mm, 1200 mm and 1500 mm diameter open ended steel piles with concrete in-fill
- 750mm, 1050mm and 1200 mm diameter close ended steel piles with concrete in-fill

The estimated loads at the bridge piers on land and the river piers are up to about 25 MN and 88 MN respectively. Upon consideration on the advantages and disadvantages of each pile option, final pile loads, constructability and cost, the foundation solution was optimised as follows:

- Two piers on land (Piers 1 and 4): 1050 mm diameter close ended steel piles with concrete in-fill
- Two river piers (Piers 2 and 3) : 1200 mm diameter close ended steel piles with concrete in-fill

Close ended steel piles with concrete in-fill were particularly chosen so that concrete could be directly poured into the piles without having to drill out the SRA from within the piles if they were driven open ended. However, the perceived issues/risks with driving close ended piles were:

- A significantly larger hammer is required to drive the piles close ended to achieve the required pile capacity and depth of fixity. There is a risk that a pile may encounter premature refusal prior to reaching the design penetration depth for vertical capacity and lateral capacity requirements.
- A very rigid base plate would be required to withstand the driving stresses to avoid buckling of the base plate.

Driveability assessment was carried out to assess an appropriate hammer size and to minimise the above risks. This assessment will be further discussed in the Driveability Section.

The final pile configuration of each pier is shown in Figure 3.

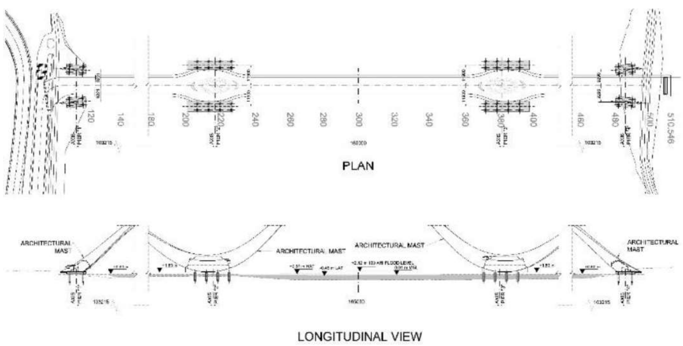


Figure 3. Adopted pile configuration at each pier

The maximum factored pile axial load for on land piers and river piers was estimated to be 5950 kN and 7400 kN respectively. Based on a geotechnical strength reduction factor of 0.77 which was adopted for this project, the required design pile ultimate geotechnical capacity for on land piers and river piers was about 7700 kN and 9600 kN respectively.

Since the 1980s several bridges have been built along the Swan River. The footings of these bridges mostly comprised steel tube piles or composite piles (steel and prestressed concrete piles). A summary of the foundation types and the required pile capacity for these bridges is presented in Table 2.

Table 2. Summary of pile load requirements for bridges built around Perth since 1980s

Reference	Location	Pile Type	Design Ultimate Geotechnical Capacity, Rd,ug (kN)
Jewell et. al (1984)	Mount Henry Bridge, Perth	Lower section: 310 UC 240 steel driven 10 m into siltstone Upper section: 550 mm square prestressed concrete	6,000
Unpublished reference dated 2002	Goongoongup and Windan Bridges, Perth	406 mm dia. x 19 mm wall thickness steel piles driven to dense sand	3,500
Novello and Woodward (2003)	Narrows Bridge Duplication, Perth	610 mm dia. x 12.7 mm wall thickness steel piles driven to dense sand and gravel	5,500 – 6,500 (inferred)

Comparing the pile capacity of the bridges presented in Table 2 with the design pile capacities for this project, it can be seen that design pile capacity has increased over the years, which is likely attributed to increasing sophistication in the structural and architectural design of the bridge structures and the availability of dynamic testing and larger hydraulic hammers.

4 PILE DESIGN

The static pile capacity for the steel tube piles was calculated in accordance with API WSD (2010). The following design assumptions were adopted in the calculation of the static pile capacity:

- The very loose to loose sand layers were not considered in the pile capacity calculation since there are no longer any recommended values for β in these sand layers. The calculated pile capacity is expected to be slightly conservative in the absence of the contribution of shaft resistance in these layers.
- Due to the relatively large pile diameters, it is expected that the contribution of the long term shaft resistance will be significant and that end bearing resistance is not likely to be fully mobilised at service or working loads. For the purpose of assessing pile capacity, a reduction factor of 0.75 was applied to the ultimate unit end bearing resistance. This factor was chosen primarily based on the experience and knowledge of the authors.

5 PILE DRIVEABILITY

For this project, pile driveability assessment was carried out to estimate the pile capacity at refusal and ensure that the selected hammer would have sufficient energy to drive the piles to achieve the required pile capacity and penetration. The software GRL WEAP (2010) was used to assess driveability for the contractor proposed IHC S280 hammer.

For pile driveability assessment, the static soil resistance during driving (SRD) was calculated in order to predict the blow counts and to estimate the mobilised resistance for a given pile penetration per blow (set).

For piles driven in cohesive soils (clay), the unit skin friction during continuous driving was estimated using the stress history approach presented by Semple and Gemeinhardt (1981). The pile capacity reduction factor (F_p) in clay during driving is given by:

$$F_p = 0.5\ (OCR)^{0.3} \qquad (1)$$

where the over consolidation ratio (OCR) is estimated using the equation:
$OCR = (s_u / s_{u,nc})^{1/0.85}$ and $s_{u,nc} = \sigma'_{vo}\ (0.11 + 0.0037\ PI)$; s_u = actual undrained shear strength of clay with a given PI; $s_{u,nc}$ = undrained shear strength of the same clay if normally consolidated; σ'_{vo} = effective overburden pressure and PI = Plasticity Index

The estimation of shaft friction and end bearing resistance in clay and sand layers was carried out in accordance with the API RP2A (1979) method using appropriately selected pile capacity reduction factors. For driveability assessment, the shaft resistance within the very loose to loose sand layers was considered.

The soil quake and damping parameters were used in accordance with the recommendations provided in GRLWEAP. Scaling of end bearing resistance was considered at layer interfaces.

Using the above approach, the mobilised pile resistance based on a 2.5 mm set and 1.0 mm set (practical refusal) using IHC S280 hammer have been assessed and presented in Table 3.

Table 3. Summary of estimated mobilised resistance at each pier

Pier	Pile dia. x WT (mm)	Estimated required $R_{d,ug}$ (kN)	Estimated Mobilised Resistance @ 2.5 mm set (kN)	Estimated Mobilised Resistance @ 1 mm set (kN)
1	1050 x 16	6880	10,500	12,400
2	1200 x 20	9350	11,500	13,700
			9,700	10,500
3	1200 x 20	9600	8,500	10,200
			9,400	10,500
4	1050 x 16	7700	9,400	11,000

* WT – wall thickness

As can be seen from Table 3 results, the IHC S280 hammer appeared to be able to drive the piles to achieve the required pile capacity. At Pier 3 based on a 2.5 mm set, there was a risk that the mobilised capacity would be lower than the required pile capacity. However, given that IHC S280 hammer is able to drive for a prolonged period at low sets, the risk of premature refusal was considered to be relatively low and the IHC hammer was subsequently adopted for pile installation.

6 PILE PERFORMANCE

6.1 *Pile testing results*

A minimum of one pile was dynamically tested using a pile driving analyser at each pier either at End of Drive (EOD) or on Restrike with a waiting time of up to 14 days. The tested piles were driven into the dense to very dense sand layer within either the SCD or KPF. A summary of the test results is presented in Table 4.

Table 4. Summary of estimated mobilised resistance for tested piles

Pile	D (mm)	L_p (m)	Waiting Time, t (days)	Mobilised R_{Shaft} (kN)	Mobilised q_b (MPa)
P1-S-c	1050	26.3	0	2640	6.3
P2-S-h	1200	29.0	4	2645	4.2
P2-S-h		32.0	6	7520	2.0
P3-N-a	1200	23.7	0	3525	6.4
P3-N-a		23.7	14	4600	5.2
P3-N-b		22.3	0	2700	6.4
P3-N-b		22.3	8	4690	4.7
P3-S-f		25.0	2	6100	3.5
P4-S-a	1050	36.0	0	3200	6.5
P4-S-a		36.0	3	8400	2.5

Pile	D (mm)	Q_{shaft} (mm)	Q_{toe} (mm)	D / Q_{toe}	Setup (%)
P1-S-c	1050	1.0	12.0	88	-
P2-S-h	1200	2.5	16.0	75	-
P2-S-h		2.5	12.0	100	
P3-N-a	1200	2.0	14.0	86	30
P3-N-a		2.1	10.0	120	
P3-N-b		1.8	22.0	55	74
P3-N-b		2.2	10.0	120	
P3-S-f		2.5	11.0	109	-
P4-S-a	1050	2.0	11.0	95	162
P4-S-a		2.5	3.0	350	

* N = northern pile cap, S = southern pile cap, D = pile diameter, L_p = penetration length, R_{shaft} = shaft resistance, q_b = unit end bearing, Q_{shaft} = shaft quake, Q_{toe} = toe quake, Setup = $R_{shaft(t>0)}/R_{shaft\ (t=0)}$

The following observations can be made based on the dynamic test results:

- Toe quake was observed to range between about 12 mm and 22 mm for piles tested at EOD. This equates to a ratio of pile diameter to toe quake (D/Q_{toe}) of between about 55 and 90.
- Toe quake was observed to range between about 10 mm and 16 mm for piles tested on Restrike, excluding the very low toe quake value assessed for pile P4-S-a which is likely to be attributed to the significant setup effect. This equates to a D/Q_{toe} ratio of between about 75 and 125.
- D/Q_{toe} generally ranges between 55 and 125 in the dense to very dense sand layer, which is consistent with the GRLWEAP recommendations.
- Shaft quake ranges between 1.0 and 2.5 mm, which is within the recommended values by GRLWEAP.
- The maximum mobilised unit end bearing is about 6.5 MPa, which is about 54% of theoretical ultimate unit end bearing of 12 MPa in very dense sand and about 72% of theoretical ultimate unit end bearing of 9 MPa in dense sand in accordance with API method. The theoretical unit end bearing was not fully mobilised because the toe of the pile was not able to be moved the typical 10% of diameter required to fully mobilise end bearing due to

the relatively significant shaft resistance and hammer energy limitations.

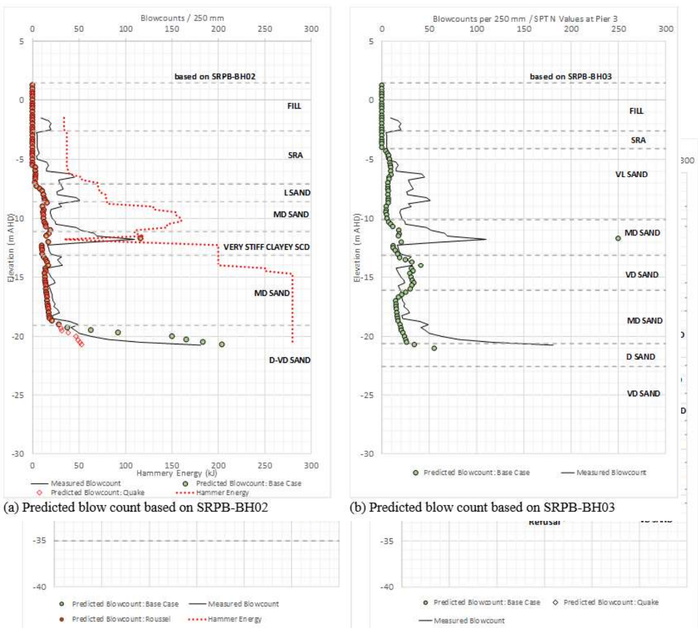
- The amount of setup varies between 30% and 162% for waiting periods between 3 and 14 days. The longest pile has the highest amount of setup.
- Restrike test results indicate that setup also occurs within the sand layers. This observation is consistent with those reported by Jardine and Chow (1996), which is likely attributed to the increase in the local radial effective stresses due to the relaxation through creep of circumferential arching established around the pile shafts.

6.2 *Comparison between measured and predicted blowcounts*

Predicted blow count was calculated based on the actual hammer energy and SRD profiles at selected piers. The following 'driving' parameters were considered for sensitivity analyses:

- Base Case - Soil quake and damping parameters recommended in GRLWEAP (2010)
- Roussel - Soil quake and damping parameters recommended by Roussel (1979).
- Quake - Adopt a toe quake of D/120 to the dense sand layer.

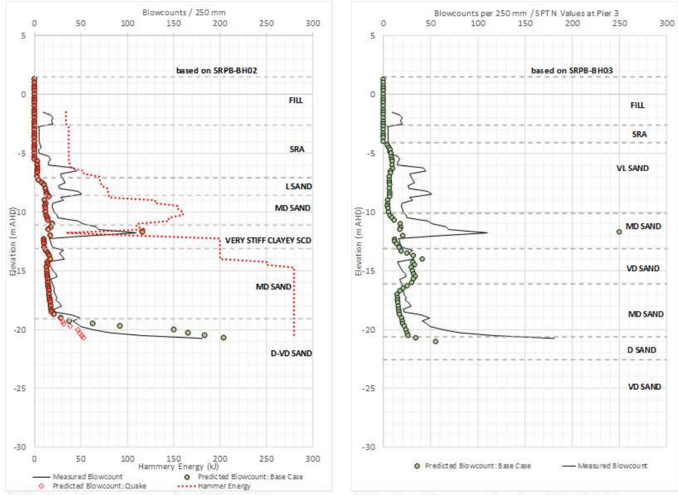
Figures 4 to 6 presents the comparison between measured and predicted blow counts on selected piles at Piers 2, 3 and 4 respectively.



(a) Predicted blow count based on SRPB-BH02 (b) Predicted blow count based on SRPB-BH03

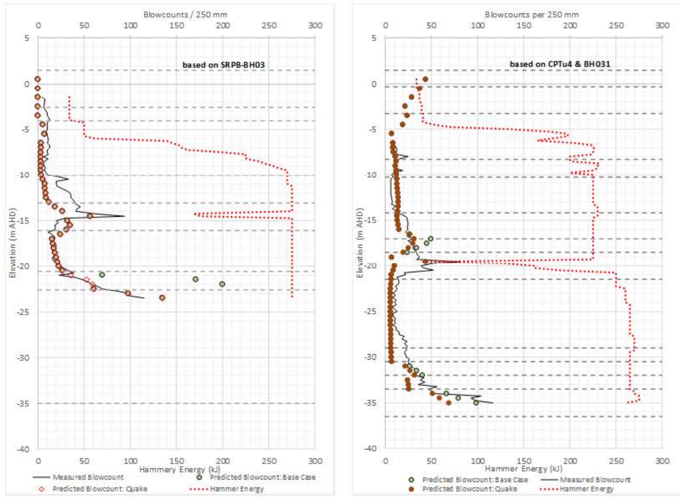
(a) Predicted blow count based on BH34 (b) Predicted blow count based on SRPB-BH01

Figure 4. Comparison between measured and predicted blow counts at pier 2



(a) Predicted blow count based on SRPB-BH02 (b) Predicted blow count based on SRPB-BH03

Figure 5. Comparison between measured and predicted blow counts at pier 3 (North)



(a) Predicted blow count based on SRPB-BH03 at Pier 3 (South) (b) Predicted blow count based on CPTu4 & BH31 at Pier 4

Figure 6. Comparison between measured and predicted blowcounts at piers 3 (South) and 4

The following observations can be made from comparing the measured and predicted blow counts using different sets of driving parameters and reference borehole data:

- Predicted blow counts using base case parameters generally provided reasonable match with the measured blow counts.
- Predicted blow counts using Roussel parameters also appear to provide reasonable match with the measured blow counts.
- Predicted blow counts using a lower toe quake in the dense sand layers seem to improve the match with the measured blow counts in most cases.
- Measured blow counts are mostly higher than predicted blow counts, which indicates either:

- Lower bound value was used in the adopted parameters for driveability assessment.
- The presence of a material layer with a higher strength than assessed.

- Where two reference boreholes were used to predict the blow counts, the measured blow counts generally fall between the predicted blow counts using data from these boreholes. This highlights the likely natural variability of the subsurface conditions over the short distance of the reference boreholes (< 50 m). The subsurface condition at the location of the installed piles is likely to be a combination of the two reference boreholes.

7 SUMMARY AND CONCLUSIONS

The following conclusions are made based on the pile driveability assessment and test results presented in this paper:

- Large diameter close ended piles were successfully installed and tested to the required capacity at the site.
- Hammer selection should be based on sensitivity analyses using multiple boreholes, Roussel parameters for pile driveability and a close ended toe quake of D/120 in dense sand.
- For large diameter close ended steel piles, it is recommended to carry out sensitivity analysis using toe quake ranging between D/120 mm and D/60 mm when founded in the dense to very dense sands to reasonably assess the mobilised capacity for a given set.
- For large diameter (> 1 m) close ended steel piles, the mobilised unit end bearing during PDA testing is likely to be in the order of 55% and 75% of theoretical ultimate unit end bearing in very dense and dense sand layers respectively.

8 ACKNOWLEDGEMENTS

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