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Geotechnical design and construction considerations for Old Mandurah Traffic Bridge Project

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ABSTRACT: The Old Mandurah Traffic Bridge (OMTB) project comprised the design and construction of a new incrementally launched bridge supported on five piers (six span structure). The new structure was constructed to the north of the existing bridge, with a realignment of Old Coast Road and Pinjarra Road required over a length of about 500 m.
This paper presents the foundation design adopted for various components of the OMTB (approach embankments, temporary casting bed, abutments, piers and boardwalk) taking into consideration geotechnical risks, temporary works, and construction constraints which resulted in a holistic design that is efficient and economical. The design is compared with relevant construction records and monitoring test results.
Various pile types used for the abutments, piers and boardwalk included 400 mm square reinforced precast concrete piles and 450 mm and 610 mm diameter close ended steel pipe piles with concrete infill. Rigid inclusions were adopted to support the western approach embankment across a zone of very loose sand.

1 INTRODUCTION

The City of Mandurah (CoM) planned the replacement of the Old Mandurah Traffic Bridge (OMTB) with a realignment of Old Coast Road and Pinjarra Road at Mandurah. The project comprised the design and construction of a new incrementally launched bridge supported on five piers (six span structure). The site location is shown on Figure 1.

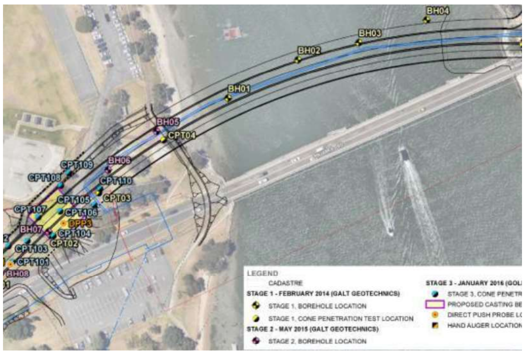

Figure 1. Site layout

2 GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

- Western approach embankment and abutment – Lagoonal Deposits.
- Eastern abutment – sand derived from Tamala Limestone.

Tamala Limestone is shown to occur about 100 m south of the eastern abutment and it was expected that limestone will underlie the sand cover at varying depths on or near to the eastern abutment.

Geotechnical investigation was carried out in few stages which comprised a total of 12 boreholes, 15 Cone Penetration Testing (CPTs) and one hand augered borehole.

The subsurface conditions underlying the site generally comprise variable thickness of very loose to loose SAND/Silty SAND layer overlying interbedded firm to hard CLAY and medium dense to very dense Clayey SAND/SAND overlying weathered Osborne Formation Siltstone (very low strength rock bordering on soil strength).

Figure 2 shows the inferred general subsurface profile along the bridge centreline and Figure 3 shows the inferred subsurface profile along the western approach embankment which better defines the zone of very loose to loose SAND/Silty SAND layer.

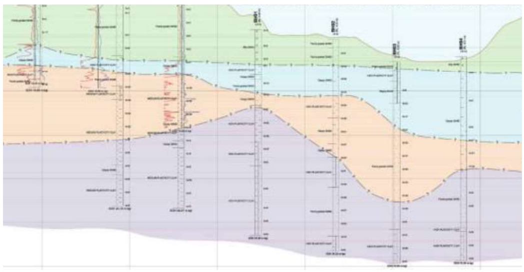

Figure 2. Inferred subsurface profile along the bridge centreline

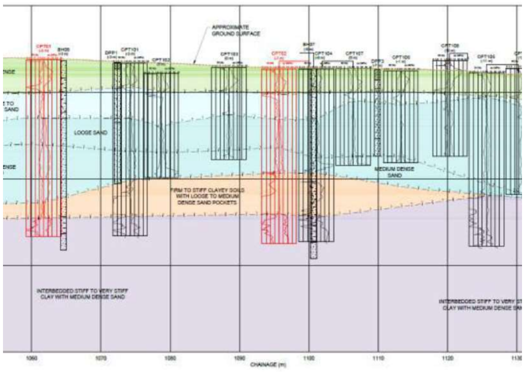

Figure 3. Inferred Subsurface Profile Along the Western Approach Embankment

3 EARTHQUAKE SITE SUBSOIL CLASS

The methods of assessing earthquake risk classification and consequential design implications are outlined in Australian Standard AS 1170.4 (2007).

From Table 6.4 of AS 1170.4 (2007), a spectral shape factor of 1.3 was adopted based on a period of zero seconds and a site sub-soil class of Ce. The hazard factor (Z) depends on the geographic location of the site. The hazard factor (Z) for Mandurah presented on Figure 3.2(D) of AS 1170.4 (2007) is 0.09. Therefore, a peak ground surface acceleration (PGA) of 0.20 g (=0.09 g × 1.3 × 1.7) was adopted based on a return period of 2000 years for the liquefaction assessment. Based on the author's experience with seis-mic hazard assessments for other projects, this PGA value is likely to be conservative. Nevertheless, the PGA value was adopted to confirm the adequacy of the design.

Based on information provided in Burbidge et al. (2012), Cocks et al. (2003), Dismuke and Mote (2012) and author's knowledge of site specific seismic hazard assessments for other structures across Perth, the earthquake magnitude to consider for the purpose of liquefaction assessment for the 100 year design life of the structure is likely to be within the range of 6.0 to 6.5. For the purpose of this assessment, an earthquake magnitude of 6.5 was adopted.

4 DESIGN REQUIREMENTS

The client's 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 form-work for their full length.

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

- 400 mm square precast concrete piles (RC piles)
- 610 mm diameter close ended steel piles with concrete in-fill
- 450 mm diameter close ended steel piles with concrete in-fill

Based on an assessed geotechnical strength reduction factor of 0.72 in accordance with AS2159-2009, the required design ultimate geotechnical capacity (Rd,ug) of the piles at the approach embankments, abutments and bridge piers are about 3500 kN and 4700 kN for RC piles and steel piles 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:

- Western Abutment, Eastern Abutment and Pier 1 : 400 mm square RC piles
- Piers 2 to 5 : 610 mm outer diameter (OD) close ended steel piles with concrete in-fill
- Boardwalk: 450 mm OD close ended steel piles with concrete in-fill

The final pile configuration of the bridge foundation is shown in Figure 4.

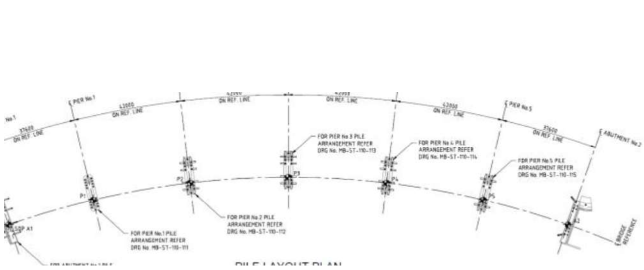

Figure 4. Adopted pile configuration at each pier and abutment

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 soil 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.

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

5 DRIVEABILITY ASSESSMENT

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 without pile overstressing. The software GRL WEAP (2010) was used to assess driveability for the proposed Junttan HHK7A and BANUT SuperRAM 8000XL hammers.

Upper range subsurface profiles have been considered in the driveability assessment. Upper range profile refers to the profile that results in upper range pile capacity for the purpose of driveability assessment. The following upper range soil parameters were adopted:

Hard CLAY (Osborne Formation): Unit shaft friction, fs = 600 kPa, Unit end bearing, fb = 15 MPa

Very dense SAND : Unit shaft friction, fs = 150 kPa, Unit end bearing, fb = 12 MPa

The soil quake and damping parameters were used in accordance with the recommendations provided in GRLWEAP.

The practical refusal criterion used in construction would need to consider the hammer and pile characteristics. For the Junttan HHK 7A hammer, the hammer refusal criterion adopted is 2.5 mm/blow (400 blows/m). For the BANUT SuperRAM 8000XL hammer, the hammer refusal criterion adopted is 1 mm/blow (1000 blows/m).

The mobilised pile capacity based on a 2.5 mm set and 1.0 mm set (practical refusal) have been subsequently assessed and presented in Table 3.

The allowable compression and tensile stresses for both pile types were estimated in accordance with Cl. 7.3.3 of AS2159 – 2009.

Table 3. Summary of Pile Drivability Outputs

Hammer	Pile Size (mm)	Location (BH Reference)	Design Load (kN)	Estimated Mobilised Capacity[b], $R_{d,ug}$ (kN)
BANUT Super RAM 8000XL	610 mm O.D. steel pile x 16 mm WT[a]	Pier 2 (BH01)	4,700	5,100-5,500
		Pier 3 (BH02)		5,400-5,500
		Pier 4 (BH03) Pier 5 (BH04)		5,500-5,800
Junttan HHK 7A	400 mm square RC	Pier 1 (BH05)	3,500	3,500-3,800
		West Abutment (BH06)	3,500	3,600
		East Abutment (BH09)	3,500	3,500-3,800

Notes: [a] Close Ended Pile, [b] Estimated mobilised capacity at recommended hammer refusal, [c] C = compression, T = tension
[d]RC = reinforced concrete, WT = wall thickness

The results of the driveability assessment indicate that Junttan HHK 7A is marginally adequate while BANUT SuperRAM 8000XL is adequate to drive the piles to achieve the required design load.

Due to hammer availability, the contractor adopted Junttan HHK 7S in lieu of Junttan HHK7A to drive the RC piles. The contractor also adopted Junttan HHK 8S in lieu of BANUT SuperRAM 8000XL which has a higher energy and as such has a greater ability to drive the steel piles.

The actual pile installation and pile testing show that:

- 400 mm square RC piles achieved the minimum required design load when driven to a set of 2 mm/blow at a maximum drop height of 800 mm using Junttan HHK 7S.
- 610 mm OD steel piles achieved the minimum required design loads (compression and tension) when driven to a set of between 1.5 mm/blow and 3 mm/blow at a maximum drop height of 1500 mm using Junttan HHK 8S with a certain minimum toe level or pile penetration depth that varies depending on the ground conditions at each pier location.
- Driving stresses were within the acceptable limit.
- Maximum unit shaft friction mobilized within Osborne Formation during testing (at 2 hours restrike) was 300 kPa. It is expected that the long term unit shaft friction will be higher than 300 kPa due to the set up effect.
- Maximum unit end bearing mobilized within Osborne Formation during testing was 11.5 MPa, which was reasonably close to the adopted design value.
- The pier piles achieved a penetration length of between about 14 m and 22 m below the river bed (i.e between about RL -17 m AHD and RL -28 m AHD). The average actual pile penetration length was within about 2 m variation from the estimated

pile penetration length calculated based on the up-per range soil parameters.
- The abutment piles achieved a penetration length of between about 12 m and 25 m below the ground (i.e. between about RL -14 m AHD and RL -23.5 m AHD). The average actual pile penetration length was within about 3.5 m variation from the estimated pile penetration length calculated based on the upper range soil parameters.

6 DESIGN AND CONSTRUCTION CONSIDERATIONS

Main design and construction issues for various structures/locations were considered and discussed in the following subsections.

6.1 *Western embankment approach*

A relatively thick zone of very loose to loose sand layer is present at the western approach embankment (up to 5.5 m high) which is likely to result in settlement that exceeds the allowable limit of 15 mm if no ground improvement was undertaken. The site was also constrained by various existing underground utilities within the western embankment approach footprint that obstruct ground improvement works (refer to Figure 5). Consideration was also given to satisfy the temporary work requirements for the casting bed which is separately discussed in the subsequent section.

To mitigate the settlement issue, few ground improvement options such as Rigid Inclusions, Deep Soil Mixing (DSM) and Dynamic Compaction (DC) were considered. Rigid inclusions option was adopted based on consideration of cost, site and time constraints.

Performance specification for the ground improvement by Rigid Inclusions was subsequently developed to ensure QA/QC measures were implemented and in compliance with the allowable settlement and differential settlement limits. The critical limits of movement are summarised below:

- The maximum settlement of any new road Pavement at any location over the applicable Defects Correction Period of 5 years following construction must be limited to 15 mm.
- Differential settlement of no steeper than 1 in 200 over 5,000 mm length within the transition zone.
- Differential settlement of no steeper than 1 in 500 over 5,000 mm length within the ground improvement zone.
- No steps or lips in the finished surface or within any existing buried service.
- Maximum total vertical movement of 15 mm for retaining structures.
- Maximum rotation of 1 in 400 for retaining structures.

Based on the performance specification, the following rigid inclusions configurations were designed by the ground improvement contractor as follows:

- Column diameter of 270 mm.
- Spacing ranging between 1.7 m and 2.6 m depending on the embankment height.
- Column length of up to 8 m.

Figure 5 shows a general CMC layout for the western approach embankment with various spacing to accommodate different embankment heights.

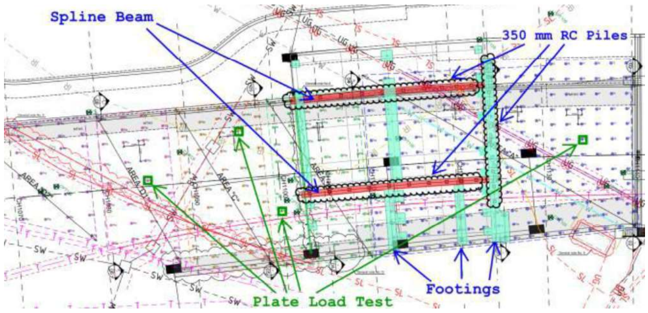

Figure 5. Rigid inclusions and spline beam layout for the western approach embankment

A total of 4 plate load tests at selected critical locations within the ground improvement footprint were carried out and the measured maximum settlement under 120% of the working load was less than 6 mm, which satisfied the performance specification requirements.

6.2 *Temporary casting bed*

The new concrete superstructure was cast and incrementally launched from a temporary bed located behind the western abutment. The specified maximum differential settlement between any two points during launching is limited to 3 mm. Another important consideration is that the casting beam would be removed following temporary works and the proposed piles would need to be designed to have compatible long term settlement with the ground improvement.

In view of the presence of very loose to loose sand layer and the above considerations, the temporary casting bed foundation was designed to comprise two spline beams supported by 350 mm RC piles and associated footings. The pile working load is 500 kN. The piles were driven to a set that was determined from initial dynamic testing. The layout of the temporary casting bed foundations is provided in Figure 5.

Monitoring results showed that the measured maximum settlement at the temporary casting bed during bridge launching was less than 5 mm, which satisfied both the temporary and permanent work requirements.

6.3 *Western river bank*

Stability of the river bank slope during a major earthquake and post-earthquake was assessed. During a major earthquake, a horizontal acceleration of 0.5 PGA was adopted. For the post-earthquake condition, an average undrained shear strength of 0.05 σ'vo was adopted for the liquefied sand layer using the approach recommended by Idriss & Boulanger (2008).

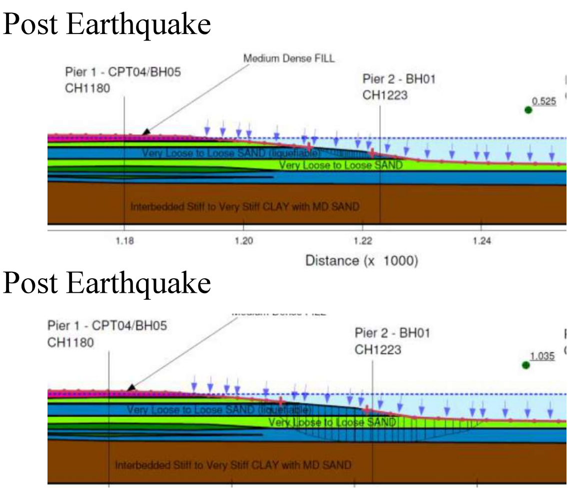

Figure 6. River bank slope stability during and post earthquake conditions

Potential slip surfaces with FOS against slope instability < 1.0 were found to be surficial and localised within the liquefiable layer daylighting in the slope below river water level.

Pier 2 piles may be subjected to lateral forces from the horizontal soil movement occurring following liquefaction during a major earthquake. However, the applied lateral force and induced pile bending moments were assessed to be small for Pier 2 as the liquefied soil would be at or very close to the riverbed and therefore would 'flow' around the piles.

Pier 1 piles may be subjected to more significant lateral forces from lateral soil movement during a major earthquake (lateral spreading of the river bank) due to the presence of an overlying non-liquefiable soil layer above the liquefiable layer. The effects of lateral soil movement on Pier 1 piles had been considered in the pile design.

6.4 *Western abutment*

The western abutment piles may be subject to additional bending moment from the horizontal soil movement due to effects of ground improvement if these piles were installed before the ground improvement was carried out. As such, ground improvement works were scheduled and carried out prior to the installation of western abutment piles to mitigate such risk.

6.5 *Eastern abutment*

Up to about 3 m thickness of fill was placed behind the eastern abutment wall after the piles were in-stalled. This fill placement was likely to initiate some vertical and lateral movement within the underlying loose sand layer and may cause bending moments to be induced in the abutment piles. Additional bending moment induced in the piles due to lateral soil movement resulting from the fill placement was considered in the pile design by using the PGYMY program to apply calculated horizontal soil movement profile obtained from PLAXIS to the pile.

Ground vibration issue was also considered due to close proximity of the business premises at the Eastern Abutment during pile installation. The specified ground vibration limit was 5 mm/s. The following mitigation measures were implemented:

- Carry out vibration monitoring during driven pile installation at the Western Abutment and Pier 1 to infer the level of vibration to be expected at the Eastern Abutment.
- Carry out trial vibration monitoring at the Eastern Abutment in advance of the actual pile installation schedule to assess the risk of pre-existing vibration exceeding the allowable limit of 5 mm/s at the location of the monitored structures.
- Limit the drop height/energy of the hammer depending on the vibration level recorded.
- Excavate isolation trenches to further reduce vibrations travelling through shallow strata. Vibration monitoring results during the actual pile

installation indicated that the measured vibration levels were within the acceptable levels. Isolation trenches were found to be effective in reducing the vibration levels at the business premises induced during the pile installation.

6.6 *Pilecap settlement during launching*

The pilecap settlement at each pier and abutment during launching was required by the structural engineer for the launch design. A suite of loading cases/stages under Serviceability Limit State (SLS) and Ultimate Limit State (ULS) was considered in computing the pilecap settlement using PIGLET software package.

The table below shows a comparison between the calculated and measured maximum pilecap settlement under SLS load at each location. The range of calculated pilecap settlement provided in the table be-low takes into consideration sensitivity analysis for lower range and upper range subsoil conditions.

Table 4. Cumulative pilecap settlement under SLS load at each pilecap location

Pilecap Location	Maximum Cumulative Pilecap Settlement under SLS Load (mm)	
	Calculated	Measured
Western Abutment	9 – 12	10
Pier 1	9 – 12	9
Pier 2	6 – 9	6
Pier 3	8 – 11	4
Pier 4	7 – 10	4
Pier 5	7 – 10	6
Eastern Abutment	7 – 10	2

The measured pilecap settlements are within expectations as measured values are lower than the calculated values.

7 SUMMARY AND CONCLUSIONS

The following summary and conclusions were made based on results of the pile testing and settlement monitoring results:

- Closed ended piles were successfully installed and tested to the required capacity at the site.
- Maximum unit shaft friction mobilized within Osborne Formation during testing (at 2 hours restrike) was 300 kPa. It is expected that the long term unit shaft friction will be higher than 300 kPa due to the set up effect.
- Maximum unit end bearing mobilized within Osborne Formation during testing was 11.5 MPa, which was reasonably close to the adopted design value.
- The average actual pile penetration length varies up to about 3.5 m from the estimated pile penetration length calculated based on the upper range soil parameters. The variance is expected given the inherent variability of the soil stratigraphy and parameters.
- Settlement monitoring results for the western embankment approach indicated that the performance specifications requirements for the ground improvement were met.
- Effects of earthquake-induced soil lateral movement on abutment piles should be considered in the pile design.
- Mitigation measures and vibration monitoring were implemented to address the ground vibration issue at the eastern abutment. The results indicated that the measured vibration levels were within the acceptable levels.

8 ACKNOWLEDGEMENTS

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