PILE FOUNDATION AND APPROACH EMBANKMENT DESIGN FOR CORONATION DRIVE VIADUCT IN BRISBANE

Q.J. Yang¹ and M.Z. Hossain²
¹Director, ²Engineer, Hyder Consulting Pty Ltd, Sydney, Australia

ABSTRACT
The design and construction of the Coronation Drive Westbound Viaduct was a component of the Hale Street Link Alliance project. This paper presents the geotechnical issues and the importance of obtaining sufficient geotechnical information highlighting the key challenges arisen from the concept to detail design developments for the viaduct foundations. The experience gained from the westbound approach embankment in limiting the long term settlement and its interaction with the new sewer rising main is also discussed.

1 INTRODUCTION
The design and construction of the Coronation Drive Westbound Viaduct was a component of the Hale Street Link Alliance project. The viaduct was to be located on the westbound (outbound) carriageway of the existing Coronation Drive, on the northern side of the Brisbane River. The existing westbound carriageway passes through the Hale Street intersection and traffic flow is interrupted by a set of traffic lights at this intersection. The viaduct was required due to the construction of the new Hale Street Bridge (Go-Between Bridge) over the Brisbane River, which traverses the Brisbane River and links the southern side to the northern side of the river at the Hale Street intersection. The viaduct allows an uninterrupted passage for the two outbound traffic lanes of Coronation Drive through the intersection. The viaduct is approximately 225 m in length with six piers and two abutments that are supported on piles with a pile cap. The main viaduct was founded on bored piles to suit the variable ground conditions and the approach embankments were constructed using precast units to achieve the project programme. This paper presents a case study of the geotechnical design methodology and resolution of some challenging issues experienced by the project team.

2 GEOLOGICAL SETTING AND DESIGN PARAMETERS
2.1 GEOTECHNICAL SETTING AND INVESTIGATION
The geological map of the City of Brisbane indicates that the site is underlain by a Quaternary deposit of soft to firm clay and sandy soils overlying Bunya Phyllite of varying weathering degree and strength which generally increase with depth, with deeply incised alluvium valleys of varying depths.

During the concept design development it was inferred that there would be a deep alluvial buried channel of about 23 m depth, with fill depth of up to 6 m along the alignment of the viaduct between the two abutments. It was noted that the subsurface conditions at the abutments comprise 2 m to 3 m of granular soils overlying soft to firm clay overlying weathered bedrock. After a critical review of the available geotechnical information additional cone penetration tests were undertaken to confirm if there would be soft clays at the abutments and their associated approaches. The Alliance proceeded with further investigation of the subsurface conditions for the western approach near Abutment B.

2.2 GEOTECHNICAL PROFILE AND PARAMETERS
A layout plan and a typical geological long section along the viaduct alignment are shown on Figure 1.

At Abutment A the subsurface conditions are indicated to comprise minor depths offilling over weathered bedrock. Granular fill was encountered at depths of 0.5 m to 1.9 m overlying hard / dense gravelly clay / clayey gravel, underlain by very low to low strength Phyllite from depths of 0.9 m to 6.1 m. Free groundwater was observed at 5.8 m depth in BH806 only.

At Abutment B the geological profile identified in the CPTs typically comprised shallow depths of filling over very stiff to hard clay and medium dense to dense sand with bedrock from depths of 3 m to 5 m. These conditions are stiffer than those originally expected from the earlier information.

The geotechnical parameters for the soils for stability and settlement assessment were primarily derived from the correlation of the CPTs and the limited laboratory testing results as well as our past project experience in Brisbane City areas.
A summary of the geotechnical design parameters adopted for pile design is presented in Table 1. Note that as the pile diameter for the viaduct is limited to 0.9 m to 1.2 m as compared to the 1.8 m for the main bridge, the shaft resistance was increased by 25%.

Table 1: Factored geotechnical design parameters for bored pile design

<table>
<thead>
<tr>
<th>Rock Class and Description</th>
<th>Shaft Adhesion (kPa)</th>
<th>Bearing Capacity (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 5 - Extremely Low to Very Low Strength</td>
<td>90</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>Class 4 - Low Strength</td>
<td>210</td>
<td>795</td>
</tr>
<tr>
<td>Class 3 - Medium Strength</td>
<td>300</td>
<td>2000*</td>
</tr>
<tr>
<td>Class 2 - Medium to High Strength</td>
<td>600</td>
<td>7000</td>
</tr>
<tr>
<td>Class 1 – High Strength</td>
<td>1200</td>
<td>15000</td>
</tr>
</tbody>
</table>

*A lower value of 795 kPa was adopted for locations where insufficient borehole length beyond pile tip.

Each rock class considered rock strength, weathering, defects and stiffness, having only one assigned ultimate shaft resistance and end bearing capacity. The main reason for the general categorisation of Phyllite into five classes was that the different categories of rock mass could only be reasonably validated on site during construction as the static and/or dynamic pile load test was not considered to be feasible for this project due to the geometrical and time constraints. Note that the proposed piles were mainly founded on low to medium strength Phyllite although the pile design parameters for better rock classes are provided. The design parameters for embankment stability analysis are summarised in Table 2.

Table 2: Geotechnical design parameters for approach embankment stability analysis

<table>
<thead>
<tr>
<th>Soil/Rock Description</th>
<th>Unit weight (kN/m³)</th>
<th>Effective Cohesion (kPa)</th>
<th>Friction Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill pavement</td>
<td>22</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>20</td>
<td>0</td>
<td>32</td>
</tr>
<tr>
<td>Dense to very dense sand</td>
<td>22</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>20</td>
<td>5</td>
<td>26</td>
</tr>
<tr>
<td>Bedrock</td>
<td>24</td>
<td>100</td>
<td>35</td>
</tr>
</tbody>
</table>

3 VIADUCT FOUNDATION DESIGN DEVELOPMENT

3.1 PILE FOUNDATION CONCEPT DESIGN

The primary constraints for the viaduct foundation design were the design and construction programme, the live traffic on the Coronation Drive and the presence of the soft clay layers in the deep alluvial channel and the pile installation method. The other key factor is the presence of existing sewer main near Pier 6. In addition the potential impact of the approach embankment on the proposed new sewer rising main at the Abutment B must be taken into account.

Three piling options were considered in the initial concept development stage:
1) Precast reinforced driven piles which was not considered to be feasible due its low lateral capacity and the need of the pile cap construction that would cause traffic control issue for the existing busy Coronation Drive;

2) Continuous Flight Auger (CFA) piles were considered by the design team with the advancement of piling technology and monitoring device available to ensure the quality of piles formed. However Queensland Department of Main Roads (QDMR) has had some experience with pile toe which has caused it to not accept CFA piles as bearing piles for structures and

3) Bored cast-in-situ piles were considered feasible for the viaduct foundation in order to minimise noise and vibration effects in the surrounding area and to suit the varying soil and rock conditions along the viaduct alignment. Based on the experience at the AMP Building site on the corner of Hale Street and Coronation Drive, where collapses occurred in certain layers within the soft clay, permanent casings were considered necessary as it is impractical to insert and withdraw temporary casings of the required length.

### 3.2 PILE FOUNDATION ANALYSIS

Pile foundation requires not only a thorough understanding of the superstructure and substructure interaction but also the soil and structure interaction. A series of vertical and horizontal springs values were initially developed by the geotechnical engineer for potential typical horizontal, vertical and bending moment for the proposed viaduct structure. This was then used by the bridge engineer to run the grillage structural model using programs such as Microstran or Stand7. The output from the bridge engineer was then provided to the geotechnical engineer for refinement of the vertical, horizontal and bending moment springs. This structure and soil interaction analysis process enabled the design team to gradually develop the pile foundation and bridge structure detail design. Generally the geotechnical pile foundation analysis results matches well with the structure analysis and the calculated pile loads are within about 5%.

Program DEFPIG developed by Sydney University was employed to analyse the pile group with due consideration of both longitudinal and transverse loadings from the column supporting the viaduct superstructure. A simplified superposition method was used to consider two directional loadings to overcome the limitation of DEFPiG which can only analyse one directional loading at one time.

The calculated single pile ultimate vertical load was ranged from approximately 5400 kN to 15300 kN. The design criteria were set to achieve a serviceability limit state settlement of up to 25 mm and the ultimate limit state settlement of up to 100 mm in the pile group analysis.

### 3.3 PILE VERTICAL CAPACITY ASSESSMENT

Once all the pile foundations were analysed using the above interaction approach an assessment of a single pile design capacity was undertaken to determine the minimum anticipated founding level using the available geotechnical information and the pile design parameters.

A geotechnical strength reduction factor was determined using the procedure set out in AS2159. However a strength reduction factor of 0.6 was adopted for the ultimate shaft resistance while a strength reduction factor of 0.5 was used for the ultimate end bearing capacity. These strength reduction factors were chosen so that the risk of unsound end bearing capacity as experienced in some of the projects would be minimised. The pile socket lengths in rock were computed to be between 5 m and 17 m, with lowest at the abutments, by discounting any contribution from the upper soils which are relatively negligible.

It is worth noting that although the minimum one borehole per pier location was satisfactory after the second round of site investigation the anticipated pile founding level at some of the piles was lower than the drilled borehole depth. As such the required minimum depth beyond the pile tip cannot be satisfied as per QDTMR’s requirement. What we did in the design and construction validation process was to downgrade the rock class so that the required minimum end bearing capacity can be guaranteed. This approach was acceptable to both the project verifier and the QDTMR without the need of pile testing to validate the pile capacity.

### 3.4 PILE LATERAL RESPONSE ASSESSMENT

The pile lateral responses under the various loading conditions have been also analysed by the program DEFPiG. The calculated lateral deflection of a pile group under the worst case combination has been limited to 10 mm at the ground level. The lateral deflection under the ultimate limit state loading has been assessed to be no more than 15 mm.

Pier 2 was supported by 4 numbers of 900 mm diameter bored piles which will be subject to collision loading as per AS5100. The nominal ultimate collision load is to be 2000 kN, acting at 1.2 m above the pavement level at an angle of 10 degrees from the direction of road centreline. Based on the British Standard BD48/93 the pile group lateral response was analysed using an equivalent static load of 25% of the collision load. The calculated pile group lateral deflection was found to be about 8 mm, as shown on Figure 2 (a). We also have undertaken the alternative analysis using dynamic
soil Young’s modulus for this case. It has been found that the calculated pile head lateral movement is to be of the order of 10 mm to 17 mm, as shown on Figure 2 (b) when the adopted dynamic Young’s modulus being about 5 to 3 times of the static one respectively. Based on this analysis it was concluded that the proposed pile foundation was satisfactory even under the collision load for Pier 2.

![Figure 2: Lateral deflection of pile group at pier 2 due to collision load](image)

A collision guard system, as shown on Figure 3, was designed to minimise the hazard in the clear zone as per Section 3.7 of the Australian Road Design Guide.

![Figure 3: Photo of Crash Cushion around Pier 2 approach embankment design](image)
4 APPROPRIATE EMBANKMENT DESIGN

4.1 EMBANKMENT CONCEPT DESIGN

The concept design development for both east and west approach embankments was driven by project programme time constraints. The length of eastern and western approaches is about 39 m and 54 m respectively, with their height being about 4.2 m. The retained embankment is about 9 m wide.

The conventional reinforced soil wall (RSW) was discounted due to 1) the backfill material being required to be stockpiled on site and placed as the construction of the wall progresses; 2) the small footprint of the site which was insufficient space to stockpile the backfill and 3) significant amount of double handling for construction of this type of wall. L-shaped cast in situ reinforced retaining walls were also considered in the concept but it was not further considered due to the need of formwork and safety issue of the work in the close proximity of the live carriageway. Precast wall units together with a stitch pour for the middle section was finally adopted for final design development for the ease of construction and safety in design. A typical embankment section is shown in Figure 4.

![Figure 4: Typical cross-section of the approach embankment](image)

4.2 EMBANKMENT STABILITY ASSESSMENT

As the approach embankments were constructed close to the edge of the existing Brisbane River bank it was important to ensure the additional embankment loading would not cause any excessive movement while maintaining an acceptable factor of safety against the potential deep-seated and shallow slip planes. Both the short term construction stage and the long term operational cases were analysed using program Slope/W for the undrained parameters and the drained parameters respectively. In the stability analysis both circular and non-circular slip planes were investigated using Bishop, Ordinary and Janbu method. The results show that a minimum factor of safety of the river bank against instability would be greater than 1.5. Figure 5 shows the output from Slope/W across the abutment B location.

![Figure 5: Result of embankment stability analysis](image)
4.3 EMBANKMENT SETTLEMENT AND ITS IMPACT ON THE SEWER MAIN

At the western approach there was an existing rising main that was relocated for the construction of the new Coronation Drive Westbound Viaduct. A new rising main was to be a buried flexible DN800PE100PN12.5 pipe and constructed at shallow depth within the footprint of the embankment prior to construction of the embankment. Under the fill embankment some 10 m in from the abutment, the rising main has a vertical bend and the cover decreases to approximately 0.3 m below the existing surface level. The pipe was constructed with a 150 mm thick bedding layer and 300 mm thick side cover and overlay. The embedment material was well-graded gravel as per AS2566.1 1998.

The modulus of the well graded gravel was conservatively assumed to be 3 MPa for assessment of settlements, which is applicable for where there is no compaction and hence this value is considered to be a lower bound. Under the embankment a layer of 150 mm thick, 1.4 m wide compressible material was placed over the pipe embedment material. The stiffness of the compressible layer was taken as 1 MPa. Based on the geotechnical model and the design parameters the settlement profile for the short-term and long-term loading has been calculated using program Plaxis 2D. Six typical cross-sectional models were established for the embankment approach of about 54 m length to evaluate the likely differential settlement along the traffic direction. The height of fill and loading estimated are presented in Table 3. Figure 6 shows a summary plot of the settlements at the ground surface and at the pipe crown along the alignment.

Table 3: Fill height and loading at and beyond Abutment B

<table>
<thead>
<tr>
<th>Section</th>
<th>Location</th>
<th>Test</th>
<th>Fill Height / Loading *</th>
<th>Traffic Loading</th>
<th>Structural Loads</th>
<th>Total Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Abutment B</td>
<td>CPT01</td>
<td>4.65 m / 93 kPa</td>
<td>20 kPa</td>
<td>34 kPa</td>
<td>147 kPa</td>
</tr>
<tr>
<td>2</td>
<td>-10 m west</td>
<td>CPT02</td>
<td>4.25 m / 85 kPa</td>
<td>20 kPa</td>
<td>22 kPa</td>
<td>127 kPa</td>
</tr>
<tr>
<td>3</td>
<td>-20 m west</td>
<td>CPT03</td>
<td>3.13 m / 62 kPa</td>
<td>20 kPa</td>
<td>29 kPa</td>
<td>111 kPa</td>
</tr>
<tr>
<td>4</td>
<td>-32 m west</td>
<td>CPT04</td>
<td>2.10 m / 42 kPa</td>
<td>20 kPa</td>
<td>28 kPa</td>
<td>90 kPa</td>
</tr>
<tr>
<td>5</td>
<td>+5 m east</td>
<td>CPT01</td>
<td>0 m / 0 kPa</td>
<td>0 kPa</td>
<td>0 kPa</td>
<td>0 kPa</td>
</tr>
<tr>
<td>6</td>
<td>+10 m east</td>
<td>CPT01</td>
<td>0 m / 0 kPa</td>
<td>0 kPa</td>
<td>0 kPa</td>
<td>0 kPa</td>
</tr>
</tbody>
</table>

*Based on an assumed fill bulk density of 20 kN/m³. Note the above tabulated values are based on the advice provided by the structural engineer.

Figure 6: Predicted settlement along the west approach embankment

The calculated maximum immediate settlement at the ground surface was indicated to be 37 mm. Creep settlements after 100 years were estimated to be of the order of 3 mm at the abutment and up to 11 mm beyond the abutment. The immediate settlement would occur during the construction of the embankment. The creep settlement is likely to occur after the construction of the run on slabs and pavement. Therefore the differential settlement on the bridge abutment is expected to be of the order of 5 mm. The results also indicate that the greatest curvature of the pipe would occur at the change in loading at the abutments as expected. The curvatures / change in settlements were indicated to be minor from 5 m out from the abutment and under the embankment from 20 m back from the abutment. The maximum gradient is estimated to be less than 0.9%, which is about 35 mm over 5 m length. The long term vertical settlement of 35 mm is well within the tolerable limit of 7.5% for the pipe as per AS2566.1:1998.
5 RESULTS AND DISCUSSION

One of the important aspects was to use the permanent sacrificial steel tubular casing for successful installation of bored piles. This avoided any potential collapse of the soft soil into the drilled shaft to ensure the quality of concrete within the bored pile. The other factor was to ensure the casing was driven to sufficient depth so that there would be limited groundwater inflow into the drilled hole. The site geotechnical engineer and the contractor made every effort to control the groundwater inflow so as not to cause any problem for the durability of the formed bored pile.

A site validation procedure was developed for the site geotechnical engineer to use as guidelines for rock quality assessment and determination of the pile final founding level. The key item considered was the method of assessing the classes of the Phyllite which was dependent upon the returned material assessment using the experience gained from similar projects as well as the profile from the available borehole logs at the particular pier location. Where the rock quality improved then point load testing on the returned chips was undertaken to evaluate the rock strength. The other aspects were to ensure the shaft roughness was achieved by scrubbing of the drilled shaft and cleanliness of the base prior to pour of the concrete by tremie technique.

The fact that there is no sign of distress and/or cracking on the pavement after construction is good testimony that the ground movements are within the tolerable bounds although no specific monitoring points were installed for the viaduct works. Figure 7 shows snapshots during pile construction.

6 CONCLUSIONS AND RECOMMENDATIONS

The design construction of the Coronation Drive viaduct imposed a number of geotechnical challenges and constraints. The additional site investigation for the approach embankment enabled the establishment of a realistic geotechnical model for design and construction to meet the long term settlement requirement of the project. The use of bored cast-in-situ pile foundation with permanent sacrificial steel casing for viaduct structure has proved to be successful to avoid any collapse of the drilled hole and groundwater inflow while achieving the high quality of the tremie concrete during construction. The consideration of the generalised rock mass classification was found to be practical during pile installation. The excessive lateral deflection due to collision load was successfully reduced by the use of a crash cushion. The vertical and differential settlements around the sewer main under approach embankment were found to be within acceptable limits. The refined geotechnical model together with rigorous settlement analysis enabled the Alliance to have the high level of confidence in the adaptation of the precast retaining walls for the approach embankments. This is one of the factors which allowed completion of the viaduct about 2 months ahead of the originally allowed programme.

7 ACKNOWLEDGEMENTS

The Brisbane City Council is acknowledged as the client of the project, whose vision and commitment to the project was valuable in ensuring its success. All members of the Hale Street Link Alliance who assisted in the design and construction of the bridge are greatly appreciated.

8 REFERENCES

AS2566.1 (1998) Buried flexible pipelines- Part 1 Structural design