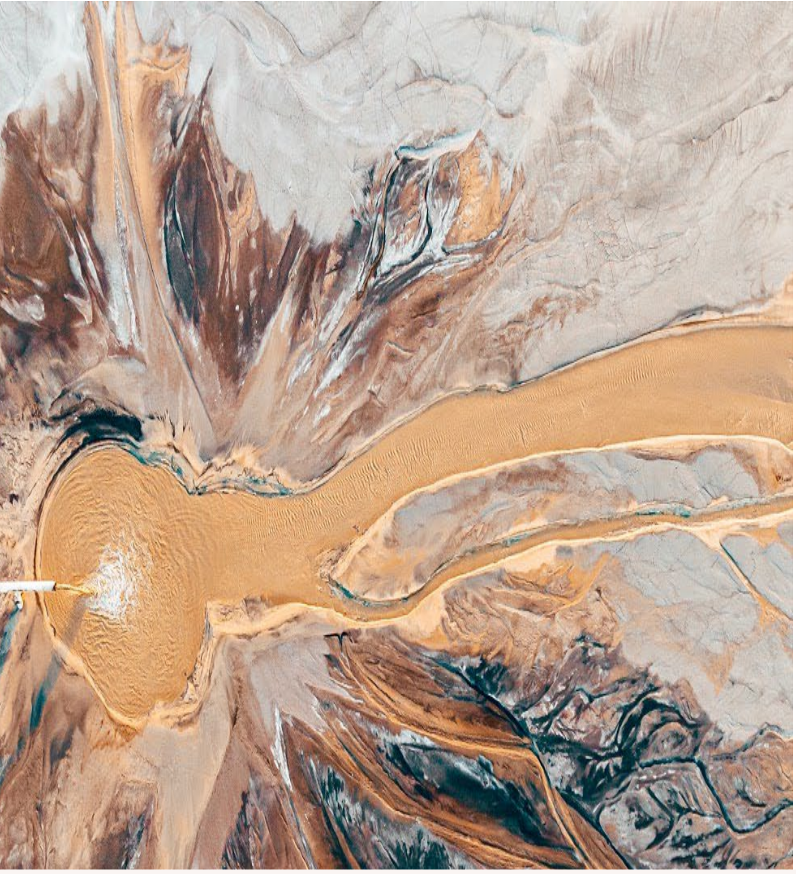
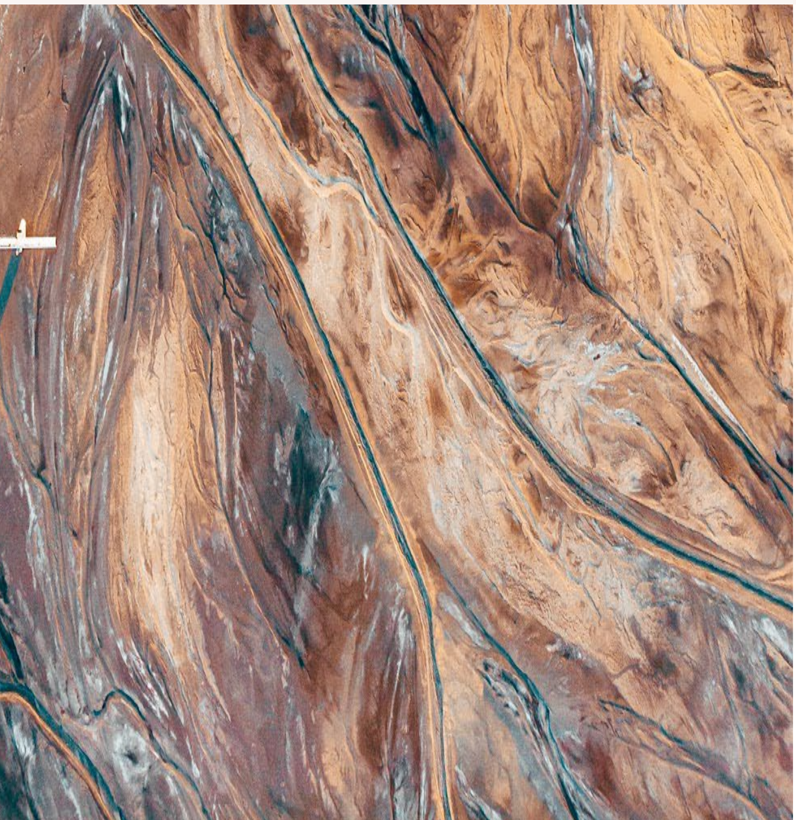


redfox

White Patch Esplanade

Preliminary Design Report



www.redfoxadvisory.com

Document Control

Project			
Document Title	White Patch Esplanade – Preliminary Design Report	**Document No.**	01016-RFA-01-CI-RPT-001
Prepared by:	Geoff Blundstone/Kevin Cheng/Jill Bardos	**Signature:**	
Reviewed by:	Marty Scrogings	**Signature:**	
Approved by:	Michael Price	**Signature:**	

Issue Summary

Revision	Date	Issue Description	Distribution
0	09/09/22	Preliminary Design Submission	MBRC

Contents

Figure Index

Table Index

Limitations

This Report has been prepared based on the representations and information provided by Moreton Bay Regional Council. This Report was prepared in accordance with and is made available subject to the terms and conditions of the Contract under which Red Fox Advisory has been appointed and the particular instructions of the Client. No individual, entity, or other body may rely on any aspect of this Report unless that individual, entity, or other body is the Client. Reliance on this Report is subject to the terms and conditions in the Services Contract including the limitation of liability.

This Report is subject to the following limitations and disclaimers:

a. This Report has been prepared solely for the purpose set out in the Contract under which Red Fox Advisory has been engaged.

b. To the maximum extent permitted by law no warranty, express or implied, is given by Red Fox Advisory and no duty of care is owed by Red Fox Advisory to any person other than the client in connection with the professional advice contained in this Report.

c. To the maximum extent permitted by law and subject to the requirements of the Corporations Act 2001 (Cth), no other warranty, expressed or implied, is made as to the professional advice included in this Report.

d. Red Fox Advisory does not and cannot represent that all issues relevant to the matters investigated have been identified in this Report. The Report has been prepared within the time and budget context of its engagement by the Client and utilising the information available to it at the time.

1 Introduction

1.1 Background

This document has been prepared to summarise the design developments made during the White Patch Esplanade rectification project Preliminary Design Phase. Moreton Bay Regional Council (MBRC) have commissioned Red Fox Advisory to undertake planning and design of the White Patch Esplanade Causeway Reconstruction including all necessary associated works and approvals in accordance with the Specification for Services.

The works include rectification of existing causeway structure that has been washed away during 2 separate flood events in the past 15 years. The most recent was in February 2022, which completely washed out the centre of the causeway. Under state and local authority schemes, build back from natural disaster is to be completed within 24 months of the event occurring, which has driven the compressed timeline to undertake design and approvals for the project.

1.2 Purpose

The purpose of this report is as follows:

- Document the design of the crossing and discuss relevant issues encountered including issues that are not specifically covered in Australian Standards and DTMR design specifications
- Provide a historical record as a design reference

This report represents the design completed up to and including the Preliminary Design phase for the White Patch Esplanade rectification project.

A hydraulic analysis of the waterway is currently being undertaken by WMS, with preliminary hydraulic output data being used for the Preliminary Design phase. Once the detailed opening geometry is finalised, the corresponding final hydraulic analysis will be completed and the results reported and incorporated into the design as part of the following design phase. The final hydraulic output will also include the possible effects of future climate change.

2 Site

2.1 White Patch Esplanade

The project location is situated at White Patch Esplanade, Bellara with the structure crossing Wrights Creek. This is on the North-western side of the island, North of the Bribie Island township.

White Patch Esplanade is the only formalised road link that connects White Patch to the main community of Bribie Island. This road services approximately 75 residential dwellings as well as being the access to many 4x4 tracks on Bribie Island.

The proposed reconstruction will see the existing crossing removed from use and replaced with an offline single span bridge option that was selected through the planning and options analysis (Doc. Ref 01016-RFA-00PM00-RPT-OA-01).

Figure 2-1 Locality Plan

3 Structural Design Basis

The proposed White Patch Esplanade bridge is designed to AS 5100 and generally in accordance with TMR's Design Criteria for Bridges and Other Structures (2021). The specific standards utilised, and the design criteria chosen for the bridge design are as detailed within the Basis of Design Report – Refer Appendix A – Basis of Design Report Rev .

3.1 Matters For Resolution

Matters for Resolution listed in AS5100 (2017) and any proposed deviations from AS5100 are outlined in Appendix C – Matters for Resolution (AS 5100.1).

3.2 Flood Hydraulics

A hydraulic analysis is ongoing at the time of writing this preliminary design report with the current findings of both flood level and velocity being incorporated into the design. Both the bridge structure and abutment scour protection have been designed for the 0.05% AEP (2000yr ARI) flood event. The interim flood design parameters are summarised below with the corresponding hydraulic static forces on the superstructure and substructure determined in accordance with AS5100.2. The provided velocities and flood levels in Table 3-1 are based on preliminary models utilizing an initially proposed channel cross section geometry.

Table 3-1 – Interim Flood Hydraulic Design Criteria (For 18m Single Span Bridge)

Event	Maximum Velocity (m/s)	Flood Level (m AHD)
Q100 Flood Event – without Climate Change Effects	1.48 nom.	0.8 nom.
Q100 Flood Event – with Climate Change Effects	TBC	TBC
Q2000 Flood Event – without Climate Change Effects	4.2 nom.	2.11 nom.

3.3 Scour Effects

Scour protection to bridge abutments following flood events has been considered following the preliminary hydraulic study output. A full scour assessment will be undertaken following geotechnical results from testing along with finalised hydraulic modelling. This will further inform the scour protection and bridge substructure design.

4 Bridge Structure

4.1 General Arrangement Development

The proposed White Patch Esplanade 18m single span bridge carries two lanes of traffic along White Patch Esplanade. The carriageway has a width of 8m with two 3.5m lanes and 0.5m shoulders. The deck has a two-way 2.5% crossfall and a longitudinal grade of 0.5%. At the lowest end, the deck level is 2.5m AHD at the bridge control line. The bridge contains a pedestrian/cyclist shared user path on one side which is 2.5m wide to a balustrade positioned on the outside kerb of the bridge. A typical cross-section of the superstructure arrangement is shown in

Figure 4-1.

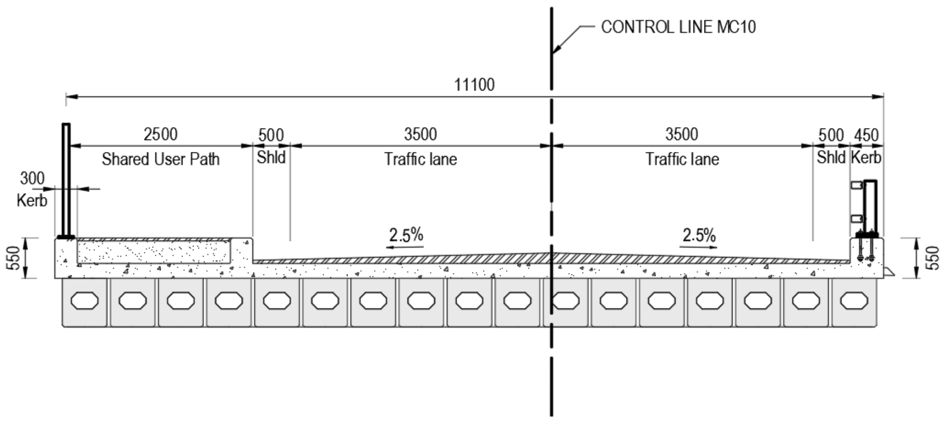


Figure 4-1 - Superstructure Typical Cross-section

4.2 Superstructure

4.2.1 Arrangement

The superstructure is to be composed of:

- A 40mmm thick minimum deck wearing surface as specified within Section 8.6
- A 200 mm thick minimum concrete deck slab
- 650 mm deep x 596 mm wide prestressed concrete deck units. The lateral load distribution between deck units is achieved by the cast in-situ structural topping slab.

4.2.2 Articulation

The deck units on abutments shall be supported on elastomeric bearing strips (cut to size to support units). A Granor XJS expansion joint system or approved similar product is utilised at each abutment. All girders are pinned at each abutment.

4.2.3 Kerbs/Barriers

A 450mm wide kerb profile has been adopted for Preliminary Design along the southern edge of the bridge carriageway for supporting the traffic barrier. Two 300mm wide kerbs have been adopted on the northern side of the carriageway and below the pedestrian balustrade to enclose the asphalt paved shared user path. A typical cross-section of the current design arrangement is shown in

Figure 4-1.

4.3 Substructure

4.3.1 Piles

The substructure consists of two abutments with reinforced concrete headstocks supported on three 550mm diameter octagonal piles. A basic view of the pier substructural elements is shown in Figure 4-2.

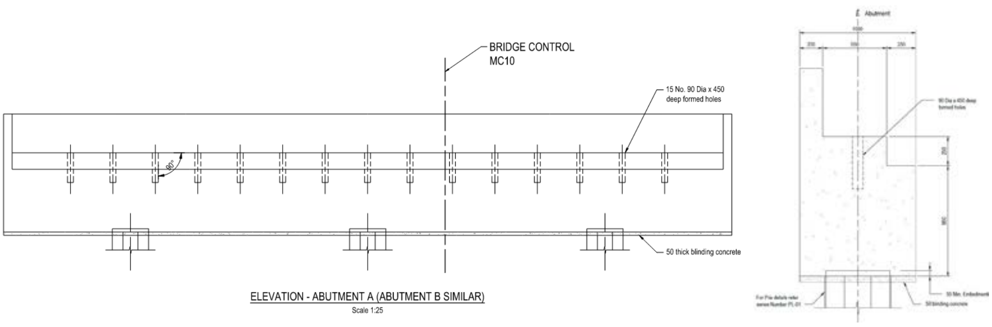


Figure 4-2- Substructure Cross Section

4.3.2 Abutment Headstocks

Abutment headstocks shall be 1200 mm deep x 1000 mm wide cast in-situ reinforced concrete. Each abutment headstock shall have a 250 mm deep x 250 mm wide jacking shelf on each side to facilitate future bearing replacement. Abutments shall have the same jacking shelf on the exterior face.

4.4 Abutment Scour Protection

The Preliminary design Abutment and Causeway Scour Protection has been based on the Q2000 flood velocity and comprises of:

- Rock revetment on top of geotextile
    - Rock sizing: D50 @ 0.7 m with grading to be specified in the following design phase.
    - Total Rock protection thickness: 1.25 m min.
    - Slope: 1V:1.5H
    - Toe width: 2.5 m

- Compacted select/rock fill:
    - Quarry run or other specified fill material to be specified in the following design phase.

5 Hydraulic Assessment

As discussed in Sections 1.2 and 3.2 above, the Preliminary Design Phase has utilised the preliminary model output developed by WMS based on an initial waterway cross section geometry. There have been minor changes to the cross-section geometry following a more detailed assessment of the existing causeway demolition and subsequent channel bed profile. Updated results of the hydraulic assessment will be provided and implemented in the Detailed Design Phase submission.

6 Geotechnical Considerations

The scope of the geotechnical design includes:

- Interpretation of the available geotechnical investigation to develop design parameters
- Geotechnical assessment of the proposed bridge foundation including axial capacity, lateral stability and settlement
- Assessment of the stability of the approach embankments
- Assessment of the anticipated settlement of fill embankments
- Advice for the design of the pavements
- Constructability advice for the proposed design solutions

6.1 Geotechnical Design Criteria

A summary of the relevant standards and criteria are in the Basis of Design Report refer Appendix A – Basis of Design Report Rev .

6.2 Available Geotechnical Information

6.2.1 Desktop Study

Bribie Island and surrounding areas formed as a result of significant sea level fluctuations that have occurred in the recent geological past. Tidal movements continue to heavily influence the Bribie Island shoreline, and the depositional environment representative of a high-energy tidal channel (i.e. larger side materials including sand and gravels).

The 1:100k geology maps indicate that the site is underlain by:

- Qpe: Estuarine deposits, including estuarine channels and banks (sandy mud, muddy sand, minor gravel).
- With adjacent areas underlain by Qpcb (2 and 3): Beach ridges, sand and shelly sand.

An extract is shown in Figure 6-1 .

The underlying geology of the area is sandstone of the Landsborough sandstone formation, at varying depths approximately 6m in the north of Bribie Island to 26m in the south-east.

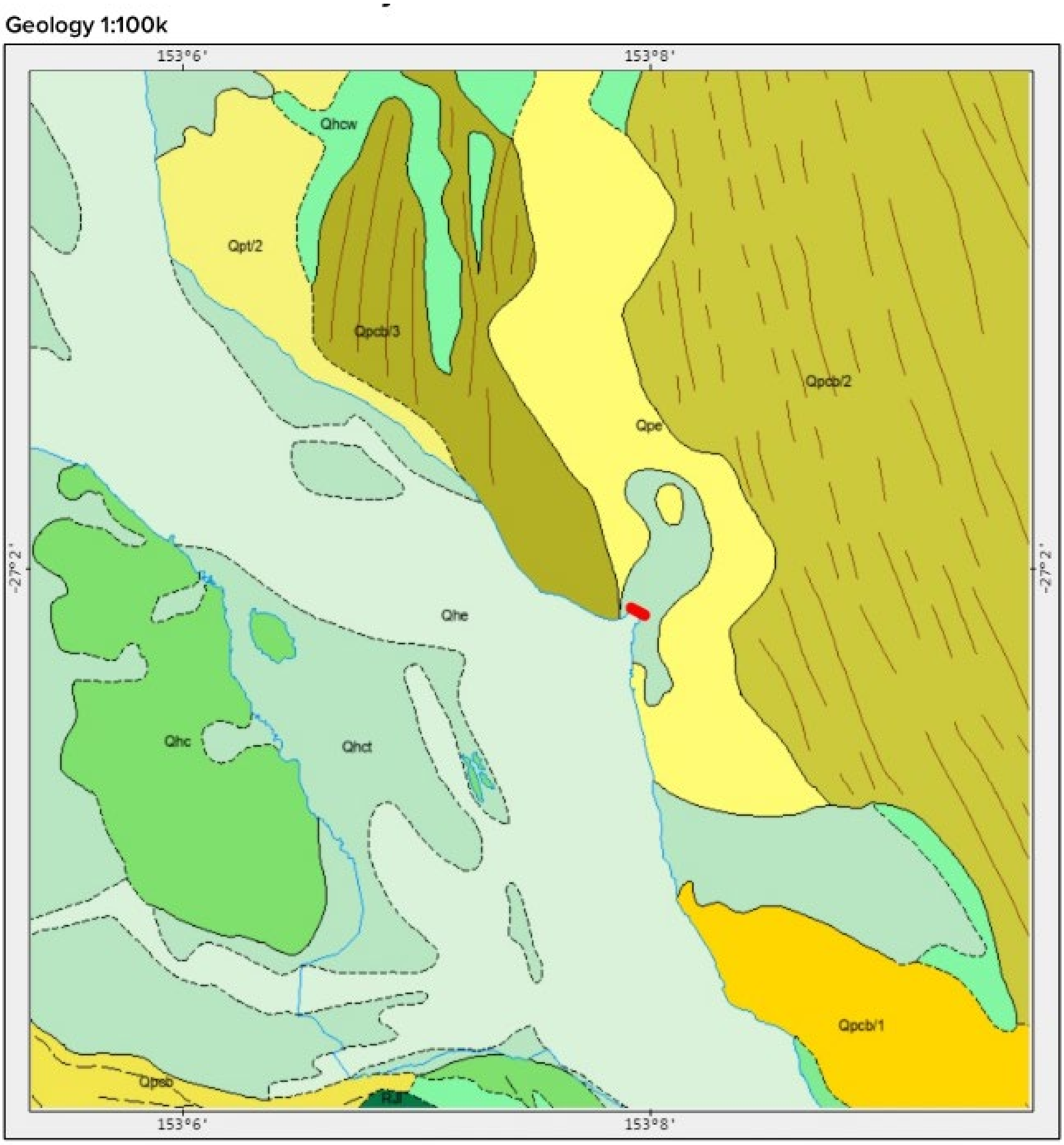


Figure 6-1 - 1:100k Geological Summary; White Patch Causeway is shown as the red line, 1986 (accessed July 2022, Queensland GeoResGlobe).

6.2.2 Previous Geotechnical Investigation

One geotechnical investigation was made available for this stage of design, undertaken by Core Consulting in March 2022 for the utility crossing along the causeway (Report Number J001552-002-R-Rev0). The investigation comprised three geotechnical boreholes, two along the western portion of the causeway and one along the eastern portion. A summary of the borehole locations and termination depths are in Table 6-1, and proximity to the site is shown in Figure 6-2.

Table 6-1 - Summary of CORE geotechnical investigation locations.

BH ID	Description	Northing	Easting	Termination Depth (mBGL)
CORE-BH1	Existing BH	7009523.3	513257.3	Drilled to 15.45mBGL
CORE-BH2	Existing BH	7009613.9	513050.5	Drilled to 15.45mBGL
CORE-BH3	Existing BH	7009596.5	512957.2	Drilled to 15.45mBGL

The previous boreholes were drilled to a depth of 15.45m below ground level (mBGL) and comprised variable density alluvial sands. Weakly cemented to cemented coffee rock was encountered in all three boreholes.

BH2 and BH3 showed medium dense to dense sands to approximately 13mBGL, and then very loose to loose sands until termination. BH1 indicated medium dense to very dense sands, becoming medium dense again around 12mBGL.

Figure 6-2 - Summary of existing boreholes and location to the project site (Core Consultants, March 2022).

The results from the acid sulfate soil (ASS) testing indicate that of the 24 samples analysed, the net acidity exceeded the relevant QASSIT 'Action Criteria' (for bulk earthworks) in 21 samples. This indicates that management and/or lime neutralization treatment will be required if these soils are disturbed during the construction.

6.2.3 Planned Geotechnical Investigation

Red Fox Advisory are currently in the process of executing a supplementary geotechnical investigation to provide additional information closer to the abutment piles and to extend beyond the pile toe. A summary of the proposed geotechnical investigations is shown in the summary tables and figures below.

Figure 6-3 - Proposed additional investigation and the existing geotechnical investigation from Core (March 2022).

Table 6-2 - Summary of proposed geotechnical investigation locations for the proposed 'offline' bridge alignment option.

BH ID	Description	Northing	Easting	Termination Depth (mBGL)
BH1	Eastern abutment	7009569.873	513106.725	25
BH2	Western abutment	7009561.202	513123.968	25
BH3	Western approach	7009599.087	512999.127	5
BH4	Western approach, towards bridge	7009593.319	513058.223	10
BH5	Eastern approach, towards bridge	7009540.177	513165.781	10
BH6	Eastern approach	7009507.244	513249.399	5

Table 6-3 - Summary of proposed laboratory testing; to be confirmed upon completion of the additional investigation.

Test Type	Estimated Quantity
Atterberg Limits including Linear Shrinkage	TBC
Particle Size Distribution	10
Moisture Content	10
CBR (soaked, 10-day)	4
Maximum Dry Density (standard compaction)	4
Optimum Moisture Content	4
Direct shear box (single stage)	4
Uniaxial Compressive Strength Test	TBC
Point Load Index Test (Axial and Diametral)	TBC
Soil Aggressivity Suite (moisture content, pH, SO4, Chloride, Resistivity)	6

6.3 Preliminary Geotechnical Model

Based on the existing geotechnical information available, a preliminary geological model has been developed for this stage of design development. This model will be revised with the additional geotechnical investigation.

The model is summarised in Table 6-4, and based on the "worst case" stratigraphy.

Table 6-4 - Geological model used for preliminary design.

Top [mAHD]	Bottom [mAHD]	Material	Density	Unit Weight, γ [kN/m3]	Effective Friction Angle, θ' [°]	Young's Modulus, E [MPa]	Poisson's Ration, v [-]
+0	**-4**	Alluvial Sand	Loose to Medium Dense	18	30	6	0.25
-4	**-7**	Alluvial Sand	Medium Dense	19	33	15	0.25
-7	**-13**	Alluvial Sand	Dense	19	36	40	0.3
-13	**NE**	Alluvial Sand	Very Low to Low	17	28	3	0.25

6.4 Seismicity

The hazard factor (Z) of the site is taken as 0.08 based on AS1170.4-2007, from Figure 3.2(F), the Hazard Design Factor (Z) for Queensland. Additional seismic design parameters used for the geotechnical assessment include:

- Earthquake Design Category: TBC
- Annual Probability of Exceedance (P): 1/500 (i.e. a 10% probability of exceedance in 50 years)
- Probability Factor (kp): 1.0.

The hazard factor is equivalent to an acceleration coefficient with an annual probability of exceedance of 1/500 (i.e. a 10% probability of exceedance in 50 years).

7 Geotechnical Design

7.1 Bridge Foundations

The preliminary design of the bridge comprises a single-span across the causeway, supported by two abutments and approach embankments. It is proposed to support the abutments with deep foundations. The preliminary assessments include a review of:

- 3 no. 0.9m diameter bored concrete piles, supported by steel casing; or
- 3 no. 0.55m diameter driven octagonal piles.

The second alternative (driven PS octagonal piles) will be assessed as the preferred design with the bored pile option assessed if costs or construction dictate.

7.1.1 Available Geotechnical Information

The available geotechnical information is summarised in Section 6.

7.1.2 Pile Design and Analysis

At this stage of design, the assessment included:

- Undertaking preliminary assessment of pile depths based on a provisional axial load of 7000 kN per abutment (Dead Load + Live Load, factored by 1.5); and
- Generating a lateral assessment on a single pile to derive springs for use in the structural analysis.

Settlement was not assessed at this stage.

7.1.2.1 Axial Capacity

The axial capacity of the piles was assessed using a combination of in-house spreadsheets and AllPile for verification. For the preliminary analysis the following assumptions were made:

- The models assumed a medium dense material at -18mAHD underlying the loose to very loose sandy material.
- For the bored pile option, the thickness of steel was ignored.
- For the driven pile option, densification around the pile tip was ignored.
- For both the bored and driven options, long-term softening was ignored.

The following pile quantity and depths to support the preliminary loading were:

- 3 no. 0.55m driven octagonal piles to approximately 10m below the ground surface.

It should be noted that depth and quantity of piles may change due to additional geotechnical investigation or revisions to the proposed bridge loading.

7.1.2.2 Lateral Stability

A provisional model was generated in Oasys Alp to generate springs for the structural modelling. A 100kN lateral load was applied to each pile head, and the deflection (mm) vs generated soil pressure (kPa) was exported at 0.5m intervals along the pile depth.

7.1.3 Bridge Foundation Constructability Considerations

To be further described in subsequent design phases.

7.2 Approach Embankments

The approach embankments for the bridge are relatively low on the northern side and are proposed to be flat (1V:6H) whilst the embankments on the northern side will range from 4:1 up to 2:1 for the higher embankments.

7.2.1 Assessment Methodology

7.2.1.1 Slope Stability

The slope stability analysis will be carried out using a working stress approach with target Factors of Safety (FoS) based on current best practice and past experience in designing cut slopes in similar materials. In this approach, all load factors and strength reduction factions are 1.0 (i.e. no reduction in soil strength and no increase in applied loads). A summary of the methodology to be used in the embankment slope assessment is summarised below.

1. Development of a representative geotechnical model for the critical sections of the cut slope (i.e. highest cut section, minimal slope angle, and/or weakest geotechnical materials) by interpretation of the geometric design and available geotechnical investigation information;
2. Development of the associated geotechnical design parameters for the materials encountered, such as: unit weight γ, and effective friction angle (θ’);
3. Undertake the analysis on the critical sections using the limit equilibrium software Slope/W to assess the FoS (ratio of resisting to disturbing forces) against circular failure mechanisms for the permanent long-term and temporary short-term design considerations;
4. Review the minimum FoS achieved in the analysis and update the design sections if required.

7.2.1.2 Settlement

Settlement from the fill embankments will be assessed using a 1-dimensional consolidation approach. Due to the existing stratigraphy being sand and the relatively low heights of the fill embankments, post-construction settlement will be negligible. Long term creep is not envisaged to be an ongoing impact for the proposed design.

7.2.2 Slope Stability Design Cases

The adopted design cases, corresponding assessment parameters and minimum Factor of Safety (FoS) required are summarised in Table 7-1.

Table 7-1 - Design cases for the bridge approach embankments.

Design Case	Assessment Parameters	Minimum FoS
Permanent, Long-term	Drained soil conditions using effective strength parameters. No additional surcharges.	1.5
Rapid Draw-down, Short-term	Case to simulate a flood event with rapid increase and decrease in water pressure. No additional surcharges.	1.2
Over-excavation, Short-term	Total stress strength parameters, nominal 0.5m additional cut in front of the toe of the cutting. No additional surcharges.	1.2
Temporary construction, Short-term	Total stress strength parameters, a 10kPa loading is applied to the top of the embankment to simulate light construction loading	1.2
Seismic, Short-term	Total stress strength parameters, a horizontal acceleration will be applied to simulate a seismic event.	1.2

7.2.3 Critical Sections

The analysis will be undertaken on critical sections, with are assessed based on the largest fill height, steepest slope and/or weakest subgrade. The critical sections will be assessed in subsequent design stages when the civil earthworks are further developed.

7.2.4 Results and Conclusions

Outputs of the slope stability and settlement assessments will be presented here in subsequent design submissions.

7.2.5 Embankment Constructability Considerations

To be further described in subsequent design phases.

7.3 Material Re-Use

To be further described in subsequent design phases.

7.4 Pavement Design

Based on the existing geotechnical investigation, a medium dense alluvial sand material is assumed to the in-situ founding layer for the pavements.

A California Bearing Ratio (CBR) of 5% has been assumed for this material for the preliminary pavement design. Values of CBR following testing are expected to be of the oreder of 5 to 12%. Additional CBR testing is scheduled for bulk samples obtained from the additional geotechnical investigation.

7.5 Auxiliary Structures

This section will be populated in subsequent design phases, and include reviews for:

- Lighting and power foundations (if required); and
- Advice on pavements for pedestrian pathways.

8 Civil Works

8.1 General

The design objectives for the civil works are as follows:

- Maintain connectivity with the existing road network
- Ensure geometry meets current design standards by maintaining or improving the existing alignment where needed
- Ensure roadworks lengths are kept to an acceptable minimum to service the bridge crossing
- Ensure road formation is suitable for design flood events

8.2 Design Standards

The design is in accordance with MBRC, TMR and Austroads guidelines where possible. The specific standards utilise for the design are as detailed within the Basis of Design Report refer Appendix A – Basis of Design Report Rev B.

8.3 Design Criteria

Refer to Appendix A – Basis of Design Report Rev B for Preliminary Design criteria.

8.4 Geometric Details

8.4.1 General

The design vehicle for the project is a 12.5m Heavy Rigid Vehicle, and the check vehicle is a 19m Prime Mover and Semi-Trailer.

8.4.2 Design Speed and Posted Speed

Section 8.3 details the design criteria consistent with a design speed of 70km/h, with a posted speed of 60km/h.

This design speed for the proposed alignment was adopted based on the existing posted speed in the project area.

8.4.3 Horizontal Alignment

Horizontal curve radii have been selected to meet a 70km/h design speed, minimize the width of the crossing and minimize the footprint of new construction. The proposed alignment deviates off-line from the existing road for approximately 365m (including 18m of bridge) before returning online to the existing formation. This constitutes the current total length of the new approach roads.

The tangent which makes the crossing of Wrights Creek is positioned as close to the existing formation as possible while keeping the existing lane operational during construction under traffic control. This approach allows for maximum reuse of the existing formation for scour protection and embankment stability resulting in reduced embankment costs and environmental impacts.

The large R500 provides adverse crossfall and maximum sight distance for pedestrian movements at the crossing.

A summary of the horizontal alignment configuration across the alignment can be found in Table 8-1.

Table 8-1 - Summary of horizontal components

Curve ID	Speed (km/h)	Radius (m)	Start Chainage	End Chainage	Super (%)
W1	70	120	30.687	116.968	2.5%
E1	70	-500	228.666	342.024	Adverse

Curve Widening

Applied in accordance with RPDM and AGRD Part 3 Table 7.13. Requirement for 0.6m to be applied to the inside of W1 (R120) for the design vehicle. No curve widening required for E1 (R500)

Design Crossfall Transitions

The design crossfall was developed and applied in accordance with DTMR's RPDM vol 2 and AGRD Part 3. The crossfall transitions at 2 locations (excluding eastern tie in to existing crossfall) throughout the project area, specifically at:

Table 8-2 - Summary of Crossfall Transitions

Rotation ID	Length	Element Type	Start Crossfall	End Crossfall	Comment
T1	30m	Tangent	-0.5 (existing crossfall)	2.5	LHS
T2	30m	Tangent	2.5	-2.5	LHS

Transition T1 begins at the tie in point of the project and extends through the tangent that approaches curve W1 for approx. 30m. This transition length satisfies rate of rotation criteria and relative grade criteria defined in AGRD Part 3 Section 7.7.

Transition T2 is applied in accordance with AGRD Part 3 Section 7.7.10 for a length of. 30m. This transition length satisfies rate of rotation criteria only to assist with reducing aquaplaning risk. Refer section 8.7.2 for aquaplaning risk outcomes.

8.4.4 Vertical Alignment

Vertical alignment design aims to achieve the following:

- Ensuring adequate SSD
- Constructability by keeping vertical curves off the bridge and relieving slabs
- Maintaining adequate grades within Normal Design Domain
- Minimise earthworks requirements
- Maintain min 0.5% longitudinal grade on the structure to assist with drainage without the requirement for bridge scuppers

Table 8-3 - Summary of vertical components

Curve ID	Design Speed (km/h)	Type	K Value	Chainage Range	Grade in / Grade out (%)	Total length (m)
W1	70	Sag	41.554	15.33-65.33	-0.694 / 0.5	50
E1	70	Crest	100	176.17-271.46	0.5 / -0.44	95.3

8.5 Cross Section

The preferred cross-section has been derived from the MBRC Planning Scheme Policy for Living Residential typical cross section. The project was divided into two sections.

- Western Section - from Western tie in to back of trailing relieving slab immediately east of the bridge (Start-176.17).
- Eastern Section - from back of trailing relieving slab immediately east of the bridge to the Eastern Tie in (Ch176.17-End)

Refer to Figure 8-1 and Figure 8-2 for typical cross sections for Western and Eastern Sections respectively.

All shoulder and lane transitions have been applied at 1:50 taper in accordance with TMR's RPDM vol 2. For crossfall discussion refer to Section 8.4.3.

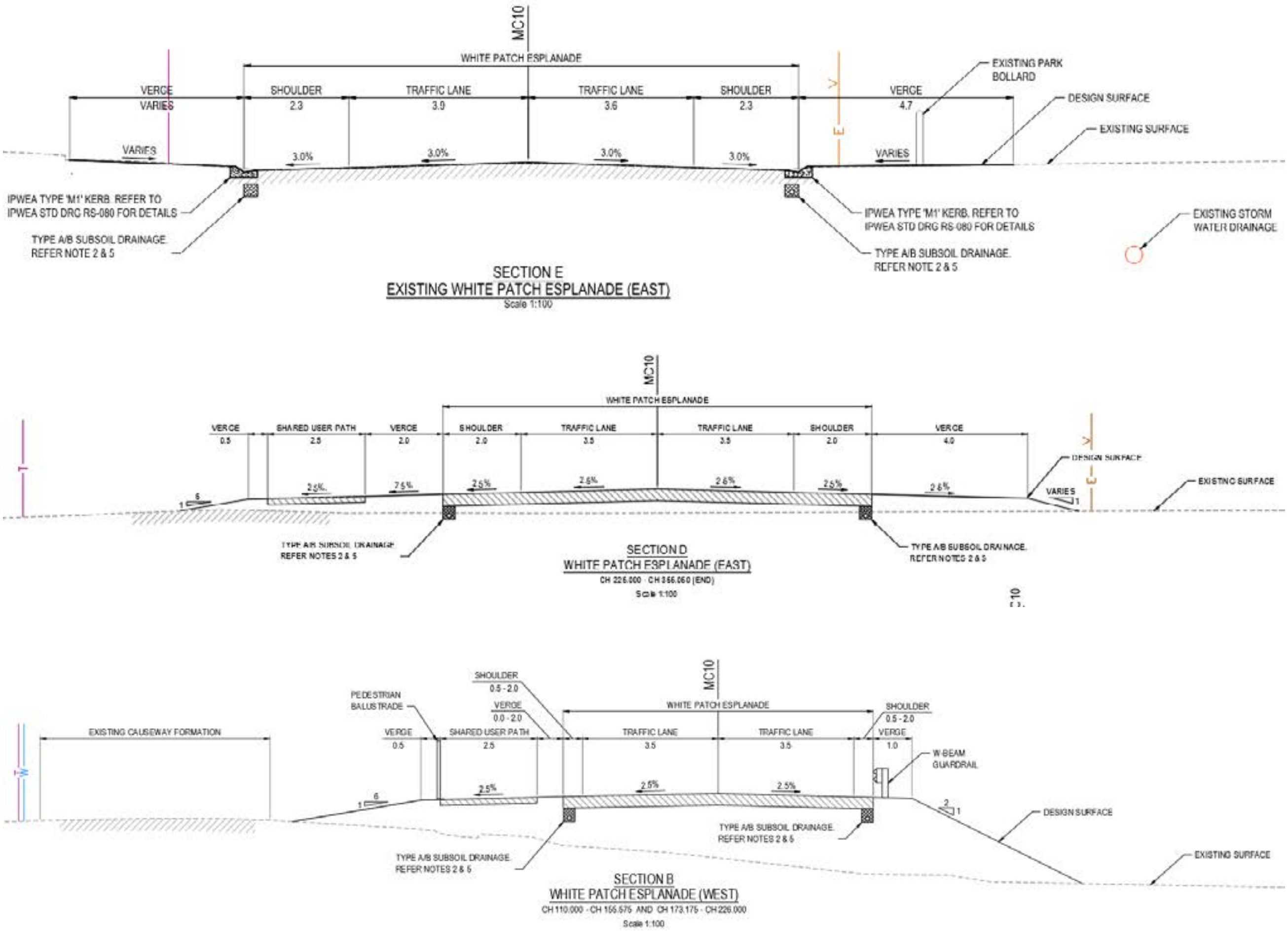
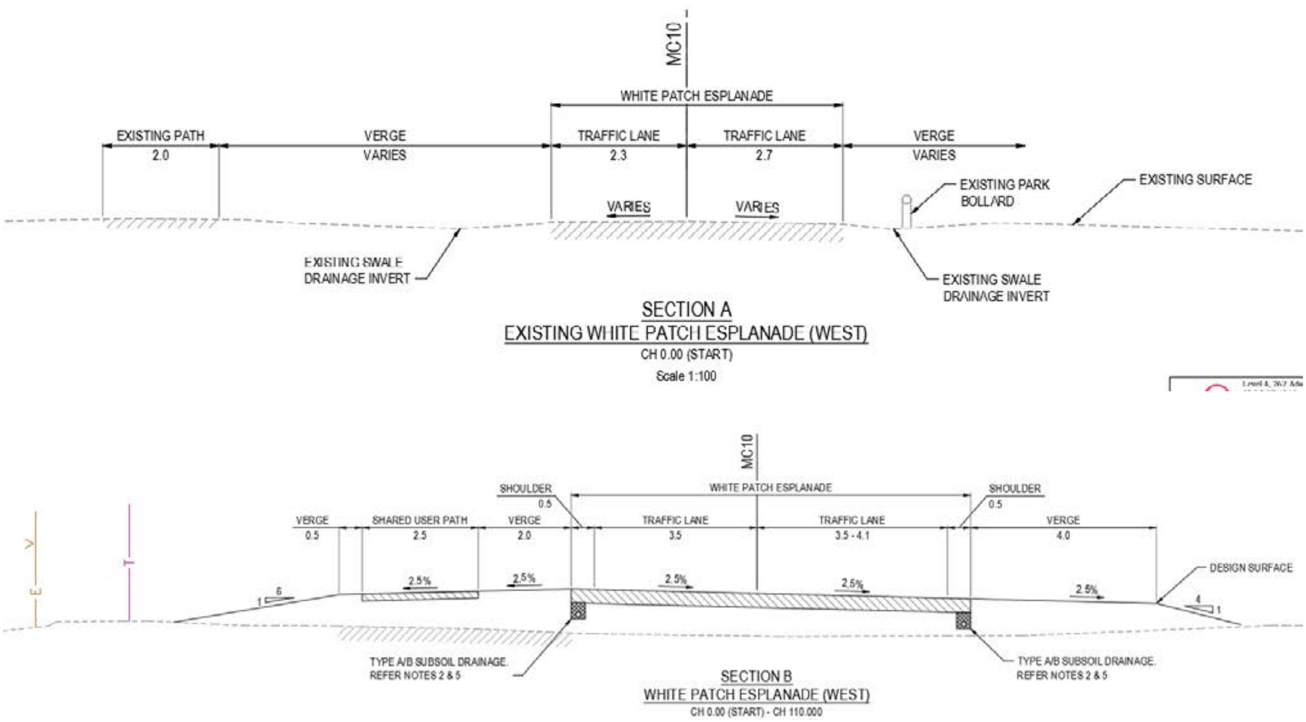


Figure 8-1 Eastern Approach (existing and proposed profiles)



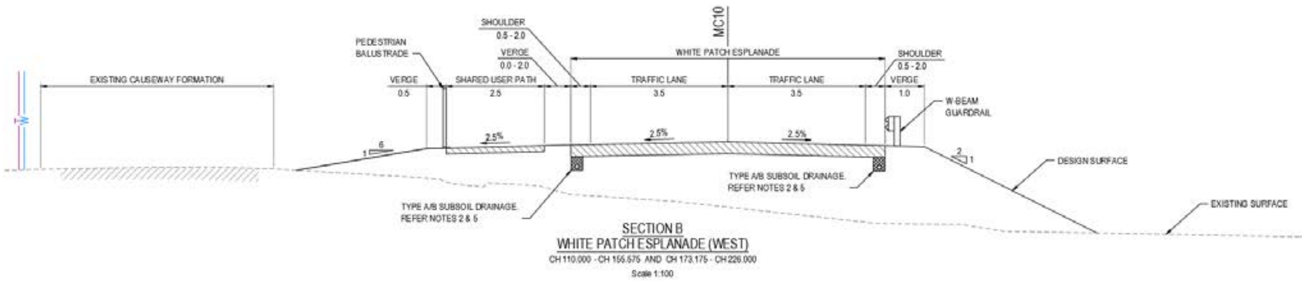


Figure 8-2 – Western Approach (existing and proposed profiles)

8.6 Pavement

8.6.1 Road Pavements

Preliminary pavement design has been assessed based on the criteria defined within the Basis of Report – refer Appendix A – Basis of Design Report Rev B. The relevant design parameters for the design of the pavements have been replicated below.

Table 8-4 Pavement Design Parameters

Parameter	Value	Reference
DESA	$4.55\text{x}10^5$	Provided by MBRC
CBR	5%	Refer section 7.4 of this report

The proposed pavements are as follows

Approach roads - Flexible Pavement:

40mm AC10M C320

150mm Granular Base (Type 2.1)

220mm Granular Subbase (Type 2.3)

Bridge Structure - Deck Wearing Surface:

40mm AC10M C320
Waterproofing seal and S/S seal
AC10M C320 As required
Prime

8.6.2 Shared User Path Pavement

The concrete shared user path as been designed in accordance with IPWEA Standard Drawing RS-065. The asphalt shared user path has been designed in accordance with MBRC Standard Drawing PN-6540 and BCC S310.

The proposed pavements are as follows

Approach Roads - Concrete Shared User Path:

100mm MP32 Concrete
SL72 Mesh (Central)
50mm Sand Bedding

Bridge Structure - Asphalt Shared User Path:

25mm BCC Type 1 Asphalt Surface or DTMR Alternative (AC7M) C170

275mm Granular Subbase (Type 2.3)

8.7 Drainage

Surface flows from the pavement are proposed to be conveyed through sheet flow into the grassed verge areas adjacent the road and onto the embankment batters for discharge to the existing natural surface. At the eastern connection to the existing road, the works will implement IPWEA Type M1 Kerbs which will be an extension of the existing kerb arrangement.

0.5% longitudinal grade combined with kerb on both sides of the carriageway forms the drainage system for surface flows at the bridge. The kerbs are proposed to outlet onto the scour protection at the western side of the bridge.

8.7.1 Subsoil Drainage

Subsoil drainage is proposed at the edge of the pavement boxing and beneath proposed kerb and channel configurations. Refer Appendix F – Preliminary Design Drawings for subsoil drainage details and locations.

8.7.2 Aquaplaning

An assessment of the design surface was undertaken to identify issues regarding aquaplaning. Results can be found in Appendix A – .

There are 2 locations where the risk of aquaplaning is increased which is at the superelevation transitions at curve W1 (R120).

Transition 1 (CH0)

Film depth <3.25mm – no requirement for additional treatments

Transition 2 (CH130)

Film depth <4mm – transitioning from +2.5% to -2.5% with longitudinal grade 0.5%. Superelevation transition shortened to allow for rate of rotation criteria only.

8.8 Signs and Pavement Marking

Line marking design has been developed in accordance with AS 1724 and TMR Manual of Uniform Traffic Control Devices (MUTCD) (Harmonised). The geometry and formation width requires a single barrier line for the length of the project area with corresponding shoulder edge lines (including the bridge).

Refer Appendix F – Preliminary Design Drawings for road signage and pavement marking details.

8.9 Roadside Safety and Barriers

The requirement for road safety barriers was assessed in accordance with AGRD part 6 and TMR's DCBOS with the following outcomes.

W-Beam guardrail has been proposed to transition from the bridge rail on the southern carriageway edge at the approach and departure ends. The guard rail extends for a length of 32m to the west and 40m to the east with approach melt terminals proposed at both ends. The w-beam guardrail is proposed to connect directly to the bridge rail using a standard w-beam to bridge rail connection.

Pedestrian balustrade is proposed to extend from the northern edge of the shared user path located at the bridge, to approximately 15m further to the west and east of the bridge abutments. Additional balustrade is proposed to protect path users from steep batters/drops in this location (batters steeper than 1V:6H).

9 Demolition of Existing Causeway

The existing causeway is proposed to be excavated to RL-1.2m between the bridge abutments to facilitate water conveyance at the opening. This is approximately 270mm below the LAT level to ensure there is water maintained in the crossing. The eastern excavation interface is proposed to be shaped at approx. 45 degrees to allow funneling of flows and reduce turbulence and risk of scour. Refer Appendix F – Preliminary Design Drawings for causeway demolition details.

10 Road Lighting

To be further described in subsequent design stages.

11 Active Transport

A 2.5m wide shared user path is proposed for the extent of the project area. The path connects to an existing footpath to the north/west and terminates at a perpendicular crossing to the east. The crossing connects to an existing footpath on the southern side of the crossing. High angle ramp connections are proposed to the west and east of the structure to allow cyclists travelling east in the road shoulder to transition to the shared user path at the structure. Pedestrian balustrade is proposed at the bridge structure – refer Section 8.9.

12 Utilities/Services

Existing utilities/services are present along the existing crossing. Red Fox Advisory propose to engage service providers in the Preliminary Design Phase to ensure design complies with expectations and to limit risk during construction.

Water main

Unity Water have advised that the water main will be modified from above ground to an underbored solution below the creek. The alignment of this underbore will be 10m to the north of the existing crossing hence is not expected to cause clashes with the new crossing. Red Fox Advisory propose to engage Unity Water in the Preliminary Design Phase to ensure design complies with expectations and to limit risk during construction. Opportunity exists to locate the service at the bridge structure.

Electrical

Energex overhead electrical services are present in the proposed area of works. The new crossing structure is proposed to be constructed below the existing overhead low voltage cables. Line survey has been requested and is expected to provide clearance heights to new construction to inform the consultations and requirement to relocate/raise existing lines.

Telecommunication

Telstra optic fibre cables are present in the project area running adjacent the carriageway for the full length of the project. Opportunity exists to locate the service at the bridge structure if required.

Stormwater

Stormwater services are present within the project area however this infrastructure is located outside the footprint of construction.

13 Environmental Considerations

13.1 Permanent Crossing Structure Works

The following outlines the list of Approvals applicable to the permanent crossing structure works. The removal of the existing causeway structure would be included in the scope of these approvals as required.

Table 13-1 Permanent Crossing Works Approvals

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
EPBC Referral	*Environmental Protection and Biodiversity Conservation Act 1999*	DCCEEW	-	EPBC Referral of the project to be made for MNES items including threatened species and RAMSAR wetlands.
Operational Works – Waterway Barrier Works (Permanent)	*Fisheries Act 1994*	SARA	DAF	The new structure within the tidal waterway is likely to be a waterway barrier and will require assessment against State Code 18
Marine Plant Disturbance	*Fisheries Act 1994*	SARA	DAF	The proposed structure is likely to require the disturbance of marine plants and will require assessment against State Code 11. Offsets are required under the *Environmental Offsets Act 2014*.
Tidal Works / Quarry Material Allocation	*Coastal Management Act 1995*	SARA	DES	The proposed structure involves tidal works and requires assessment against State Code 8. As part of this approval owners consent is required through the Department of Resources' State Land Asset Management (SLAM) for works under the high water mark. The associated quarry material allocation under the Coastal Act is

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
				not required as the structure is an approved tidal work..
Moreton Bay Marine Parks Permit	*Marine Park Act 2004*	DES	NA	The development footprint is within the Moreton Bay Marine Park and requires a marine park permit.
Maritime Safety (Harbour Master)	*Transport (Marine Safety) Act 1994*	SARA	DTMR	For the structure approval may be required with respect to navigation. Advice from Maritime Safety yet to be received.
ERA 16 - Dredging	*Environmental Protection Act 1994*	SARA	DES	Depending on the detailed design for the structure this may be triggered and will require assessment against State Code 23

13.2 Geotechnical Investigations

Approvals that may be required for the geotechnical site investigation works during the design phase of the Project:

Table 13-2 Geotechnical Investigations Approvals

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
Operational Works – Waterway Barrier Works (Temporary)	*Fisheries Act 1994*	SARA	DAF	Geotechnical investigations within the tidal waterway may constitute a temporary (<6mths) waterway barrier and will require pre-work and post-work notification as per the ADR for WWBW, but does not require assessment.
Marine Plant Disturbance	*Fisheries Act 1994*	SARA	DAF	Geotechnical investigations may require the disturbance of marine plants. Minor works (<25m2 of disturbance) are able to be undertaken as per the ADR for Marine Plants. Based on works planned it is unlikely a greater

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
				disturbance will occur requiring assessment against State Code 11.
Tidal Works / Quarry Material Allocation	*Coastal Management Act 1995*	SARA	DES	Geotechnical investigations can be conducted as a 'reasonable excuse' for removing quarry material without an allocation notice. Pre-work notification to DES is required but works are exempt from requiring assessment.
Moreton Bay Marine Parks Permit	*Marine Park Act 2004*	DES	NA	Geotechnical investigations are within the Moreton Bay Marine Park and shall require a minor works level marine park permit.
Maritime Safety (Harbour Master)	*Transport (Marine Safety) Act 1994*	SARA	DTMR	Depending on methodology for the investigations (i.e. use of barge) notification may be required with respect to navigation.

13.3 Construction

Approvals that may be triggered during the construction phase of the Project depending on contractor methodology:

Table 13-3 Construction Contractor Approvals

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
Operational Works – Waterway Barrier Works (Temporary)	*Fisheries Act 1994*	SARA	DAF	The construction methodology may require the placement of temporary (<6mths) structures (e.g. rock platform, sediment curtain) within the tidal waterway is likely to be a waterway barrier that may require notification as per the ADR for WWBW. If the barrier is planned to be in place for

Approval Name	Legislation	Assessment Manager	Advice / Referral Agency	Notes
				>6mths, it will require assessment against State Code 18.
Moreton Bay Marine Parks Permit	*Marine Park Act 2004*	DES	NA	Depending on the construction methodology for the structure, a permit may be required. Included as part of overall permit for the project.
Maritime Safety (Harbour Master)	*Transport (Marine Safety) Act 1994*	SARA	DTMR	Depending on the construction methodology for the structure, this approval may be required with respect to navigation.

14 Safety in Design

A preliminary internal Safety in Design discussion was undertaken on 01/09/22 outlining some key risks within the design. The preliminary risk register developed within this discussion is provided as Appendix D – Safety in Design Preliminary Discussion Risk Register.

It is proposed that during the following phases of the design that a Safety in Design Workshop will be held with relevant parties including MBRC to communicate and explore potential risks within the design.

15 Construction Considerations

To be further described in subsequent design stages.

16 Preliminary Estimate of Costs

As part of the Preliminary Design Phase an estimate of costs was developed using measured quantities of materials from the design models. This estimate will be further refined in the following design phases. Refer Appendix G – Preliminary Estimate of Costs for preliminary costs estimate report.

17 Issues for Resolution

During the Preliminary Design phase, some assumptions were made due to the need for further clarification or detail. A record of these issues can be found in the summary log in Table 17-1:

Table 17-1 - Issue Log

ID	Issue	Category	Description	Status
1	Hydraulics associated with crossing	Hydraulics	Further hydraulic modelling is underway at the time of submission to assess climate change sensitivity to the depths and velocities at the crossing location. The results will be provided the weeks following preliminary submission.	Ongoing

Appendix A – Basis of Design Report Rev B

redfox

Appendix A - MBRC Specification of Services

Appendix B – Aquaplaning Risk Assessment

Appendix C – Matters for Resolution (AS 5100.1)

Appendix D – Safety in Design Preliminary Discussion Risk Register

redfox

Appendix E – Comments Register

redfox

Appendix F – Preliminary Design Drawings

Appendix G – Preliminary Estimate of Costs

Appendix A – Bill of Quantities

Red Fox Advisory Pty Ltd

ABN 25 631 948 135
Level 4, 262 Adelaide St, Brisbane, QLD 4000
GPO BOX 1240, Brisbane, QLD 4001
+61 7 3724 9000
www.redfoxadvisory.com